A MODEL TO EVALUATE VARIOUS UNSIGNALIZED INTERSECTION GEOMETRIES AND OPERATIONS FOR IDENTIFICATION OF POSSIBLE LOCATIONS TO USE IN LIEU OF A TRADITIONAL SIGNALIZED INTERSECTION

by

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__________________________________________
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DEDICATION

This work is dedicated to my late father, Howard George Schrader, one of the many unsung engineering heroes who devoted a lifetime to the engineering profession, making the world a better place.
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CHAPTER 1 “INTRODUCTION”

1.1 General

Traffic signals have a unique cultural place in the United States; they not only serve a utilitarian function, specifically providing for the orderly movement of traffic at intersections, but are also an indication of the importance of a city. The phrase “one-stoplight” town is indicative of the cultural status of the traffic signal. While “one-stoplight town” can be an indication of smallness, for many small communities (Landphair, 2012; Counts, 2013), having a traffic signal means that they are important and busy enough to warrant one (Cichowski, 2013; “Not a one-stoplight town, but...”, 2014). Depending on how a community wants to view itself, as a quaint small-town or an important city, is how that community responds to the presence of traffic signals, with changes in the number of traffic signals creating passions and controversies (Blackburn, 2001; “Public Safety Building...”, 2004).

Given the cultural significance of traffic signals as an indication of a community’s importance, communities that are losing population and no longer need a traffic signal may not be willing to give them up (Blackburn, 2001), even though doing so would provide greater efficiency in the transportation network, and save money on operation and maintenance and fuel wasted stopping unnecessarily (Philipsen, 2014). In larger cities like Detroit, Cleveland, Saint Louis, and Baltimore whose populations have declined by hundreds of thousands in recent decades, the savings created by removing traffic signals that are no longer needed could potentially be millions of dollars annually. According to the Frequently Asked Questions (FAQ) for the signals section of the 2009 Manual on
Uniform Traffic Control Devices (MUTCD), a good “rule of thumb” for signal installation is one signal per 1000 population; using this “rule of thumb”, for cities such as those previously mentioned, hundreds of signals should have been removed as the populations declined. For example, from 1950 to 2010, Detroit’s population has dropped from its peak of 1,849,568; to 713,777, a decrease of over 1.1 million residents, or 61 percent of its peak population (U.S. Census, 2010). Using the “rule of thumb”, one would expect to find 714 signals in Detroit; however, Schrader and Hummer (2015) counted 1510. In other words, Detroit has 796 more traffic signals than what the “1 per 1000 population” rule would indicate. As the annual operating cost of a traffic signal ranges from $1000 to $8000 (United States Department of Transportation, 2007c; Kansas Department of Transportation, 2014; Washington State Department of Transportation, 2014), the cost of operating these additional signals ranges from $796,000 to $6,368,000 annually.

Based on an operational analysis of a representative random sampling of the existing signal inventory, Schrader and Hummer (2015) estimated that 460 signals can easily be removed from the inventory without a significant degradation of service for any approach. These 460 are predominantly at intersections where all approaches have relatively low volumes, and all 460 can be replaced with either minor street stop sign control or all-way stop sign control. To reach the “rule of thumb” ideal signal inventory, an additional 340 signals will need to be removed. Given that many of these signals are located on major arterials with high volumes, simply removing them and replacing them with minor street stop control will cause degradation of service on the minor streets. As communities have been known to resist the removal of signals due to the cultural
significance attached to the signals as opposed to the operational benefit obtained by keeping the signal (Blackburn, 2001), resulting in a bias to maintain not needed traffic signals, overcoming this bias can be a Herculean task when removal of the signal will cause a deterioration of the level-of-service for the street that provides access to the local community. The challenge, then, is to determine strategies to remove these unneeded signals without causing a degradation of the quality of access and level-of-service to the neighboring community. That is the objective of this research.

1.2 Importance of this research

Signals that are not needed have costs to society – operational and maintenance expenses that could be used elsewhere, excessive delay and fuel consumption for motorists, and injuries and deaths due to collisions. Because of these societal costs, it is important to develop strategies to aid in the removal of these not needed signals. However, community resistance to traffic signal removal (Connor, 1988; Blackburn, 2001) necessitates that these strategies do not have a serious negative impact on the local community; specifically, that they do not cause a deteriorated level-of-service for minor street approaches to achieve an improved level-of-service for the intersection overall. The intent of this research is to investigate how to accomplish this.

The cost of routine operation and maintenance of a traffic signal ranges from $1000 to $8000 annually; for a large city with a thousand signals, this cost is in the millions of dollars. For a cash strapped city that is having difficulty provide basic services, this represents a noticeable sum that could be used for other services, such as public safety. If a signal malfunctions and has to be replaced, the cost of replacement could range from
$50000 to $500,000, depending on locale (Cichowski, 2013; City of Tuscon, 2014; Kansas Department of Transportation, 2014; Washington State Department of Transportation, 2014). For a large city with a thousand signals, if five percent of the signals need to be replaced in any given year, the cost of replacement, depending on locale, would be between $2.5 Million and $25 Million, substantial sums.

The 1982 Gerth publication, *Handbook of Traffic Engineering Practices for Small Cities*, provided a method for monetizing delay, specifically, the cost of speed change versus maintaining a constant speed. (Gerth, 1982) Using present day dollars, for 1000 vehicles having to stop daily the cost of the speed change is $25,000 per year (Gerth, 1982; U.S. Inflation Calculator, 2014). For a city with one thousand signals, if every signal has 1000 vehicles per day stopping, the total cost of delay is $25 million. If ten percent of those signals are not needed, then a savings of $2.5 million in delay will be realized. Persaud, Hauer, Retting, Vallurupalli and Musci (1997) analyzed 199 one-way intersections in Philadelphia where signals were removed and found a 24 percent reduction in crashes. According to the National Safety Council, the cost of a property damage only (PDO) collision in 2012 was $8,900, a disabling injury crash, $78,900, and a fatality, $1,410,000 (National Safety Council, 2014). In 2014 dollars, those costs are $9,200, $81,900, and $1,463,000, respectively (U.S. Inflation Calculator, 2014). If just ten crashes are eliminated, then the potential cost savings ranges from $920,000 to $14,630,000, depending on the crash severity.

To maximize the benefits of removing not needed traffic signals, it is crucial to minimize the costs of the removal, both the quantitative monetized costs and the
qualitative social costs. The quantitative monetized costs include the physical costs of removing the not needed signals, including materials, rights-of-way, construction costs, delay, and the like. The qualitative social costs such as speeding, pedestrian and bicycle access, neighborhood interaction, property access, diversion of traffic, and similar considerations should be recognized when analyzing various strategies for removal of non-needed signals.

For example, roundabouts are one such strategy, and have been recommended by several studies as an alternative to signalized intersections (Wallwock, 1997; Moscovich, 2003; Turner, 2004; Retting, 2006; Nambisan & Parimi, 2007; Mery, 2009; Cruz, 2010; Voigt & McCombs, 2010; Chou & Nichols, 2014). While roundabouts have benefits such as signal operation cost savings, reduced crashes, reduced delay, and reduced speeds, they are expensive to build, as they require substantial rights-of-way to construct. In 2000, the United States Department of Transportation estimated that the average cost of a roundabout was $250,000 (U.S. Department of Transportation, 2000), which is approximately $350,000 in 2014 dollars (U.S. Inflation Calculator, 2014).

To minimize construction cost and time and overall environmental impact, it is important to evaluate strategies that do not require acquisition of new rights-of-way or contribute to an increase in impervious area (Khanna, 1993; Kalhammer & Bellela, 2001; Dock, Zimmer, Becker & Abadi, 2006). There are four common strategies that have been used that fulfill this requirement that have not been evaluated extensively for their effectiveness when used as an unsignalized alternative to a signalized intersection – raised medians, Median U-turns (MUTs), Redirected Crossing U-Turns (RCUTs), and Indirect Lefts
and Throughs (ILACs) – as they have traditionally been viewed as strategies for enhancing signalized intersection performance, and not for conversion of a signalized intersection to unsignalized operation.

1.3 Research overview

Currently, the concepts of “signal removal” and “alternative intersections”, are, for all practical purposes, independent of each other. There is a sizable body of literature and policy on removing not needed traffic signals, which are traditionally in declining urban areas. Likewise, there is a sizable body of literature and policy on alternative intersections. However, there is a dearth of research on using alternative intersections as a strategy to remove not needed signals in declining urban areas, as such geometries have not been used on low-speed urban corridors, even busy ones. While voluminous literature exists pertaining to replacing signalized intersections with roundabouts, a type of alternative intersection, and using unsignalized RCUT and ILAC intersections on rural expressways in lieu of signalization, the base assumption with this literature is that signals are needed. Research is lacking in using alternative geometric designs to eliminate not needed signals. This research will contribute to that limited body of knowledge.

Given that signals can easily be removed by simply removing the hardware and installing stop sign control where needed, why, then, the need to delve into the use of alternative intersection design to accomplish this? First are the historical and social issues. As Detroit was established by the French, its layout is very French in nature, with property being subdivided to maximize the number of parcels with access to the river, resulting in long and narrow lots, with the long axis running perpendicular to the river. (Parkins, 1918;
Base, 1970; Schrader, 2014b). Because of this property configuration, there are more streets running perpendicular to the river than parallel to it, with these more numerous streets being smaller; conversely, since there are fewer streets running parallel to the river, these streets tend to be wider and busier, with much of the traffic on these streets either originating in or traveling to locales outside of the City of Detroit. If signals are removed from intersections of these smaller streets that run perpendicular to the river and the larger streets that run parallel to it, the larger street would be assigned the right-of-way, with the consequences of cutting off access of the neighborhood further away from the river from the neighborhood closer to the river (and ultimately, the river itself), and giving priority to traffic produced outside of the city over traffic produced within it. Either of these consequences could be used to justify maintenance of the signal, and could be used to argue against signal removal. Alternative intersections, by their very design, would help mitigate these negative consequences, by reducing the number of lanes that would need to be crossed, preserving and possibly even enhancing connectivity between the neighborhoods on both sides of the larger street.

A second reason to explore the use of alternative intersections to facilitate signal removal instead of simply removing the necessary hardware is environmental. By design, alternative intersections reduce the amount of pavement at an intersection with the use of turf medians, with the benefits of increasing green space, improving aesthetics, and reducing runoff. A review of papers by Waller (2014) revealed how treatments such as alternative intersection designs have a positive impact on these environmental issues, and that policy-makers value these positive impacts. For a city such as Detroit which
historically has struggled with drainage issues (Parkins, 1918), any strategy that can reduce runoff is worthy of serious consideration.

The objective of this research consists of two outputs. The first of these is to compare an overall impact analysis of several signalized intersections along an urban travel corridor with an overall impact analysis of the same intersections with the signals removed using a variety of context sensitive alternative intersections designed for the specific existing geometric and land use considerations of each location, with said designs containing one or more of the four concepts previously mentioned (raised medians, MUT, RCUT, ILAC). This particular analysis was of a 5.5 mile corridor in Detroit with 17 signalized intersections considered for conversion to unsignalized operation. This overall impact analysis included two different and unique types of operational performances – behavioral (i.e. those controlled by the driver) and non-behavioral (those out of the driver’s complete control). The two notable and most important behavioral performances are delay (a driver’s delay is affected by decisions that that driver, and other drivers, make), and travel time (the driver chooses the travel path and speed he or she uses). Delay performance was measured, for each operational condition, for each approach and then aggregated, with the aggregations used for overall comparison of the changes with respect to each other. Travel time performance was measured, for each operational condition, for each movement whose travel path is affected by the geometric changes implemented and then aggregated, with the aggregations used for overall comparison of the changes with respect to each other. Non-behavioral performances are those in which the driver’s choices and behaviors, for all intents and purposes, are irrelevant (i.e. these
performances are the same if all drivers were removed and all vehicles become automated and driverless). The two non-behavioral performances included are also environmental in nature. They are: stopping impacts created by repetitive vehicle deceleration, braking, and acceleration (e.g. air pollution, vehicle wear and tear), and runoff, impacts created by the change in pervious area (e.g. water pollution, flooding) with the implementation of the alternative geometric designs. Both of these environmental operational performances are holistic, i.e. they were measured for the intersection as a whole.

The second output is a pedestrian safety verification of the changes; in other words, for each modification, an analysis is performed to determine if the design provides adequate and sufficient crossing opportunities for pedestrians. For each modification, the number of acceptable pedestrian gaps was calculated, and this value was then compared with actual pedestrian crossings; where the gaps exceeded the actual number of pedestrians crossing, the proposed modifications did not have an impact on the ability of pedestrians to cross and were considered acceptable. To aid in the application of this method, a design matrix was created enumerating the total traffic volume that can accommodate selected pedestrian volumes for various pavement widths. Specifically, six different pavement widths were used: 20, 22, 24, 30, 33, and 36 feet, reflecting the use of 10, 11, and 12 foot lanes for two and three lanes of traffic. This matrix can be used by a designer contemplating a conversion of a traditional signalized intersection into a non-traditional one to determine the practicality of such a conversion prior to delving into the in-depth analysis. In short, a designer would use the second output to determine the
practicality of using the methodology of the first. (Details of the research objectives are provided in CHAPTER 2.)

If the changes were simply removing signals, bicycles, transit, and heavy vehicles would not necessarily be needed to be included in the input, as such changes would not alter the physical environment of the roadway network and most likely would not alter the demand of these types of motorized and non-motorized vehicles. However, since this particular methodology entails changing the physical environment, the demand of transit, bicycles, and heavy vehicles to use this intersection may or may not change. Not only is it indeterminate whether or not these demands will change, the attributes of these changes (e.g. increases or decreases) and the magnitude of these changes are indeterminate as well. Because of the plethora of possibilities, it is not practical to include these demands in the input for the methodology, as any values used would be speculative at best and would call into question the validity of the output. (The needs of bicycles, transit, and heavy vehicles were treated qualitatively as much as possible.) The most effective way to reduce the entropy of these demands is through the collection of before and after data at intersections that have been modified to determine, with empirical evidence, the impact of such modifications on these demands.

A mathematical model or methodology is only as useful as its applicability. One of the overarching parameters of this research was the creation of such a model that is ubiquitous. In other words, not only was it important to create a methodology to analyze the replacement of traditional signalized intersections with unsignalized alternative ones in Detroit, Michigan, but also to create a methodology that can be used in Baltimore,
Cleveland, Akron, Huntington, Saint Louis, and any other city that has experienced significant depopulation. With this research, these locales, and other similar ones in the United States and elsewhere, will have a model to help them make better use of limited and scarce financial resources, as well as help create a blueprint for future redevelopment.
CHAPTER 2 “RESEARCH OBJECTIVES”

2.1 Overall paradigm

The objective of the research is to determine if selected unsignalized intersection geometries can be used in lieu of traditional signalized intersection geometries without significantly degrading the operations and safety on all the approaches, as well as the social and environmental quality of the intersection locale. Specifically, this research will focus on urban signalized intersections on an arterial with heavy through traffic and few turning movements whose primary function is expediting through traffic along a travel corridor, with little interaction with adjacent neighborhoods. Some of these types of signalized intersections, when removed and replaced with a traditional two-way stop controlled intersection, can experience improvements in the operation (as measured by delay) of not only the major street but also the intersection overall, while simultaneously experiencing a degradation in the operational performance of one or more of the minor street movements.

An example of a location where this particular phenomenon often occurs is where the imbalance in traffic volumes between the major street and the minor street is extreme (e.g. a multiple of five or greater), with few drivers desiring to turn from the major street onto the minor street or to proceed straight through on the minor street. At this particular type of intersection, traffic from the local street desires to turn onto the arterial, and traffic on the arterial is desiring to continue on the arterial. The primary purpose of a signal at this type of location is to facilitate left turns from the local street onto the arterial by creating gaps in the arterial traffic stream large enough and frequent enough for these
turns to be completed safely. The drawback of such a signal is that it reduces the capacity of the arterial; since the arterial through volumes are much higher than the left turn volumes on the local street, these signals result in an overall lower performance of the intersection than what would be if the signals were not there, as well as make two-way progression along the arterial practically impossible as they are not spaced with two-way progression in mind. Despite this, if the signals were not there and the volumes on the arterial were high enough, vehicles attempting to turn left onto the arterial may not be presented with enough safe gaps, resulting in long waits to complete the turn. Long waits can result in motorists becoming impatient and taking gaps that are less than the size required to complete the turn safely, which may lead to more collisions. (Kyte, Clemow, Mahfood, Lall, & Khisty, 1991) Thus, there is a need to evaluate alternative geometric designs that will allow agencies to remove these signals without causing situations where motorists are willing to engage in risky driving behaviors.

In addition to considering minor street left turns in evaluating unsignalized intersection performance when compared to a traditional signalized intersection, other factors, such as pedestrians, transit, parking, and adjacent land use, must also be considered. Signals provide pedestrians with a protected crossing, and any attempt at removal should be cognizant of pedestrian volumes, needs, and capabilities, and the interaction of pedestrians with the physical roadway environment. In many urban areas, transit routes will loop through residential neighborhoods instead of staying on the major arterials; in these instances, the ability to maneuver on to and off of the minor streets should be in the calculus prior to even seriously considering removing signals. If transit
stops are located at the signal under investigation, the impact on both vehicles and pedestrians if the signal were to be removed should be explored. Vehicles can park up to a signalized intersection without necessarily degrading safety, as all directions receive a protected crossing due to the signal. This is not the case with an unsignalized intersection, where parked vehicles can and often do cause a sight distance obstruction that creates an unsafe condition for vehicles entering the intersection. Thus, conversion of a signalized intersection to unsignalized two-way stop control necessitates the restriction of parking to prevent the creation of blind spots, thus resulting in a net loss of parking spaces. In locations where parking demand exceeds parking supply, this reduction can be problematic. Finally, whether or not an intersection is signalized or not frequently dictates compatible land uses. Heavy commercial uses, such as a big box retailer that generates significant traffic, and heavy industrial uses, i.e. ones that require large numbers of heavy vehicles, are frequently more compatible with a signalized intersection than an unsignalized one.

For the purpose of this research, traditional signalized intersection geometries are those in which the approaches on the major street (the one with the highest volume) either have no median or a flush median. Furthermore, the selected unsignalized intersection geometries tested all fit within the existing right-of-way, thereby eliminating the necessity of land acquisitions or building demolition in order for them to be implemented. Each of the locations analyzed had a unique, context-sensitive unsignalized geometry that may or may not have been like any other intersection. Nonetheless, all of the unsignalized configurations studied fell into one or more of three basic concepts, or
variants thereof: the Median U-turn (MUT), the Redirected Crossing U-turn (RCUT), and
the Indirect Left and Crossing (ILAC). While a variant of the roundabout, the mini-
roundabout, can fit within the existing rights-of-way, the mini-roundabout is not an
appropriate geometry for a multi-lane facility, especially one with heavy vehicle traffic;
thus, it was not considered.

2.2 Scope of this research

The objective of this research is to create a methodology to determine the
monetized benefits, if any, of replacing signalized intersections with unsignalized ones,
and specifically, non-traditional, or alternative, unsignalized configurations, along an
urbanized and built-out travel corridor. In order to make this methodology useful beyond
the scope of this particular study, it should be universally applicable, i.e. should be able to
be successfully used at any location anywhere. Because every intersection is unique, this
methodology must be flexible and adaptable enough to be applicable at every
intersection, regardless of specific localized constraints. Finally, it must be straightforward
enough to be easily understood by practitioners and policymakers of varying levels of
technical knowledge.

Numerous different analysis and evaluation methods are available and have been
detailed. Of these methods, the one that best fits the last specification enumerated in
the previous paragraph is monetization of benefits and costs. As budgets are a key factor
in determining which projects are completed and which are not and are ubiquitously
understood, it is logical to express the desirability of a project in budgetary terms.
Monetization allows for easy prioritization, as well as for ease in modifying the
comparison methodology to account for unique local conditions.

Secondary to the monetized benefits and costs of replacing signalized intersections with alternative unsignalized ones, but nonetheless important, are social costs of these changes. Will the changes degrade pedestrian and vehicle access to adjoining properties? Will they increase speeding? Will they discourage pedestrians? Will they increase traffic in the neighborhoods? Just because a change may result in monetary savings does not necessarily mean it should be done if it has social costs; thus, it is imperative to investigate these costs.

As signalized urban intersections typically function as part of overall travel corridor, to truly gauge the effectiveness of replacing a signalized intersection with an unsignalized one, the changes must be evaluated within the proper spatial context; in other words, as part of the overall corridor. To aid this contextual evaluation, all the intersections selected for evaluation as part of this research will be part of a corridor, to allow for analysis on both the individual intersection level as well as the comprehensive corridor level. Previous research by Turner (2004) and Isebrands (2009) has shown that different geometries may be used on the same corridor without a degradation of overall performance; thus, each intersection within the corridor may have different geometries. This flexibility is important because no two intersections, even on the same corridor, are exactly the same, especially on older urbanized corridors.

2.2.1 Alternative intersections: roundabouts

As stated previously, the objective of this research is to create a methodology to determine the monetized benefits, if any, of replacing signalized intersections with
unsignalized ones, and specifically, non-traditional, or alternative, unsignalized configurations along an urbanized and built-out travel corridor. Because the corridor is urbanized and built-out, the signalized intersection can only be replaced with a modified unsignalized alternative intersection design that fits entirely within the existing rights-of-way, as expanding beyond the right-of-way has a real possibility of requiring the acquisition and demolition of existing structures. This possibility not only increases the cost of implementing the change, with a strong likelihood that such a change will not economically feasible, but also increases the social costs as well, as demolition of structures often results in the decimation of a neighborhood and destruction of its cohesiveness and character. Roundabouts are a ubiquitous alternative unsignalized intersection; however, they frequently require significant additional rights-of-way that often require the costly demolition and replacement of structures, both super and subterranean, and thus the practicality of using them in a built up urban corridor is limited. (It should be noted that other types of alternate geometries may require the acquisition of rights-of-way for the construction of loons and similar enhancements, but these acquisitions tend to not require the removal of structures.) Additionally, on a street network of an older city like Detroit, with many smaller parallel streets running one direction and less numerous and wider streets running the perpendicular direction, traffic volumes are imbalanced with the heavier loads on the less numerous streets, which is not conducive to the use of roundabouts and similar alternative intersections that function poorly with imbalanced traffic volumes. Because of these the limited application of a roundabout, it will not be considered as a viable alternative to a signalized intersection in
a built-up urban corridor.

2.2.2 Alternative intersections: raised medians

The construction of a raised median has been documented to improve safety and performance. (Bowman & Vecellio, 1994; Jacobs, Rofé & Macdonald, 1994; Jacobs, Rofé & Macdonald, 1995; Bonneson & McCoy, 1997; Schrader, 2002b; Durrell, 2007; Abdelgawad et al., 2014; Alluri, Gan & Haleem, 2014) While some may argue that an intersection with raised medians is an alternative design due to the access limitations of the medians, practically, it is not. A raised median does not change the geometric configuration or operation of an intersection; thus, a traditional intersection with raised medians is functionally the same as one without them. The safety and operational improvements that result from the use of raised medians is not a result of reducing conflict points in the intersection, but improving guidance and channelization. Alternative intersections not only provide the operational and safety improvements derived from improved guidance and channelization, but also from reducing types of conflicts in the intersection; thus, only installing raised medians is not an option that is included in the study.

2.2.3 Alternative intersections: Median U-turn (MUT)

As stated previously, all of the alternative designs studied were, or had functional components of, three basic design types, of which the MUT is the first. As previously stated, all designs must be able to fit within the right-of-way to eliminate the costly and time consuming acquisition of property, and the MUT can, and has been, built within existing rights-of-way of many urban arterial corridors, especially those planned to
accommodate railroads, median parking, a future freeway facility, or intended and designed as an urban parkway. The MUT entails the construction of two U-turn crossovers in the median of the major street, one on each side of the intersection. These median U-turns can be used for left-turns from the major street to the minor street, the minor street to the major street, or both. Since the minor street approaches at urban intersections where signal removal should be considered are typically not wide enough to accommodate medians, such an installation on minor streets is, for all intents and purposes, not practical, and thus not considered. An operational schematic of an MUT is shown in FIGURE 2.1.

As can be seen from FIGURE 2.2, an MUT can function in three different ways. With the MUT-S operational scheme, side street left turns are prohibited at the intersection proper; motorists desiring to turn left from the side street onto the through street must turn right and proceed downstream to a U-turn facility, and then proceed upstream back through the intersection in the desired direction. This scheme is useful at locations with heavy through and light left turn volumes on the through street, heavy through volumes on the side street, and left turns must share a lane with through and

![Diagram of Median U-turn (MUT)](image)

**FIGURE 2.1.** Operational schematics for Median U-turn (MUT)
right movements on the side street approaches. Under these circumstances, a left turning vehicle would impede the flow of the through movement on the side street while waiting for not only the through street traffic to clear, but also the through traffic on the side street to clear as well, as left turns from the side street have the lowest priority of movement of any of the movements at an unsignalized intersection; thus, eliminating that movement and turning it into a higher priority movement, right turns, will improve traffic flow. While theoretically the MUT-S scheme improve traffic flow at an unsignalized intersection, in reality it is not a practical solution; side street left turns would be using an “honor system”, and with no physical barrier to making left turns, there is a high probability that many side street left turns would not use the indirect left turn and would choose to illegally turn left at the intersection, as is a common occurrence along MacCorkle Avenue in Kanawha City, West Virginia, where left turns are prohibited from the side streets to improve traffic flow.

Under the MUT-T operational scheme, left turns are allowed from the side street, and through street left turns are prohibited at the intersection. Through street motorists
wanting to turn left from the through street onto the side street must proceed through the intersection, utilize the downstream U-turn facility, and then proceed upstream back to the intersection and turn right to complete the maneuver. This scheme is useful at locations where side street lefts have a separate lane, side street through movements are minimal, the side street has no heavy vehicles, and the through street median is deep enough to safely store one vehicle. One variant of the MUT-T is the Reversed Redirected Crossing U-turn (RRCUT). Because of these specifications, this application of an unsignalized MUT-T operational scheme is quite limited.

The final MUT operational scheme is the MUT-U. Under this scheme, all left turn are prohibited at the intersection proper. Left turning vehicles on the through street must proceed through the intersection, utilize the downstream U-turn facility, proceed back upstream to the intersection, and then make a right onto the side street to complete the maneuver. Left turning vehicles on the side street must turn right onto the through street, utilize the downstream U-turn facility, and then proceed upstream in the originally intended direction. Like the MUT-S, the MUT-U is not really practical for unsignalized intersections due to the lack of physical barrier enforcing the prohibition of left-turning movements at the intersection proper from the side street.

2.2.4 Alternative intersections: Redirected Crossing U-turn (RCUT)

The second alternate geometry is the RCUT. The RCUT is more restrictive of minor street movements than the MUT because unlike the MUT, the RCUT does not allow any minor street movements that would cross a major street through movement; minor street traffic wanting to go straight through or turn left must turn right and make a U-turn at the
downstream median openings in order to complete these movements. (FIGURE 2.3) Conversely, the RCUT is less restrictive of major street movements than certain operational schemes of the MUT, as the RCUT permits all major street movements to be made directly at the intersection, while often the major street left turn must travel through the intersection and then U-turn downstream to complete the movement at an MUT. Thus, while the RCUT redirects more total movements than the MUT-T (FIGURE 2.4), it does not impact through street traffic movements as only side street movements are redirected.

FIGURE 2.3. Operational schematics for Redirected Crossing U-turn (RCUT)

FIGURE 2.4. Comparison of redirected movements at four-legged MUT-T and RCUT. (Redirected movements shown in gray.)

2.2.5 Alternative intersections: Indirect Left and Crossing (ILAC)

The final alternative geometry, the ILAC, is the most restrictive, as it combines the indirect movements of the MUT and the RCUT and allows the fewest direct movements.
With the ILAC design, no vehicular movements are allowed across the intersection except for through movements on the through street. (FIGURE 2.5) In other words, there are no direct left turns from any direction and no direct through movements from either of the minor approaches. (FIGURE 2.6) The ILAC only allows six of the possible twelve movements at a four-legged intersection to be made directly – all four right turn movements and the through movements on the two through street approaches.

FIGURE 2.5. Operational schematics for Indirect Left and Crossing (ILAC)

FIGURE 2.6. Redirected movements at four-legged ILAC. (Redirected movements shown in gray.)

2.3 Summary of purpose and scope

Although roundabouts are commonly used in lieu of traditional signalized intersections, they are not included in this study. Roundabouts have been used in the
United States for two decades, and thus have been extensively studied; thus, including roundabouts will not add anything significant to the existing body of knowledge regarding their performance. Furthermore, there are limited locations where a roundabout can be installed on existing right-of-way, so the practicality of their use on built-up urban corridors is limited. While both the RCUT and the ILAC have been used in some form for decades, their usage has been less widespread and less studied, with the available literature limited in scope. The Median U-Turn is used commonly at high-speed, high-volume intersections in suburban Detroit and other parts of Michigan, and are typically signalized; an unsignalized application of the any operational scheme of the MUT is unusual. The RCUT is most commonly used on high speed suburban arterials and is typically signalized, allowing for each direction of the arterial to be timed independently of the other, functioning as a one-way couplet and not as a two-way facility, allowing for perfect progression in both directions along the arterial corridor. The RCUT has also been used as a safety countermeasure at unsignalized intersections on high-speed rural expressways. The scope of this research is to analyze the effectiveness of these designs at unsignalized intersections on low-speed urban streets with land use and right-of-way restraints, a unique application of these concepts. In other words, the scope of this research is to answer the following question — it is cost effective to replace a traditional signalized intersection in a built-out urban corridor with an alternative unsignalized MUT, RCUT or ILAC intersection (or variant thereof)?

To answer this question, the approximately 5.7-mile corridor of East Jefferson Avenue between Rivard and Alter in Detroit was studied. There are several reasons for
the selection of this corridor. First, it is a heavily used commuter corridor, with high through traffic volumes and minor interactions with the surrounding neighborhoods. Second, it has twenty-five signalized intersections with many different characteristics, thereby providing a broad sample for study. Third, Jefferson does not have medians at twenty-four of the twenty-five intersections (the intersection of East Grand being the exception), and therefore the social effect of installing raised medians can be ascertained. Fourth, Jefferson is of sufficient width that the geometries to be tested can fit within the existing right-of-way footprint. Fifth, Jefferson is not adjacent or parallel to a freeway along this corridor, thus, the operation of this corridor is, for all intents and purposes, unaffected by traffic conditions on the freeway system, resulting in consistent and steady traffic volumes. While Jefferson does intersect a stub freeway at the western end of the corridor, the stub functions as a free-flow movement to and from Jefferson west of this corridor and does not directly impact this corridor. In other words, one can be confident that a traffic volume sample collected on Jefferson is a good representative of the typical traffic volume, and not an anomaly caused by a malfunction or change elsewhere in the transportation network, particularly on one of the freeways.

To facilitate the successful completion of this analysis, and the make the methodology as utilitarian as possible, a mathematical model will be used. This mathematical model will attempt to objectify subjective elements such as walkability to provide results that are duplicable and not subject to personal preference. This model will illuminate the impacts of converting traditional signalized intersections to alternative unsignalized ones with respect to factors such as environmental impact, traffic flow, cost,
and neighborhood cohesion as a mathematically objective value. Most importantly, this methodology will be context sensitive, meaning that it will applicable to any design and any existing geometric, access, and development conditions. A model is only as valuable as the applications for which it can be used; by making the methodology utilitarian, this model can be used in a broader range of locations.
CHAPTER 3 “LITERATURE REVIEW”

3.1 General

It is challenging to categorize the literature pertaining to the replacement of not-needed signalized traditional intersections with unsignalized alternative ones due to the complexity of the task itself. This task is essentially a compounding of two independent tasks - signal removal, and the replacement of a traditional signalized intersection with a non-traditional one - to create an entirely new concept. These two tasks can be categorized as operational (signal removal) and design (intersection geometry), with these two categories often viewed as mutually exclusive. The literature generally pertains to either operation or design. While operational literature will mention design, and vice versa, in the available literature one category will predominate over the other, depending on the point-of-view of the author. In other words, most of the literature views operation and geometry as completely independent of each other, and mutually exclusive. The notable exception to this is found in literature pertaining to roundabouts, as roundabouts, by their very nature, assume an interdependence and interaction between operation and design, as the desired level of operation determines the design, and the design determines the level of operation.

While the literature generally pertains to either operation or design, certain topics within the literature pertain to both, although not simultaneously. For example, literature pertaining to intersection safety is written from both points-of-view (although not simultaneously) – safety can be improved by operational changes, or by geometrical changes. Similar phenomena occur with respect to pedestrians, bicycles, capacity, and
speed. The challenge, then, is to synthesize similar topics from these independent categories. This literature review will attempt to create such a synthesis. Thus, instead of grouping the literature into the two broad categories of operations and design, the literature will be grouped topically.

3.2 Safety

Because of the importance of crashes to policymakers and the general public, much of the literature that references operations or geometric design pertains to crashes and safety issues. The safety literature can be broken down into three subcategories – effects of operational changes on safety, effects of geometrical changes on safety, and effects on simultaneous operational and geometrical changes on safety. These three subcategories can further be broken down into sub-subcategories. For example, operational changes that can impact safety include turning movement restrictions, signal phasing, cycle lengths, and mode of operation. Correspondingly, lane width, intersection width, lane configuration, and the presence and type of median are all geometric features that impact safety. The combinations of operational and geometrical changes that can affect safety are practically limitless.

3.2.1 Operational changes that affect safety

As mentioned, there are several different operational changes that can be made at a signalized intersection that will affect safety. The most extreme of these is also the most relevant to this research – total signal removal. One of the seminal works regarding the safety effects of signal removal is by Persaud, Hauer, Retting, Vallurupalli and Musci (1997). As previously detailed, Persaud et. al. studied the removal of 199 signals at one-
way intersections in Philadelphia, and found a 24 percent reduction in crashes. This study has since been alluded to in subsequent major studies by the Insurance Institute for Highway Safety [IIHS] (Redding, Chapline & Williams, 2000), the Institute of Transportation Engineers [ITE] (2003) and the National Cooperative Highway Research Program [NCHRP] (Antonucci, Hardy, Slack, Pfeffer & Nueman, 2004), and The Crash Modification Factors Clearinghouse (U.S. Department of Transportation, 2014). Although previous literature on signal removal, such as Criteria for Removing Traffic Signals (Kay, Neudorff & Wagner, 1980), User Guide For Removal Of Not Needed Traffic Signals (U.S. Department of Transportation, 1980), Handbook of Traffic Engineering Practices for Small Cities (Gerth, 1982), Do we really need traffic signals? (Orcutt and Sullivan, 1991), and previous literature discussing safety at signalized intersections such as Traffic engineering – myths and realities (Burritt, 1990), and the MUTCD (1978, 1988), postulate that the unwarranted signals increase crashes (with the corresponding deduction that removing unwarranted signals reduces crashes), this effect wasn't definitively and decisively proven until the publication of the Persaud et. al. study. Retting (2006) studied red light running crashes and concluded that the elimination of traffic signals is an effective countermeasure to this problem, thus validating the results of the previous Persaud et. al. study. Since it has been proven that removing unwarranted signals can reduce crashes under certain circumstances such as replacing them with all-way stop control, this increases the importance of endeavoring to remove unwarranted signals wherever possible.

Other researchers have found correlations of other operational parameters with safety. Almonte and Abdel-Aty (2010) found that delay, cycle length, and volume all have
direct relationships with the number of crashes; specifically, that as either delay, cycle length, or volume increases, so do the number of crashes. Fleck and Yee (2002) concluded that something as simple as allowing right turns on red reduces crashes. Nije and Talens (2001) suggest that making road categories more obvious and discernible to motorists will improve safety. Box (1995), with his recommendation that signals should only be used on certain classifications of streets, also alludes to this safety benefit. According to Ostensen (2003), educating motorists is a strategy to reduce crashes and improve safety; Kwasniak and Kuzel (2009) also share this opinion. In addition to his segregation of classifications, Box (2002) establishes a direct correlation between the presence of on-street parking and crashes; however, Box's results should be used with caution as they apply to specific circumstances and may not be applicable to all. Duncan (2008) delves into driver psychology, and suggests a relationship between the appearance of the roadway environment and how recklessly motorists drive, e.g., that a disorderly road encourages apathy and lawlessness by the user. Thus, a way to improve safety is to ensure that everything is clean and tidy and in good working order. Ibarguen and Kar (2009) come to a more abstract connection between operations and safety, namely that reducing the number of conflicts improves safety. As there are a multitude of operational changes that reduce conflicts, one being eliminating left turns, there are a multitude of operational changes that can improve safety. One such change is the Quasi-Couplet (Q-Couplet), also known as the unbalanced flow couplet (UFC), where two parallel two-lane two way streets are modified to operate like a one-way couplet (Hummer, Reese, Dunlap, Harden, and Schrader, 2017) with the introduction of an unequal number of lanes in each direction,
the safety benefits being the reduction of rear end crashes by providing space for vehicles to safely bypass stopped left-turning vehicles as well as the reduction of potential red light running with simplification of signal phasing schemes.

One operational change that has been found to improve safety, albeit temporarily, is reducing statutory speed limits. Lindenmann (2005) studied residential streets in urban areas in Switzerland and determined that reducing speed limits reduces crashes and improves safety. Grundy, Steinbach, Edwards, Wilkinson, and Green (2008) tested Lindenmann's conclusions with a detailed analysis of the numerous zones in London where speed limits had been reduced, and while they, in general, concurred with Lindenmann, they cautioned that the speed and corresponding crash reductions dissipate over time. This same dissipation phenomenon was observed by Schrader (1999) in his study of school speed zones. Lv, Lord, Zhong, and Chen (2013) noted the same dissipation pattern with the Peltzman Effect, in which people who feel safer engage in riskier behavior, resulting in statutory safety measures, such as seat belts, resulting in a degradation of safety due to riskier and more aggressive behavior (Peltzman, 1975).

The use of impedances, by reducing volumes and or speeds, can improve safety. The City of Portland, Oregon found that volumes, and correspondingly, potential crashes, decreased on streets where impedance, in the form of traffic calming, increased (Portland's city-wide..., 2000). Cottrell (1997) observed a similar phenomenon with the use of stop signs. Both Portland and Cottrell noted negative consequences of the increase in impedance, namely diversion and negligible decreases in speeds. Thus, in the latter case, although the potential for crashes decreased, the potential for a particular crash
being severe did not. In a comprehensive stop sign study in suburban Detroit, Beaubien (1976, 1989) found that not only did stop signs not reduce speeds, they increased collision potential due to high violation rates. Because of the simplicity of Cottrell's analysis relating collision potential only to volumes and not other factors such as disobedience of the traffic control device, his conclusions should be viewed with skepticism and used with extreme caution.

3.2.2 Geometric changes that affect safety

There have been numerous studies on the safety of various geometric configurations, and how changes to geometry can affect safety. A study in Edmonton (Abdelgawad, Garcia, Hadayeghi, and Karunaratne, 2014) revealed that chicanes, raised medians, and narrower lanes reduce speeds, and lower speeds inherently improve safety by not only providing more time and space in which to react to sudden changes to driving conditions, but also reducing crash severity, as vehicles striking an object with a lower velocity will have less momentum and do so with a lower force. Less momentum and a lower force, by their very nature, result in lower severity as manifested by the necessary lower displacement or deformation. According to the laws of physics, in a collision energy must be conserved, thus the energy differential between a moving object and a stationary one is transferred from velocity prior to impact to deformation subsequent to it; thus, a vehicle traveling at a higher speed and a higher kinetic energy will, upon impact with a stationary object, transfer more energy to deformation than the same vehicle traveling at a lower speed with a lower kinetic energy. As deformation is a valid surrogate for severity (as it is intuitive that more deformation means higher severity), then a lower level of
deformation is indicative of lesser severity, and since higher speeds cause more
depression, then a logical conclusion is that higher speeds cause greater severity, and
that the converse is true as well, i.e. lower speeds cause lower severity. In short, lower
speeds result in a lower severity of crashes, and a lower severity of crashes is safer than a
higher severity; lower speeds increase safety. Because the Edmonton study (Abdelgawad
et. al., 2014) shows a direct correlation between speeds and geometric treatments, those
treatments that lower speeds could be installed at locations where signals have been
removed to offset any increases in speed resulting from the removal. Having such
methods available will help silence skeptics who oppose the signal removal due to this
possibility (i.e. increased speeds).

Alluri et. al. (2014) and Bowman and Vecellio (1994) found similar results as the
Edmonton study. Both of these studies, like the Edmonton one, concluded that the use
of raised medians improved safety, with Bowman and Vecellio (1994) finding that raised
medians improved safety for both motorists and pedestrian as indicated by reduced crash
rates for both groups. In addition, Alluri et. al. (2014) also found that the safest way to
accommodate left turns on a facility with raised medians is with a unidirectional U-turn
downstream of the intersection as used in the MUT, RCUT, and ILAC alternate intersection
geometric designs, a validation of the superiority, with respect to safety, of these
concepts.

3.3 Alternative intersections

3.3.1 Roundabouts

One of the most commonly used alternative intersection designs is the
A significant drawback of the roundabout is the amount of land required. (Google satellite) Roundabouts provide both quantitative and qualitative benefits. The most obvious quantitative benefit of a roundabout is the elimination of the cost of operating and potentially replacing a signal. In addition, roundabouts have been found to reduce crashes (Wallwock, 1997; Moscovich, 2003; Retting, 2006; Nambisan & Parimi, 2007; Cruz, 2010) and reduce delays (Moscovich, 2003; Turner, 2004; Cruz, 2010). Wallwock (1997) suggests they should be used in lieu of signals for a qualitative benefit as well – reduced speeds. A recent incident in Michigan involving a vehicle being pursued by law enforcement at speeds of 90 miles per hour being unable to navigate a roundabout (Smith, 2016a) verifies the effectiveness of roundabouts at controlling speeds and the benefits of such. Voigt & McCombs (2010) also use a qualitative benefit, sustainability, in arguing for the use of roundabouts instead of signals. The benefits of roundabouts are such that not only have traditional circular roundabouts been recommended, but also a
non-traditional “peanut-about” (Mery, 2009) and “triangabout” (Chou & Nichols, 2014).

Despite these benefits, roundabouts have costs, both quantitative and qualitative. Roundabouts are typically as expensive, if not more so, than new signals - in 2000, the United States Department of Transportation estimated that the average cost of a roundabout was $250,000 (U.S. Department of Transportation, 2000), which is approximately $350,000 in 2014 dollars (U.S. Inflation Calculator, 2014). Wallwock (1997) and Eustace (2000) found that roundabouts only provide superior operational performance with respect to delay for specific conditions, such as balanced flows and high left turn volumes; thus, inappropriately used roundabouts can increase not only delay (McCord, 2011), but crashes as well (Kanitz, Malick & Oulevski, 2004). Not only can operationally inappropriate roundabouts increase crashes, but ones that are designed inappropriately can too, as noted by Kanitz et. al. (2004) and Angelastro (2010).

There exist several qualitative costs to roundabouts. First, roundabouts make navigation of the intersection more complicated for pedestrians and bicycles (U.S. Department of Transportation, 2000; Kanitz et. al., 2004). Second, roundabouts, by their inherent nature, require the removal of more on-street parking than would necessarily be removed for other non-signalized intersection configurations, thus limiting access to adjacent properties in the community. Finally, many drivers are not comfortable driving through more complex roundabouts (Helmer & Davis, 2009), thus increasing the probability of diversion onto other streets in the network as these uncomfortable drivers find other travel routes, with such diversions adversely impacting other parts of the community. (Portland's city-wide..., 2000)
3.3.2 Median U-Turns (MUTs)

The available research on alternative intersection other than roundabouts, such as MUTs, RCUTs, and ILACs, is not as extensive as that for roundabouts due to the limited use of these designs. Of these three, the MUT is more common with respect to the number of intersections, although not necessarily with respect to geographic and cultural context, as a large percentage of MUTs are found in the Detroit metropolitan area. Because of the heavy use of MUTs in Michigan, they were formerly called Michigan U Turns, and much of the research on the operational and safety benefits of these intersections has originated in Michigan. The seminal study on the MUT using field data was by a group of researchers out of Michigan titled “The operational aspect of the Michigan design for divided highways” (Dorothy, Maleck, and Nolf, 1997). This study showed that the MUT was superior to other designs for left turns on divided arterials. Given that the median U-turn crossovers that are an integral part of the MUT are also an integral part of the RCUT and ILAC, and the similarity of the operations of the three designs, it would be logical to conclude the same for the other two designs, as well. Prior to this study, Kramer (1986, 1987) argued the virtues of the concept, and Hummer and Thompson (1991) and Hummer and Boone (1995) simulated it. Dorothy et.al. (1997) validated these previous works.

Subsequent research has confirmed these findings. Bared and Kaiser (2002) concluded that using U-turn crossovers to accommodate left turns resulted in fewer crashes, higher capacity, and an overall improvement in safety due to the elimination of conflict points. As previous research was limited to the use of MUTs at signalized intersections, where most of the U-turn crossover elements are also signalized and right-
FIGURE 3.2. Unsignalized four-legged MUT. The Median U-turns are spaced at the end of the block prior to the intersection, which provides for the one-way U-turns independent of the intersection.

of-way is controlled, Levinson, Potts, Harwood, Gluck and Torbic (2005) focused on their use at locations where the U-turn crossover elements of the MUT are unsignalized and right-of-way is not controlled, and found that MUTs worked well with unsignalized U-turn crossover elements, also. The use of unsignalized U-turn crossover elements saves on the cost of signal installation, operation, and maintenance, and reduces delay during periods of low volume, as U-turning motorists are not forced to wait for the signal to turn. In locales like Michigan where left turns from a one-way street to a one-way street are permitted on red (thus allowing these U-turn crossover signals to function as a stop sign during periods of lower flow), or North Carolina where U-turn crossover signals flash yellow when not green, the delay caused by having to wait for the signal to change is effectively a moot point; in those locales where that maneuver is not permitted, the delay caused by having to wait for a signal to change can be substantial. Thus, if unsignalized U-turns are safe, then they become a useful strategy for reducing delay and improving performance in those locales. Ibarguen and Kar (2009) also advocated using MUTs as a
safety enhancement based on the reduction of conflict points. Alluri et.al. (2014) also verified the safety benefits and concluded that medians with unidirectional U-turns, such as found at a MUT intersection, are the safest way to accommodate left turns. Since the use of MUTs has expanded outside Michigan, the FHWA has published an informational guide to assist design engineers and policy makers in their proper use. (Reid, Sutherland, Ray, Daleiden, Jenior, and Knudsen, 2014)

3.3.3 Redirected Crossing U-Turns (RCUTs)

Like the MUT, a key component of the RCUT is the downstream U-turn crossover, typical in the center of the street through the median. Operationally, the RCUT uses this geometric element quite differently than the MUT. In an MUT, the downstream U-turn crossover is used to accommodate traffic wanting to turn left at the intersection, typically from the larger street to the smaller one. With an RCUT, these movements are accommodated at the intersection itself through unidirectional, back-to-back turn lanes. (FIGURE 3.3) The movements utilizing the downstream U-turn crossing in an RCUT are the minor street left turning and through movements. The RCUT, then, gives priority to all major street movements over all minor street movements. Whereas the MUT is the preferable design when there is a high proportion of through movements on each minor street approach, the RCUT is preferable when that proportion is low, as to not penalize major street left turns for this low volume movement and to eliminate conflict points and potential crashes.

The concept of the RCUT design was originally promoted by Kramer (1987) who promoted it for its possible safety and operational superiority. As the RCUT does
FIGURE 3.3. Unsignalized four-legged RCUT. Note how all traffic on both minor approaches must turn right, while all turns are allowed for both major street approaches.

incorporate some of the geometric design features of other alternative intersections, specifically the downstream U-turn crossovers that are an integral part of the MUT, some of the research that has been completed with respect to the MUT applies to the RCUT as well. Since RCUTs have not been used as long as MUTs, there have not been as many opportunities to study the effectiveness of the former as compared to the latter; thus, research on MUTs that also apply to RCUTs, namely the performance of the downstream U-turn crossovers, is important in creating an accurate mosaic of how successfully RCUTs actually perform. As previously mentioned, Alluri et.al. (2014) concluded that the use of U-turn crossovers enhance safety. Bared and Kaiser (2002)
found that the use of the geometric element of downstream U-turn crossovers as part of the operation of an intersection reduced accidents, by reducing the number of conflict points, and improved efficiency. Interestingly, despite the longer geometric footprint, Bared and Kaiser found that when used as part of a signalized intersection, the use of downstream U-turn crossovers as part of the operation of the intersection reduced travel times as well.

The limited number of studies of RCUTs in their totality involve comparisons of RCUTs with traditional intersections for the same traffic control conditions, i.e. unsignalized RCUT with unsignalized traditional or signalized versus signalized. Xu (2001) studied the safety benefits of signalized RCUTs, and found that the use of signalized RCUT intersections in lieu of traditional signalized intersections on six-lane arterials reduced the crash rate by over 25 percent. Hummer and Ras (2016) calculated crash reduction factors for signalized RCUTs, and determined that for all signalized intersections converted to an RCUT, the overall number of crashes dropped 15 percent and injury crashes dropped 22 percent, a verification of Xu’s findings. Dissanayake, Lu, Castillo, and Yi (2002) performed a comprehensive safety analysis of the use of unsignalized RCUTs compared to traditional unsignalized intersections and found that the RCUTs reduced the conflict rate with respect to both time and volume, as well as reducing the severity of crashes. Dissanayake et. al. also found that, overall, the use of the RCUT was safer than a traditional intersection when the major street was multi-lane and had high volumes. It should be noted that the arterials of Detroit that cross the streets that follow the property lines perpendicular to the river meet these criteria.
Zhou, Lu, Yang, Dissanayake and Williams (2002) determined that the use of indirect minor street left turns by means of a downstream U-turn crossover, a key operational component of both the RCUT and the ILAC, provided superior performance than traditional direct left turns when the volumes on the minor streets are much less than those on the major street, such as what one would find at the intersection of an arterial with a local residential street. These results were verified by Liu, Lu and Piriccioglu (2006), who studied the operational performance of RCUTs and found that when the RCUT movements for the minor street are unsignalized, delay for that movement is reduced. Maze, Burchett, Welch, and Hawkins (2006) studied high speed rural expressway intersections that had been converted to RCUTs and concluded that the use of an RCUT at these types of intersections improves the safety of these intersections. Hummer, Blue, Cate, and Stephenson (2012) added a twist to the operational analysis by comparing the RCUT with not only a traditional intersection but with other alternative designs and tested the various configurations on their ability to maintain the performance on an arterial while accommodating traffic from a site of a proposed mixed-use development, and found that of the designs tested, the RCUT performed the best overall. Since the use of RCUTs has increased over the past decade, the FHWA has published an informational guide to assist design engineers and policy makers on their proper use. (Hummer, Ray, Daleiden, Jenior, and Knudsen, 2014)

3.3.4 Indirect Left and Crossing (ILACs)

The ILAC is a ubiquitous concept that has been used for decades as both a safety measure and an access control mechanism. (FIGURE 3.4) For example, Neumann includes
FIGURE 3.4. Unsignalized four-legged ILAC. Due to the physical obstruction preventing vehicles from crossing the intersection, all left-turning traffic and traffic wanting to cross from one minor approach to the other must use the downstream U-turns.

elements of the ILAC in the 1985 TRB publication, Intersection Channelization Design Guide. Interestingly, despite its common use by practitioners, little research has been done to verify its effectiveness. Because the ILAC shares some of the same operational and geometric elements of the RCUT, namely the replacement of direct minor street through and left-turn movements with indirect movements via a downstream U-turn crossover, then the research pertaining to that particular element that has already been completed on RCUTs applies to ILACs as well. Based on this research, one can safely conclude that this operational and design element reduces delay and crash potential for unsignalized intersections. The only direct research on the ILAC configuration performance is a study by Edara, Sun and Breslow (2013), who analyzed the effects of converting a traditional intersection on a rural high-speed expressway to an ILAC configuration (called a “J-turn” by the researchers). The researcher found that after the
conversion to an ILAC, the intersection experienced fewer crashes and less delay. These results are consistent with the results found for the RCUT, which is operationally and geometrically similar to the ILAC.

Based on an exhaustive literature review, the following conclusions regarding alternative intersections can be drawn. First, if used correctly, they improve operational performance. Of the three designs, the MUT provides the most operational flexibility, as no movements are physically restricted but are redirected to downstream median crossovers as a matter of operational policy through the use of signage and enforcement. (This flexibility allows for periodic changes in which movements are redirected based on traffic operations; if done properly, these changes can help to reduce or eliminate negative impacts, such as queuing of redirected movements into the through traffic stream.) Both the RCUT and the ILAC physically restrict and redirect movements, with the RCUT only redirecting minor street movements. Because the ILAC is the most physically restrictive and redirects movements on both the minor and major streets, it should not be used for an intersection where the redirected movements are large enough (with respect to traffic volume rates) to cause adverse affects and failure of the intersection. Second, all the alternative designs improve safety by eliminating conflict points. Third, all can be successfully used for unsignalized intersection applications. What hasn't been determined, and the premise of this research, is to determine how effectively they can be used to either replace an existing signalized intersection with an unsignalized one, or to eliminate the need for a future signalized intersection.
3.4 Environmental and sociological considerations

One of the earliest works on the environmental and sociological influences of the transportation networks is Jane Jacobs' *The Death and Life of Great American Cities*. (1961) Jacobs details how small changes to the transportation network, such as block size, can have significant impacts on a neighborhood's quality of life, especially with regards to pedestrians. Jacobs argues that from the sociological perspective, a good design encourages the integration of pedestrians and vehicles. Such an integration is possible with raised medians and other such pedestrian refuges, where pedestrian traffic is more fluid, as opposed to wide avenues with signals, where pedestrian traffic is focused at specified locations. Jacobs details how focusing pedestrian traffic to certain locations has a negative impact on the long term sociological health of a neighborhood. Langley (1976) also delved into the environmental and sociological effects of transportation with an in-depth analysis of the effects, over time, of a transportation project on property values, specifically a limited access highway. Langley concluded that reducing travel impedances like traffic signals increases the value of adjacent properties by improving accessibility and reducing noise and air pollution endemic to repetitive starting and stopping of motor vehicles. Langley also stressed the importance of considering the impacts of a transportation project on the surrounding neighborhood when evaluating the overall benefits and costs of a project, and that designers should strive to ensure that any project does not adversely affect the neighborhood. In other words, Langley argues that even if a project has an overall positive impact, it should not necessarily be recommended if it adversely impacts nearby properties. Thus, if the removal of a traffic
signal yields an overall improvement in operational performance, but causes a degradation of access or operational performance for those wanting to access the surrounding neighborhood, such a removal should not automatically be undertaken. Gurin (1976) synthesizes both Jacobs' and Langley's theses and argues that good transportation policy should include multiple points-of-view and be cognizant of how professional values affect the planning process, as what may technically the superior option may not necessarily be the best option from an environmental or sociological view. Burkhardt (1984) makes a similar point with his study of socioeconomic reactions to highway development. Flyvbjerg (1984) expresses alarm at the trend among engineers away from an open approach of decision making towards a closed decision-making process focused on efficiency and neo-classical economic theory at the expense of environmental and sociological considerations, and argues that a continuation of the trend could breed distrust and cynicism towards the results of the process. Loukissas and Mace (1984) concur with Flyvbjerg with their case study of three transportation projects in Pennsylvania. Despite this trend towards technical exclusivity, sociological and environmental considerations are still part of the calculus, due to the absorption of different professions into traffic and transportation engineering, and with them, different understandings of the basic concepts (Mladenović, Mangaroska, & Abbas, 2014), and a complete lack of understanding of basic concepts among practitioners, with this lack of understanding, and the potential for including concepts beyond the technical such as sociological or environmental, being incorporated in designs and analyses (Hurwitz, Brown, Islam, Daratha, & Kyte, 2014)
There has been ample thought pertaining to the effect of transportation infrastructure on the environment, particularly visual aesthetics, air pollution, and storm water runoff and management; in older cities like Detroit, where all three of these elements are of a concern, designs and concepts that can improve these elements should be given thorough consideration. Duncan (2008) found that a disorderly road encouraged bad driver behaviors, as drivers took a “if they don’t care, why should I?” attitude towards such roads. By breaking up the paved area with islands and green spaces, and eliminating many traffic control devices that often may be in a state of disrepair, the alternative intersection designs mitigate this disorderliness, and help restore motorists’ respect for the roadway environment. Grzwskowski (2007) reports that local governments view green space as vital to the quality of urban life; thus, it is important to strive to increase it. Khanna (1993) states that it is environmentally important to minimize impervious areas and maximize green areas to reduce storm water runoff and related flooding issues. Waller (2014) performed a meta-analysis of research on the impacts of road diets, including the construction of medians like those used in the alternative intersection designs, and found that road diets improve aesthetics, air pollution (due to improved traffic operation), and reduce runoff and flooding.

### 3.5 Comparison and evaluation strategies

Since the purpose of this research is to create a methodology for analyzing the feasibility of converting traditional signalized intersections into non-traditional unsignalized ones, it is imperative to explore the various strategies for making such a comparison. When evaluating the effectiveness of intersection operation and comparing
different modes of operation, namely signalized versus unsignalized, it is important to consider not only the engineering and operational aspects, but also the social and environmental aspects as well. Many of these social and “quality-of-life” aspects are qualitative and cannot be easily quantified for comparisons either between one location and another or between various designs and operations at the same location. Thus, it is imperative to use quantifiable surrogates for these qualitative aspects. For example, green space enhances aesthetics, but aesthetics are not easily quantifiable, as they are a matter of personal preference and taste. How much green space to use is often the subject of vigorous debates among design professionals. While green space does improve aesthetics, and thus the “quality-of-life”, there is a quantifiable benefit to green space as well – reducing runoff through the increase in pervious area. Reduction in runoff, a quantifiable value, can be used as a way to measure and compare the value of green space, a qualitative one.

NCHRP 456 (Forkenbrock and Weisbrod, 2001) addresses how to assess the impacts of transportation infrastructure projects, specifically the social and economic effects. NCHRP 456 identifies eleven different factors that should be considered in this assessment: changes in travel time; safety; changes in vehicle operation cost; transportation choice; accessibility; community cohesion; economic development; traffic noise; visual quality; property values; and distributive effects. While these are worthy factors to consider, NCHRP 456 does not detail how to calculate them, only that they should be used. Some of the factors, such as changes in travel time, safety, and transportation choice, have been studied extensively and can be calculated. The more
subjective factors, such as community cohesion, are not as easy to calculate and quantify. This lack of quantification can prove to be a challenge to integrating these factors into common used methods for comparing competing options such as benefit-cost analysis.

3.5.1 Monetization of Benefits and Costs

Benefit-cost analysis entails the monetization of the value of the benefits and costs of a particular alternative, expressed in a particular monetary unit for a particular period of time. There are two different methods of performing a benefit-cost analysis: a quotient of benefits and costs, and a difference of benefits and costs. (White, Case, Pratt & Agee, 1998; Sepulveda, Souder & Gottfried, 1984) With the quotient method, the monetized value of the benefits is divided by the monetized value of the costs (B/C); in other words, this method is a calculation of the return on investment (ROI). Using the ROI method, the alternative with the highest ROI is the best alternative, and any improvement or change that yields a ROI less than the ROI of the default (i.e., the status quo or “do-nothing”, alternative), then the improvement should not be constructed. However, for the purposes of this analysis, the do-nothing alternative does not exist, as even maintaining an intersection as a traditional signalized one will entail improvements such as replacement of the hardware and infrastructure, including cabinets, controllers, heads, wiring and posts.

With the difference method, the total value of the costs of the improvements are deducted from the total value of the benefits of the improvement, with the highest value representing the superior alternative. With the difference method, also known as the net benefit value (NBV) (Sepulveda et. al., 1984), a B-C value less than zero indicates that the
costs exceed the benefit, and any option with a value less than zero should not be considered as a viable option. White et. al. (1998) found that the NBV method to be superior to the ROI method as it is less sensitive to small changes in benefits and costs, such as an increase in the price of a toll or a fee, with such an increase reducing the benefits and increasing the costs. When calculating a quotient, even a small difference can produce a different result, as a quotient represents a proportion, and if the total numbers are small, a small change is magnified in the calculation of the ratio. As Sepulveda et. al. (1984) observe, using the NBV eliminates the quandary of how to assign factors that could argue be either a benefit or a cost, such as a toll. With these types of factors, whether they are assigned as benefits or costs determines whether they are in the numerator or denominator, which in turn affects the value of this quotient. If one of these factors is assigned incorrectly, a viable project may be deemed unviable, and vice versa. As the NBV does not have a denominator, it does not have this issue. Because of this, White et. al. (1998) and Sepulveda et.al. (1984) encourage the use of the NBV as the superior method to perform a benefit-cost analysis.

Despite its inherent sensitivity, the ROI is not without its usefulness. As already discussed, the NBV, because of its more binary, absolute nature, is the preferred method for determining if a project should be done. For jurisdictions with large capital improvement budgets and few, if any, fiscal and budgetary constraints, all projects with a positive NBV have a realistic probability of being completed and thus should be programmed. However, for jurisdictions with limited resources and significant fiscal and budgetary restraints, it is not possible to program every project with a positive NBV. In
these cases, the ROI becomes a useful tool to help prioritize project programming, with those projects with a higher ROI given highest priority. Where such budgetary constraints exist, it behooves the analyst to periodically reanalyze and reprioritize those projects with a positive NBV that could not be funded to ensure their continued viability and to reflect changes to the transportation network and local environment, including land use and material cost changes.

While the monetized benefit-cost analysis is ubiquitous, other methods of quantifying qualitative factors, and comparing these factors for several alternatives, should be explored. One of the most common, the “design charrette”, is often heavily influenced by sociological and political considerations. Some of these methods, such as data envelopment analysis (DEA) and the analytic hierarchy process (AHP), are often used in other branches of engineering; other methods, such as a rank scoring matrix, a weighted ranking evaluation, objective ranking, and normalized numbers, are from other realms. It is important to discuss these methods, as well as benefit-cost monetization, in greater detail, to determine which strategy is the best to use for the comparison of different intersection operational concepts.

3.5.2 Design Charrette (Analysis by committee)

A design charrette is a select committee representing a variety of interests and points-of-view. The committee is empowered to evaluate various alternatives and choose the best alternative. While the ultimate outcome, a vote on the preferred alternative, is objective, the rest of the analysis process may be highly subjective and based on the personal biases of the individual committee members. There are also other significant
drawbacks to the use of a committee that precludes its use as an effective analysis method. First is the need for conformity, what is known as the “Ashe Effect”. (Myers, 2014) The noted psychologist Ashe studied this phenomenon and concluded that the need for conformity is so strong that individual members of a group will intentionally choose a wrong answer to fit in. Ashe’s experiment has been repeated numerous times with the same result. Thus, based on this effect, one would have to view results from a design committee with a healthy dose of skepticism – is the preferred alternative truly the superior one, or was it chosen because of the need by committee members for conformity?

This need for conformity within a committee has several negative consequences. First, due to the need for conformity, analysis by committee results in a general blandness and lack of innovation due to the pressure to conform. Often, this conformity results in choosing the traditional, known, alternative over the innovative, potentially controversial one. In addition, due to the need for conformity, committees tend to be dominated by a “strongman”, a particularly strong personality whose outspoken views, no matter how outlandish, tend to be adopted by the less outspoken in order to conform. Combining the need for conformity with the domination of a particular personality results in a tyrannical majority that disregards and diminishes the perspectives and viewpoints of those not in agreement, using the need for conformity as a means to obtain concurrence, even when the minority viewpoint may result in a superior outcome. Because of the weaknesses, a design charrette or committee is not a viable option for evaluating geometric and operational changes of an intersection.
3.5.3 Data Envelopment Analysis

Data envelopment analysis, DEA, is a tool used by some in engineering, most notably those concerned with logistics, processes, and efficiency, to quantify qualitative factors that contribute to the efficiency of a process. (Cooper, Seiford & Zhu, 2004) The DEA method uses a series of scatterplots of various parameters to establish an efficiency boundary, or envelope, with maximum efficiency occurring along the envelope. As the process allows for the comparison of non-relational parameters, qualitative as well as quantitative parameters are allowed to be compared and analyzed. Points resulting from these comparisons are assigned an efficiency based on the ratio of the distance of the point from the origin divided by the distance from the envelope to the origin along a line passing through the point. For example, the distance along a line passing through the data point from the origin to the envelope is 100 units, and the distance along that line from the origin to the data point is 65 units, yielding an efficiency of 65 percent. Using this method, various data points can be compared. Because this method becomes complicated and unwieldy when there are more than a handful of parameters to be considered, its applicability to transportation is limited. Due to the number of parameters that need to be considered when comparing intersection operations and geometry, this particular method is ill-suited for this particular application and will not be considered further.

3.5.4 Analytic Hierarchy Process

The analytic hierarchy process (AHP) is a method to objective and quantify subjective and qualitative criteria through the use of relative binary comparisons and
matrix algebra. (Saaty, 1992) For a particular decision, inputs and considerations are broken down into criteria and alternatives, with all alternatives tested for each criterion. Using pairwise comparisons, the relative importance of criteria with each other (and alternatives with each other for each criterion) is noted. These relative values are placed in a matrix, and using mathematical tools, a final number for each criterion and alternative is generated. Although the AHP does create a quantitative value for qualitative factors, it is still subject to bias as the relative importances are subjective. The results of an AHP analysis may not be reproducible, as two different human evaluators may assign different relative importances, thus culminating in different results.

3.5.5 Rank Scoring Matrix

A rank scoring matrix, sometimes called supernumbers, is a type of alternative preference matrix that quantifies preference for a particular alternative for a variety of criteria, and then allows a numeric comparison between these alternatives. With a rank scoring matrix, the available alternatives are ranked from most preferable to least preferable, or vice versa, for each of several criteria, with the rankings for all of the criteria aggregated for each alternative. The aggregated rankings for each alternative are then compared, with the best aggregated value scored as the most preferred alternative, and the worst aggregated value score the least preferred alternative. The AP Top 25 college football poll methodology (AP, 2012) is a classic example of a rank scoring matrix. In this methodology, college football programs are ranked from 1-25 by individual sports journalists on an inverted point scale (i.e. a rank of 1 being assigned a point value of 25, and a rank of 25 being assigned a point value of 1). The ranking points are then aggregated
for each team, with final rankings determined by comparing these total individual aggregations. The team with the most aggregated points is given the highest rank, and the team with the lowest points is given the lowest rank. When applying the AP method to a comparison of various alternatives using several unique criteria, the football teams are analogous to the alternatives and the sports journalists are analogous to the criteria.

The aggregated ranking is sometimes called a supernumber. While supernumbers are an easy to use and understand method of comparing various alternatives given a set of criteria, supernumbers should be used with caution for the following reasons. First, all criterion are assigned equal weights. While this is practical in the case of the AP football poll where all sports journalists selected to participate are considered to be equal, it may not be in cases where the criterion clearly are not. For example, when analyzing proposed changes to an intersection, the cost of the change may not necessary be viewed as being equally important as delay, nor may either be of the same importance as safety. Different jurisdictions may, and often do, place different importance on each of these three. Jurisdiction A may view even an incremental safety as paramount no matter what the cost, while Jurisdiction B may view incremental safety improvements as conditional on the cost. Jurisdiction C may view reducing delay as the most important consideration of all. Assigning all criteria an equal weight discounts these differing and unique local priorities.

A second significant limitation with supernumbers are that they treat the differential in ranks for the alternatives for a particular criterion equally, and also applies this equality between ranks for all criteria. For example, suppose the three criterion for analyzing various intersection modifications are safety, cost, and delay. For the safety
criterion, the difference between the highest ranking alternative and the second highest ranking is one third of the difference between the second highest and third highest alternatives; however, using supernumbers, the magnitude of these differences is not taken into account, as the difference between the highest and second and the second and third are considered to be equal. Thus, whatever factors and variables that contributed to the difference in magnitude of the differences between the ranks, are, for all practical purposes, nullified. Supernumbers also assume that the difference between the rankings is the same for all criteria, which may not necessarily be the case. The cost criterion may have a sizable difference between the first and second ranked alternatives and a small difference between the second and third, unlike with the safety criterion. By assuming all differences between ranks are equal for all criteria, criterion specific variations in the relative rankings, which may or may not be significant, are ignored.

3.5.6 Weighted Ranking Evaluation

The weighted ranking evaluation (WRE) method is a variation of supernumbers where the relative importance of each criterion is taken into account in the analysis of various alternatives. This modification addresses one of the limitations of supernumbers, namely that not all criterion may be viewed as equally important. With the modification, the WRE method allows for local customization of the general supernumbers procedure, recognizing that different locales have different needs and priorities. A ubiquitous example of the WRE method is the U.S. News ranking of colleges (Morse, Flanigan & Tolis, 2014), where each of the criterion are assigned a particular weighting. Thus, the highest ranking for one particular criterion may not be of equal value to the highest ranking of a
different criterion. While the WRE does address the supernumber limitation of assuming that all criterion are of equal importance, it, like supernumbers, does not address the limitation that the difference between ranks for all alternatives within a particular criterion are the same, thus ignoring potentially criterion-specific variations in rank importance.

3.5.7 Objective Ranking

Objective ranking is a ranking methodology where ranks are assigned based on objective criteria. Ranking assignments are based on objective measures, ideally binary measures which are easily discernable and not subject to subjective interpretation. In their methodology for prioritizing signal structure assembly replacement, Schrader and Bjorkman (2006) detail an objective ranking system where characteristics (e.g. rust, holes, missing bolts) are evaluated using a binary stratagem (i.e. yes or no) and assigned a specific point value, with points being awarded for affirmative binary responses. The points are then tabulated and the prioritization of replacement assigned by total point value, with higher values being given higher prioritization. Schrader and Bjorkman explain that by assigning a binary stratagem to each characteristic rather than a range of values (e.g. 1-10), subjectivity is eliminated. With a range of value for each characteristic, one inspector may rate a particular characteristic differently than another, resulting in a different outcome of replacement prioritization for a particular signal structure based on the opinion of the inspector. By converting all characteristics into binary stratagems, Schrader and Bjorkman eliminated this problem, as each characteristic either existed or did not, and the subjective assessment of magnitude by the inspector was eliminated.
3.5.8 Normalized numbers

A normalized number evaluation scheme is similar to the objective ranking one except that characteristics and criteria are not ranked in importance relative to one another, but are calculated mathematically and set equal to each other through normalization. Schrader and Hoffpauer (2001) detail such a normalization scheme for criteria used to evaluate potential highway-railway grade separation locations. For all locations studied, the mean and ranges of each individual criterion were equalized, with the result that all calculated values had an equal weight (as the range of values was the same). The result, then, was that for each location studied, the calculated numbers for each criterion were aggregated together, and the aggregate for each of the locations could be compared to the aggregate of all other locations to determine prioritization. Schrader (2002a) subsequently used this evaluation scheme for prioritizing highway capital improvement projects in a multi-jurisdictional setting.

3.6 Current state of the practice in evaluation of unsignalized geometric alternatives

As detailed, there has been research in the use of alternative intersection geometries, such as the MUT, roundabout, RCUT, et. al., but only for limited and very specific applications. In the case of roundabouts, they have been installed in specific locations meeting specific land use, design, and operational criteria in order to maximize the benefit and minimize the cost. For example, on low volume urban intersections with no through truck traffic, mini-roundabouts are used within the existing rights-of-way; note how these can only be used for a very limited type of intersection. On larger and busier roadways, roundabouts are limited to those that exhibit particular traffic patterns (i.e.
approximately equal flows or large volumes of left turns), a particular adjacent land use (commercial, industrial, institutional, or open space), and reasonable right-of-way property acquisition costs. Two factors are used to evaluate the potential application of roundabouts – traffic crash reduction potential and improvement to traffic operations, specifically the reduction of delay. The use of the MUT has been limited to signalized intersections on higher speed, higher volume arterials with large medians, locations typical found in suburban areas. Like the roundabout, the effectiveness of the RCUT is measured in reduction in delay and crashes. Studies on the effectiveness of ILACs have been limited to rural applications, specifically the at grade intersection of high speed expressways with lower volume highways and roads; like roundabouts and RCUTs, the effectiveness of these installations is determined by measuring the impact of the installation on delay and crashes.

There are several significant drawbacks to such a limited measure of effectiveness (MOE). First, crashes are a reflection of entropy; thus, there is not a truly effective way to determine if a geometric change is responsible for a decrease in crashes. Non-human factors such as the weather, migration patterns of wildlife, the position of the sun, and vegetation all cause crashes, and these important natural factors are extremely challenging to predict. Human factors such as inattentiveness, driving under the influence of alcohol or drugs (both legal and illegal), carelessness, confusion, and distractions (e.g. crying children, dropping something on the floorboard, changing radio stations, conversation with passengers) all cause crashes and are not realistically predictable due to their inherent entropic state. There are so many factors other than engineering design
that contribute to crashes that the best that can be concluded when assessing the
effectiveness of alternate intersection installations is that they help reduce the possibility of crashes, which is a worthy objective, as engineers should endeavor to reduce the possibility of crashes.

That being noted, alternative intersection designs can be more expensive than what local officials are willing to spend. Many policy makers are skeptical about the effectiveness of alternative intersection installations and are not receptive to spending the necessary public moneys to install them. Unless the location is one where a high profile injury or fatal crash has occurred and the policy makers feel public pressure to install an alternative intersection, such an installation typically does not happen, with the result being the continued operation of an inherently unsafe geometry. There are many unsafe intersections where an alternative geometry would enhance safety, but the local policy makers lack the necessary political resolve to make it happen, instead playing the odds that a fatal accident will never occur. Many of these intersections are at locations that do not necessarily have severe delay issues or a high number of crashes; that does not make them any less inherently unsafe. Because the current state of the research uses delay and crash reductions as MOEs, these intersections continue to pose a threat to the health and well-being of motorists, pedestrians, and cyclists. It is important, then, to improve on these MOEs to allow the identification of these dangerous intersections and help policy makers understand the benefits of improving these locations and the true societal costs.
3.7 Summary of literature review

While alternate intersections have been utilized for at least the past forty years, it has been only within the past twenty years that the concept of “alternative intersections” in planning and design has become an independent entity worthy of its one lexicon and study. Prior to that, alternative intersections were conceived, used, and studied incidental to other topics and concepts (e.g. the use of roundabouts as part of an interchange). While earlier literature does indeed discuss the alternative intersection concepts, they are often conditional to other concepts being implemented as well; thus, this early literature is severely limited in its applicability to independent, stand-alone intersection analysis. Nonetheless, there have been studies on the individual, independent application of alternative geometric designs.

There are four types of alternative geometrics that have been studied in various degrees – roundabouts, MUTs, RCUTs, and ILACs. Of these four, roundabouts have been the most study, ILACs the least. The ubiquitousness of and long-standing use of roundabouts outside of the United States has provided a large and varied pool of applications that have been studied and analyzed, along with utilitarian analysis and design templates that are, for the most part, universally applicable. While MUTs have been used in parts of Michigan for decades (allowing for observation and analysis of their effectiveness over time and changes in land use, etc.), they have rarely been used elsewhere, and only at signalized intersections. Thus, the available literature on MUTs is limited in scope, both geographically and in application. Unlike the MUT, the RCUT has been used in a variety of places; thus, a more diverse cross-section of drivers has been
exposed to them. Unlike the MUTs, RCUTs are a relatively new application, with most installations being constructed within the past fifteen years. (This limits the ability to study their effectiveness over time.) Like the MUT, the operational application of the RCUT has been very limited in scope, with most RCUTs installed on higher speed, multilane divided suburban arterials with limited access. The ILAC (sometimes called a “J-turn”), has the least amount of research and literature available. They are not only the newest concept to be applied (within the past decade), but also very limited in where they have been built (unsignalized intersections on rural, high speed limited access expressways). Of the four, the only concept which has been used to convert a signalized intersection into an unsignalized one is the roundabout; with the other three, the status quo was maintained with respect to signalization status.

There is extensive literature on various strategies to evaluate alternatives, ranging from the subjective to the objective. The design charrette (e.g. analysis by committee), is the most subjective of those reviewed; due to its subjectiveness, it is not utilitarian nor easily applied universally, as the decisions ultimately are a result of the unique biases and preferences of the participants of the charrette. The most objective evaluation methods reviewed were normalized numbers and monetization (specifically, net benefit value, where the monetary benefits of an alternative are compared to the costs of implementing that alternative). With both of these methods, none of the preconceived preferences or biases of those performing the evaluation have an impact in the alternative selected; it is universal, ubiquitous, and easily duplicable. (Theoretically, with this method, an infinite number of evaluators will come up with the same answer). The difference between these
two objective methods is familiarity and use, as monetization is the most common method used for evaluating engineering projects, and numerous texts have been written about it. (Thus, it is more easily understood by the analyst.) Other methods reviewed such as data envelopment analysis, analytic hierarchy process, rank scoring matrix, weighted ranking evaluation, and objective ranking contain varying degrees of subjectivity, and therefore may not as easily be applied elsewhere.
CHAPTER 4 “METHODOLOGY”

4.1 Signalized versus unsignalized theory and practice

There are two primary purposes why public agencies install traffic signals – to facilitate travel along a travel corridor, or to facilitate access to and from a travel corridor. When installing signals, jurisdictions strive to satisfy either of those two criteria, facilitating travel or facilitating access. However, when removing signals, jurisdictions strive to satisfy both. In other words, it is common to install signals to improve traffic flow at the expense of access, or vice versa, but when contemplating between maintaining a signalized intersection, even one that is no longer needed, or reverting it to unsignalized operation, such as decision is commonly only answered in the affirmative for reversion if the unsignalized operation will not have a detrimental impact on both traffic operation along the major corridor and access from neighboring properties to and from the major corridor, with the latter being the more challenging parameter to satisfy. The result, then, is a much higher burden to justify unsignalized operation and the continued operation of legacy signals resulting from the inability of local transportation officials to successfully overcome this burden. As has been previously detailed, the ultimate outcome is a plethora of unneeded signals that are a financial drain on jurisdictional budgets. A mathematical model to help practitioners determine whether an intersection can operate as unsignalized without degrading the operation of a travel corridor and determine the appropriate design to use to ensure access from the intersecting streets is not degraded would be a valuable to rectify this problem. The objective of this research is to create such a model.
4.2 The unsignalized geometry model

The solution to creating a mathematical methodology to help practitioners determine if an intersection can operate in an unsignalized state instead of a signalized one requires two distinct outputs. The first of these is a mathematical indicator of whether the change from signalized to unsignalized operation is an overall benefit or a detriment. The second output is a determination of the social impact of such a conversion. As social impacts are typically subjective in nature, the social impacts of a project are in the eye of the beholder; in other words, they are based on the opinion of an individual, and different individuals may hold different opinions as to the social impact. Therefore, to create a replicable methodology, a quantitative mathematical surrogate is necessary.

Because of the importance of both present and future cost and budgetary constraints to decision makers, especially at the local level, it is imperative to create a methodology for a holistic evaluation of proposed infrastructure changes at an intersection, particularly the removal of a traffic signal and its replacement with an alternative unsignalized intersection. The most efficient methodology is a benefit-cost comparison of the various concepts. As enumerated previously, the superior method of benefit-cost analysis is with the net benefit value (NBV) method, namely the value of benefits minus the value of costs, with the alternative with the highest difference being the superior one, and an alternative with a difference value less than zero eliminated from consideration, as it is economically and monetarily worse than the status quo.

When evaluating the impact of removing signals, the factors enumerated in
NCHRP 456 are applicable in part or in whole. Travel time and safety are factors that can be considered in and of themselves using existing evaluation tools found in the Highway Capacity Manual and elsewhere. Traffic noise can be addressed when evaluating air pollution, as vehicles that are loud may also pollute more, as the loudness may be caused by maintenance problems such as malfunctioning spark plugs, clogged filters, or an incorrect oxygen-to-fuel mixture, all problems that cause an engine to operate at below peak efficiency (“How to reduce car engine noise”, 2016). Providing more pervious area improves visual quality and improves property values (Eschbacher, 2006; Durrell, 2007; Grzwskowiak, 2007; Greenberg, 2008) due to more green space and reduced runoff. In The Death and Life of Great American Cities (1961), Jane Jacobs discusses in detail how the amount of pedestrian activity is an indicator of community cohesion, with communities with greater cohesion having greater activity. It is intuitive that the easier it is for pedestrians to move not only along but across a street, the more pedestrian activity there will be. Thus, the ability of pedestrians to cross the street efficiently and safely is a good surrogate measure of community cohesion.

In addition to being a good surrogate for community cohesion, pedestrian activity is also a good surrogate for accessibility and transportation choice. All trips ultimately begin and end in the pedestrian mode, as one must walk to a vehicle when leaving an origin and vice versa when arriving at a destination. The more accessible a property is, the greater the probability that it will serve as the ultimate origin or destination, due to the ability to switch modes from vehicle to pedestrian. Jane Jacobs (1961) discusses the importance of pedestrian accessibility in depth and shows how the lack of accessibility
and lack of pedestrians has a deleterious impact on a neighborhood and community. Not only is partaking in the pedestrian mode necessary to access land, it is also necessary to switch between conveyance devices, as one must leave one device and walk to enter and embark on another. Furthermore, the inability of pedestrians to access different conveyance devices renders even the best device meaningless, as a form of conveyance cannot be used, even it is desired to do so, if it cannot be accessed.

For this research, the following monetized evaluators will be used: effect on traffic operations, specifically, travel times; effect on vehicle speeds, specifically delay; effect on operating costs; effect on water runoff and overall environmental quality; effect on air pollution and noise; construction costs. It should be noted that the first five of these will be benefits in the monetization model, while the sixth is considered a cost; the value of the sixth will be deducted from the sum of the first five in the NBV model. These six evaluators address the factors highlighted in NCHRP 456, either directly or indirectly, and are reasonable measures of effectiveness for the quantifiable concerns with changes to intersection operation (e.g. operations) as well as for the qualitative issues (e.g. aesthetics, noise, “quality-of-life”). Much of the research published as part of the NCHRP series refer to monetized benefits and costs of transportation, and monetization is a long-established method of comparing alternatives for decision-making in transportation, objective and neutral, mathematically focused, and easy to understand by not only transportation professionals but also policy and decision makers outside of the transportation profession. For these reasons, this monetization model can be valuable for evaluating the feasibility of conversion of a traditional signalized intersection to an
alternative unsignalized one.

4.2.1 Travel time benefit (B$TT)

The first evaluator in the model is the travel time benefit (B$TT). Because alternative intersections, by their very design, require some movements to travel greater distances to complete the movement, the cost of operating an alternative unsignalized intersection with respect to travel time (due to inconvenience to the motorist as manifested by increased travel times) is not the same as the cost with respect to travel time of operating the same intersection as a traditional signalized one. It should be noted that this cost does not include wait time to begin the movement (i.e. delay), but only the time it takes to complete the movement once begun. The most practical way to highlight the travel time cost differential is through monetization, specifically determining the cost required to complete the movement once begun for both the signalized and unsignalized conditions and comparing the two. If the aggregated travel time cost for the intersection after conversion is less than that before the conversion, then the conversion provides cost savings and a positive net B$TT. Conversely, if the aggregated travel time after conversion is greater than before the conversion then the conversion increased the costs and created a negative net B$TT.

Annualized B$TT can be calculated using the following equation:

\[
B$TT(\$/py) = 1.19786 \frac{W}{s} (d_s V_s - d_u V_u) \quad \text{(EQ. 4.1)}
\]

Where

- \(B$TT(\$/py)\) = Annualized travel time benefit, $ per year
- \(W\) = hourly wage, $;
- \(s\) = average speed, mph
- \(d_s\) = distance traveled, ft (signalized);
- \(d_u\) = distance traveled, ft (unsignalized)
- \(V_s\) = vehicles per hour (signalized);
- \(V_u\) = vehicles per hour (unsignalized)
For this analysis, the following assumptions were used in the calculation of this benefit:

(1) Daily traffic volumes vary with the day of the week and the month of the year; therefore, the exact same hourly volume taken from two different days represents a different proportion of daily traffic. A simple rule-of-thumb is that peak hour traffic represents 10 percent of daily traffic; this rule-of-thumb is overly simplistic and ignores these temporal fluctuations in traffic. The result could be a gross underestimation of the actual daily traffic if the hourly sample were taken for a period that represents less than ten percent of the daily traffic, which could lead to false positives and a wrong decision to convert a signalized intersection into an alternative unsignalized one.

To reduce the possibility of a false positive, the hourly volumes are assumed to represent the eighth maximum daily hour (8MDH), resulting in a conservative overestimation of daily volumes. The use of the 8MDH yields the 1.19786 constant in EQ 4.1. (The eighth maximum daily hour is the eighth highest hourly volume among the twenty-four hours in a day. Since several justifications for signal installation pertain to volumes for eight hours exceeding certain thresholds, then if the 8MDH volumes exceed these thresholds, signals are justified.) The rationale for this is two-fold. First, if this conservative assumption yields a positive net benefit in converting, then it can be reasonably be
assumed that the chance of a false positive, for all practical purposes, is zero, providing confidence to the decision maker that conversion is the correct decision. Second, 8MDH can easily be converted to daily traffic (ADT) via the thirtieth maximum annual hour (30MAH) and annualized (AVT) via the following equations:

\[ 30MAH = \frac{8MDH}{0.55} \]

\[ \text{(EQ 4.2)} \]

(Illinois Department of Transportation, 2002)

\[ ADT = \frac{30MAH}{0.105} \]

\[ \text{(EQ 4.3)} \]

(Illinois Department of Transportation, 2002)

\[ AVT = 365.25ADT \]

\[ \text{(EQ 4.4)} \]

The first of these equations, EQ 4.2, establishes the relationship between 8MDH and 30MAH, i.e. 8MDH is 55 percent of 30MAH; the second, EQ 4.3, the relationship between 30 MAH and average daily traffic (ADT); the third, EQ 4.4, the relationship between ADT and annual traffic (AVT). The 30MAH is the thirtieth highest hour of traffic expected within a year and represents 10.5 percent of daily traffic; it is frequently used to estimate peak hour traffic volumes. As with the constant velocity and value of time assumptions, this assumption can also be modified without degrading the overall effectiveness of the model as long as any changes are applied “across-the-board” to all scenarios evaluated.
(2) Vehicle velocity is a constant 44 feet per second (30 miles per hour).

The rationale for using this speed is discussed in more detail in subsequent sections of this chapter. The use of a constant speed over the entire distance is for the sake of simplicity, as some of the vehicles traveling this path will be starting from a dead stop, necessitating acceleration. Since neither the number of vehicles needing to accelerate nor the types of vehicles needing to accelerate can be determined for every possible moment in time (due to entropy and the street network being an open system, the exact characteristics of the vehicle stream are dynamic), an assumption is necessary. Any changes to this assumption need to be applied across-the-board to all analysis scenarios to ensure consistency.

(3) Time is worth $8 per hour. It is impossible to know what time is worth to every vehicle at an intersection, as the types of employment (or non-employment) vary from person to person and there may be multiple persons with different values of their time in a single vehicle. Minimum wages vary from occupation to occupation and from state to state; $8 per hour reflects a median value of the state minimum wages in 2017, with twenty-five states having a minimum wage below this value and twenty-five states having a minimum wage above it. (Minimum wage.org, 2017) This value may be adjusted; however, any adjustments need to be applied across-the-board to all scenarios.
4.2.2 Delay benefit ($B_{D}$)

The second evaluator in the model is the delay benefit ($B_{D}$). This is the companion of the first benefit, as it encompasses the wait time to embark on the movement encompassed in the first one. The first and second benefits relate to traffic operations, and the impact of the conversion of a traditional signalized intersection to an alternative unsignalized one on the performance of the intersection. The first benefit encompassed the difference in time to complete a movement one it has been initiated; the second, the wait time to initiate the movement. The delays are calculated using the HCM and its related software using the existing conditions and the proposed conditions, both geometrically and operationally. If delay is reduced, the conversion has a positive net benefit; if not, it has a negative net benefit (i.e. a cost). For consistency, the assumptions used in the first benefit are also used in the second, specifically the value of time and that the hourly volumes used represent 8MDH.

4.2.3 Maintenance and operation benefit ($B_{M&O}$)

The third evaluator in the model is the maintenance and operations benefit ($B_{M&O}$). As mentioned previously, any infrastructure constructed needs to be maintained. For an unsignalized intersection, this annual maintenance involves the occasional replacement of a sign. (As the concrete, etc. exists and must be maintained for either the signalized or unsignalized condition, the operational schematic is irrelevant to the cost of this type of maintenance.) For a signalized intersection, power has to be maintained, computer equipment has to be repaired or replaced, and even light bulbs and signal lenses frequently have to be replaced or maintained; thus, there is a real annual
maintenance and operations cost for a signal that goes away if that signal is removed. There is a cost savings, a benefit received, for removing signals. Since the assumptions used for the travel time benefit were intended to generate conservative results, the same logic was used for the maintenance and operation benefit, with an annual maintenance and operation expense of $1000, the lower side of the range given previously in Section 1.2, used in the calculation of this benefit. In other words, the $B_{M\&O}$ is $1000 for every intersection analyzed.

4.2.4 Runoff benefit ($B_{R}$)

The third evaluator, the runoff benefit ($B_{R}$), accounts for the environmental impact of converting a traditional signalized intersection to an alternative unsignalized one. There is an environmental cost when pervious areas are changed to impervious due to increased pavement; conversely, there is an environmental benefit to converting impervious areas to pervious. Alternative intersections, by their very nature, typically have more pervious area than traditional ones, as they require medians of sufficient width to safely accommodate a U-turn, and these medians are typically turf; thus, a conversion to an alternative intersection creates more pervious area and the environmental benefit associated with it. This benefit is a reduction in runoff, which not only reduces the cost of sewage treatment, but also the size of stormwater conveyance facilities. As the size and type of conveyance devices is determined by the designer for a specific set of conditions for a specific project, it would be a Herculean task to perform any reasonable monetary comparison of the cost of conveyance devices due to limitless number of types and combinations that are possible. However, it is relatively simple to compare the cost of
treat a particular unit quantity of runoff, and comparing the total cost of treating the runoff generated for various design concepts. With this comparison, the environmental benefits of conversion from a traditional signalized to an alternative unsignalized intersection where the pervious area increases can be ascertained.

One approach to determining the value of these environmental benefits is to compare theoretical assessed fees for various levels of runoff for various geometric footprints within a particular, defined, area. Many cities have impact fees for additional stormwater runoff that are assessed on property owners at the time they are improved. One significant drawback of these fees is that they are one-time fees, and do not reflect the recurring cost of the additional stormwater. However, the City of Detroit (Detroit Water and Sewer, 2015) has addressed that limitation and assesses a monthly fee based on the amount of impervious area of $565.17 per month per acre, or $0.0129 per month per square foot, of impervious area. This value can be used to calculate the total monetary benefit of conversion of a traditional signalized intersection into an alternative unsignalized one where additional pervious area will be created. Even though this value is unique to Detroit, it is applicable anywhere, as it is being used for a comparison between two designs. As with the values used for the other benefit factors, this value can be replaced with another value more reflective of a particular locale as long as that value is used for all scenarios and analyses.

For a particular intersection being considered for conversion, the infrastructure choices are upgrading the existing traditional signalized intersection or conversion to an alternative unsignalized one. Annualizing the Detroit values yields a $B_{SR}$ of $0.1548$ per
square foot of additional pervious area per year. For the purposes of comparison of different geometries for a particular location, the net benefit gained by removing pavement (such as would occur with the conversion to an alternative unsignalized configuration) is given by:

$$B_{SR(Spy)} = 0.1548(A_A - A_T)$$

(EQ 4.5)

Where

- $B_{SR(Spy)}$ = Runoff benefit of conversion to an alternative intersection, dollars
- $A_A$ = Pervious area of the alternative intersection geometry, square feet
- $A_T$ = Pervious area of the traditional intersection geometry, square feet

4.2.5 Stopping benefit ($B_S$)

Upon conversion of an intersection from signalized to unsignalized, the number of vehicles that must stop changes. On the side street, the unsignalized state forces all motorists to stop, whereas in a signalized state, only some must stop, as some vehicles will enter the intersection on green and not have to stop. Thus, the conversion will be detrimental to motorists on the side street, as now all must stop. It is the opposite for the through street. Prior to conversion, some of the traffic on the through street, those vehicles that arrived at the intersection on red, must stop, while none of the traffic has to stop after conversion. Thus, the volumes of vehicles forced to stop under each scenario determines if the conversion will be a benefit or a detriment. Gerth (1982) established that stopping costs are $25,000 per year for every 1000 vehicles per day having to stop; these values will be used to calculate this benefit. When calculating the number of vehicles stopped by the traffic signal, it was assumed that the arrival of vehicles is random, and thus for a particular movement the number of vehicles not having to stop is directly
proportional to the percentage of green available for that movement. For example, if a movement is provided 40 seconds of green for a 60 second cycle (i.e. 2/3 of the cycle), then 2/3 of the vehicles are assumed to arrive on green, and only 1/3 are required to stop. The rationale for assuming a random arrival is detailed elsewhere in this chapter.

4.2.6 Construction costs (C$\text{C})

The final evaluator is not a benefit but a cost, the Construction costs (C$\text{C}). Intersections being considered for conversion from signalized to unsignalized typically are older and close to, if not at, the end of the signals’ useful life. One common reason to explore the possibility is the need to upgrade the signalization. Thus, the practitioner is faced with a choice – make a significant capital outlay to maintain signalization or remove the signals?

One way to determine this is by comparing the level-of-service for an intersection operating in signalized mode versus the level-of-service for that same intersection operating in an unsignalized mode. While it sounds simple and straightforward, it often is more complex. For example, what kind of unsignalized mode – all way stop or two way stop? While all-way stops maintain access to the through street and provide a safe crossing for pedestrians, operationally their application is limited to locations with the intersecting streets share similar geometric and operational characteristics, a small percentage of all intersections. For most intersections, a two-way stop unsignalized condition is appropriate. Some of these intersections have severe traffic volume imbalances between the major and minor streets, a situation where removal of the signals can improve the flow of traffic on the through street corridor while degrading the flow of
traffic on the side street. Additionally, in a built-up urban area, there may be locations where traffic flow is not degraded on any of the approaches or for any movement, but a simple traditional two-way stop controlled intersection cannot be used due to sight distance hazards (e.g. trees or structures) that are not practical to remove. In these exceptional cases, the practitioner is faced with a choice - a sizable capital outlay to replace and upgrade a signal that does not enhance the overall operational performance of the network (but, if removed, may have a detrimental impact for the side street movements), or a sizable capital outlay to remove the signal and mitigate the deficiencies through the construction of an alternative unsignalized intersection.

Since this research is focused on the particular situation where the practitioner must choose between a signal upgrade or an intersection upgrade, these capital costs must be included in the monetized model. In order to reduce the probability of a “false positive”, namely choosing a conversion when it is not economically justified, a worst-case scenario, maximizing the cost differential, will be used, and if the model reveals the conversion should be made, then the practitioner can have some level of confidence that it was the correct decision. To maximize the cost differential, a low cost of signal replacement and a high cost of intersection modification is used. For the calculation of the cost differential, both the signal upgrade and the intersection modification are assumed to have the same service life, twenty years, although it would be reasonable to assume that the intersection modification will have a longer life; reducing the life of the physical intersection modification increases its costs and reduces the likelihood of an overestimate of its benefit. To maximize the cost differential, and minimize the probability
of a false positive, the assumed capital cost of upgrading the existing signalization is $150,000, reflecting the lower end of the cost range, and the assumed capital cost of modifying the intersection to an unsignalized alternative one is $1.0 million, reflecting the higher end of the cost range. (Conversions to roundabouts represent the overwhelming majority of completed conversions for which cost values are available; thus, the $1,000,000 reflects the higher end of “conversion to roundabout” cost range. Since roundabouts are more expensive than conversions to other alternative geometries due to rights-of-way acquisition and structural demolition costs, it can be reasonably argued that a conservative construction cost value for a roundabout is applicable to other alternative geometries as well.) Annualized, these costs are $7,500 for the signal upgrade and $50,000 for the intersection modification. For all intersections analyzed, the cost of conversion is the difference in the cost of modification and a signal upgrade, or $42,500. If this cost exceeds the value of the sum of the five benefits, then the intersection should not be converted and no further analysis is necessary. If the converse is true, and the benefits exceed the cost, then the intersection should be considered for conversion and a social cost, in the form of pedestrian accessibility, analysis must be performed. The procedure for this social cost analysis will be discussed in depth later in this chapter.

4.3 Operational parameters to be compared

It is intuitive that, in order to determine the effectiveness of converting a traditional signalized intersection to an alternative unsignalized one, one must compare how it operates as a traditional signalized intersection to how it operates as an alternative unsignalized one. The most accurate way would be to physically make the conversion, use
before and after operational data in the monetization model, and compare the results to
determine if there was a cost savings or cost increase due to the conversion. This,
however, is not practical, as a governmental decision making body is most likely not going
to make a sizable capital expenditure without knowing beforehand the benefits derived
from the expenditure. The most practical way, then, to determine the net benefit or cost
of such a conversion of an intersection is by using a “best guess” prediction of the
operation impacts of the conversion. As there is always a measure of randomness and
uncertainty in any prediction, often a range of possibilities is used, with the “best case”
and “worst case” scenarios, the limits of the range, highlighted. For this study, the
monetized model was applied at each of the seventeen intersections under consideration
for conversion for three different operational scenarios: existing traditional signalized;
“best-case” alternative unsignalized; “worst-case” alternative unsignalized.

4.3.1 Traditional signalized operational scenario

For each intersection analyzed, the baseline scenario tested was the traditional
signalized operational one. With the existing conditions as the baseline, if the monetized
model for either the two unsignalized scenario is higher than for the baseline, then
conversion is not cost effective and should not be considered; in other words, both the
“best case” and “worst case” scenarios must provide superiority to the baseline, as one
cannot ascertain which condition will actually exist after conversion. While actual vehicle,
heavy vehicle, and pedestrian volumes, as well as cycle length and splits, are used,
operating speeds are assumed to be 30 miles per hour for all approaches at every
intersection. The rationale behind this is threefold. First, speed limits range from 25 to
35, and for the sake of consistency and simplicity, the average, 30, was used. Second, speed limits, unlike cycle lengths and splits, are more subject to be changed for political, rather than engineering, reasons. Since the volume data was collected over time, there is a likelihood that speed limits could be changed while data collection is ongoing, which would alter the results; therefore, setting all speed limits to a constant value, the average, eliminates this possibility. Third, spot speed studies show that, depending on time of day, operating speeds fluctuate, sometimes dramatically, and in order to mitigate the impact of these fluctuations, a constant speed representing a value in the middle of the range should be used.

4.3.2 “Worst case” alternative unsignalized operational scenario

For this scenario, it is assumed that the volumes for each movement will stay the same after the conversion. This is an extreme assumption, as changes to the operation of an intersection will change the driving characteristics of those who use it. As alternative intersections, by their very nature, involve the diversion of particular movements out of the intersection proper, then intuitively, some of the vehicles making those movements prior to conversion will not make those movements after the conversion due to the inconvenience. It would be challenging, if not impossible, to determine exactly how many vehicles will not make this movement without first constructing the conversion. The worst-case, then, is than no vehicles will be so disinclined, and that the exact same number of vehicles making a particular movement before the conversion will make the same movement afterwards. The effect, then, is to underestimate some of the benefits of the conversion.
4.3.3 “Best case” alternative unsignalized operational scenario

For this scenario, it is assumed that the volumes for the movements diverted will change. Because alternative intersections, by their very nature, may add impedance for certain movements at an intersection, specifically minor street left turns and through movements and major street left turns, it is important to consider these impedances when comparing traditional and alternative geometrics, as these impedances affect volumes. While a very quick and dirty comparison can be made using the same traffic volumes for each scenario, such a comparison would ignore a potential benefit of using unsignalized alternative intersections in lieu of traditional signalized ones, namely reductions in traffic volumes on the minor streets. Traffic volumes are often perceived as having a deleterious effect on community quality-of-life, as sometimes manifested by lower residential property values on higher volume streets. Thus, it is important to capture this benefit, especially when using a monetizing model.

The gravity model can be used to capture this impedance effect, as impedance is an integral part of how trips are distributed, with routes with less impedance receiving more trips, and those with more impedance, less. Travel time is a primary component of the impedance computation, and often is the only component, due to ease of measurement, repeatability, and objectivity. Using a classic gravity model impedance of \( 1/\tt^2 \) where \( \tt \) is the travel time in seconds, a mathematical relationship between the change in travel time and the percentage decrease in trips can be established. This mathematical relationship is described by the following equation:
\[ a = - \left[ \frac{50 - \frac{100}{(\tau^2 + 1)}}{50} \right] \]  

(EQ 4.6)

Where

\[ a = \text{percentage decrease in trips due to increase in travel time} \]
\[ \tau = \text{rate of travel time increase} = \frac{tt_N}{tt_O} \]
\[ tt_N = \text{new travel time, seconds} \]
\[ tt_O = \text{old travel time, seconds} \]

This relationship is shown graphically in FIGURE 4.1.

\[ \tau = \text{rate of travel time increase} = \frac{tt_N}{tt_O} \]

FIGURE 4.1. Relationship between the rate of travel time increase and the decrease in trips due to the increase in travel time.

The genesis of the derivation of EQ 4.6 is the gravity model equation defining the interchange of trips between two areas, \( i \) and \( j \), which is given by:

\[ T_{ij} = P_i \left[ \frac{A_j FF_{ij} K_{ij}}{\sum A_j FF_{ij} K_{ij}} \right] \]  

(EQ 4.7)
Where

\[ T_{ij} = \text{the trip interchange between zone } i \text{ and zone } j \]
\[ P_i = \text{the trips produced by zone } i \]
\[ A_j = \text{the trips attracted to zone } j \]
\[ FF_{ij} = \text{the impedance factor between zone } i \text{ and zone } j \]
\[ K_{ij} = \text{the uniqueness of the relationship between zone } i \text{ and zone } j \]

Typically, \( K=1 \), as most areas there is as much possibility and desire to travel from zone \( i \) to zone \( j \) as from zone \( j \) to zone \( i \) (Morlok, 1978) In Michigan, an exception would be Mackinaw Island, where islanders have to come to the mainland for employment, shopping, etc., while mainlanders do not have any reason to visit the island except for very specific tourism purposes. Also, \( K \) may vary from one for the interaction between zones separated by a significant feature that restricts that ability to freely pass between the zones, such as a river or an international border. The friction factor, \( FF \), is related to the travel time, \( tt \), and is commonly given as (Morlok, 1978; Papacostas, 1987):

\[
FF_{ij} = \frac{1}{tt_{ij}^b}
\]

(Substituting into the numerator of EQ 4.7 (and ignoring the denominator for the sake of simplicity) yields:

\[
T_{ij} = \frac{P_i A_j K_{ij}}{tt_{ij}^b}
\]

The exponent for \( tt_{ij}, b \), is typically assigned a value of two (Morlok, 1978; Papacostas, 1987), thus making the equation:
With that set up in place, an examination of the impact of changes in travel time in the desire to travel between \( i \) and \( j \) with all else being equal needs to be undertaken. For two \( ij \) routes, \( N \) and \( O \), the friction factor for \( N \) is:

\[
FF_N = \frac{1}{tt_N^2}
\]  
(EQ 4.11)

Correspondingly, the friction factor for \( O \) is:

\[
FF_O = \frac{1}{tt_O^2}
\]  
(EQ 4.12)

If the travel time for \( N \) is 4 and \( O \) is 2, the values for \( FF_N \) and \( FF_O \) using EQ 4.10 are 0.0625 and 0.25, respectively. The ratio of \( FF_N \) to \( FF_O \) is 1:4, while the ratio of \( tt_N \) to \( tt_O \) is 2:1; thus, a doubling of the travel time results in a quartering of the friction factor. For \( N \) and \( O \) travel time values of 6 and 2, the corresponding \( N \) and \( O \) friction factor values are 0.0278 and 0.25, with the ratio of \( FF_N \) to \( FF_O \) being 1:9, and the ratio of \( tt_N \) to \( tt_O \) being 3:1. Thus, the ratio \( FF_N:FF_O \) is the inverse of the square of the ratio \( tt_N:tt_O \). Algebraically, this can be expressed as:

\[
\Phi = \frac{1}{\tau^2}
\]  
(EQ 4.13)

where

\[
\Phi = \frac{FF_N}{FF_O}
\]  
(EQ 4.14)
\[ \tau = \frac{t_{N}}{t_{O}} \]  \hspace{1cm} (EQ 4.15)

For two alternative paths, \( \Phi \) represents the proportion using path \( N \) versus path \( O \). If \( \tau = 1 \), then \( \Phi = 1 \), meaning that both paths \( N \) and \( O \) are used equally; thus 50 percent will use path \( N \) and 50 percent will use path \( O \), and \( T_{ij(O)} = T_{ij(N)} \). When \( \tau = 2 \), then \( \Phi = 0.25 \), meaning the use of \( N \) is \( \frac{1}{4} \) that of \( O \), or \( T_{ij(N)} = 0.25T_{ij(O)} \), which can be rewritten as \( T_{ij(O)} = 4T_{ij(N)} \). When \( \tau = 3 \), then \( \Phi = 0.111 \), \( O \) is used nine times more than \( N \), or \( T_{ij(O)} = 9T_{ij(N)} \). For all values of \( \tau \), the equation is:

\[ T_{ij(O)} = \tau^2 T_{ij(N)} \]  \hspace{1cm} (EQ 4.16)

For nomenclature simplicity, \( \zeta_{O} = T_{ij(O)} \), and \( \zeta_{N} = T_{ij(N)} \), thus

\[ \zeta_{O} = \tau^2 \zeta_{N} \]  \hspace{1cm} (EQ 4.17)

The total trips between \( i \) and \( j \), \( T_{ij} \), is the sum of all trips between \( i \) and \( j \) using all paths. If all trips between \( i \) and \( j \) are made on the two paths, \( N \) and \( O \), then the sum of the volumes between \( i \) and \( j \) can be expressed by:

\[ T_{ij} = \zeta_{N} + \zeta_{O} \]  \hspace{1cm} (EQ 4.18)

Substituting EQ 4.17 into EQ 4.18 for \( \zeta_{O} \) yields

\[ T_{ij} = \zeta_{N} + \tau^2 \zeta_{N} \]  \hspace{1cm} (EQ 4.19)
Simplifying **EQ 4.19** gives

\[ T_{ij} = \zeta_N (1 + \tau^2) \]  

(EQ 4.20)

If \( \tau = 2 \), then \( T_{ij} = \zeta_N (1 + 2^2) \) or \( 9\zeta_N \), and since \( T_{ij} = \zeta_N + \zeta_O \), then \( \zeta_O = 4\zeta_N \).

\( \zeta_N \) can be used as a surrogate for FF\(_N\), and correspondingly \( \zeta_O \) can be used as a surrogate for FF\(_O\), so that:

\[ \Phi = \zeta_N / \zeta_O \]  

(EQ 4.21)

For \( \tau = 2 \), **EQ 4.21** becomes,

\[ \Phi = 1/4 \]

Since \( \zeta_N \) represents the smallest share of the total trip interchange, and \( \zeta_O \) is a multiple of \( \zeta_N \), then for the sake of simplifying the mathematical computations, the smallest share, \( \zeta_N \), will be given a value of 1. Substituting into **EQ 4.20** gives:

\[ T_{ij} = 1 + \tau^2 \]  

(EQ 4.22)

It should be noted that **EQ 4.22** is used in numerator of **EQ 4.6**. Since an equal distribution between \( O \) and \( N \) results in a 50%-50% distribution, it is necessary to convert this ratio to a ratio of the whole. To do that, **EQ 4.22** is divided into 100, to give the total percentage change from the steady-state equilibrium (50-50) condition, and then
subtracted from 50, which represents the equilibrium distribution condition. This difference is then divided by 50 (again, the equilibrium), to determine the percentage change from the equilibrium condition. Since increases in the travel time result in decreases in use, then the resulting quotient is assigned the negative of its value.

Because alternative intersections, by their very nature, impede minor street through and left turn movements, and may impede major street left turn movements depending on the design, it is imperative to consider the effects of this impedance when comparing a traditional intersection with an alternative one, and adjusting the traffic volumes accordingly. These adjustments can be calculated using EQ 4.6. For the signalized operation, the travel time will be the delay, as the control delay includes an acceleration factor for clearing the intersection. For the unsignalized alternative intersection, the travel time will be the actual travel time required to complete the movement as previously described. For movements where travel time decreases, no impedance reduction will be applied (as the conversion is not negatively affecting travel times).

For the minor street movements, it will be assumed that the lost vehicles, the number of vehicles reduced as calculated by EQ 4.6, are taking alternate routes, a reasonable assumption in a dense urban network where many alternate routes are available that will satisfy the California Diversion Equation (CDE) equilibrium state. NCHRP 765 (CDM Smith, Horowitz, Creasy, Pendyala, and Chen, 2014) defines the CDE as follows (EQ 4.23):
\[ P = 50 + \frac{50[(d_a - d_b) + 0.5(t_a - t_b)]}{\sqrt{[(d_a - d_b) - 0.5(t_a - t_b)]^2 + 4.5}} \]  

(EQ 4.23)

Where,

- \( P \) = percent diverted from route A to route B
- \( d_a \) = length of route A, miles
- \( d_b \) = length of route B, miles
- \( t_a \) = length of route A, miles
- \( t_b \) = length of route B, miles

The CDE is used to determine, given a choice of two possible routes from two points, the proportion of trips to use each route. If two routes are considered to be equal travel alternatives to a motorist, then \( P \) is 50. Setting \( P \) equal to 50, one can determine how much longer a person will travel for a time savings of one minute. Solving the equation yields an increase in one-half mile of travel trip length for every one minute of travel time savings, or 44 feet per second, which is the assumed velocity used in the calculation of the travel time benefit, \( BST_T \). In a dense urban grid, there are many such alternate routes that will satisfy this equilibrium; thus, the dispersion assumption is valid. However, for less dense grids, this assumption may not be valid, especially in locations with large lots and numerous streets with only one outlet. By using both the “worst case” and the “best case” scenarios in the analysis, this issue is moot, as all possibilities are addressed.

To summarize, the “best case” scenario accounts for traffic diverted from the particular movements impeded at a given intersection to elsewhere in the transportation network. The “best case” assumes the existence of a dense urban grid where impeded
vehicles have other options and thus will divert, and where such a dense urban grid exists, can be used with some confidence. However, where such a dense urban grid does not exist, the “best case” should be used with caution, as impeded vehicles may not divert due to the lack of viable diversion options. When the “best case” is considered, \textbf{EQ 4.6} is used for all movements impeded at a given intersection to determine the percentage of vehicles diverted, with the number of diverted vehicles for each movement deducted from the number of vehicles for traditional signalized operation for those same movements.

\textbf{4.4 Social impacts}

In addition to the monetized costs of converting a traditional signalized intersection into an alternative unsignalized one, there are also social impacts that should be considered. While these impacts are qualitative, quantitative values can be used as indicators of general impacts. For example, vehicle speeds are one such indicator, especially in dense urban areas, with higher speeds being perceived as deleterious to overall neighborhood quality-of-life due to the safety concerns wrought by higher speeds. In short, streets with higher speeds are often viewed as more dangerous, not only to motorists, but also pedestrians, as crashes at higher speeds typically result in worse injuries and damage than crashes at lower speeds. Another such quantitative indicator is the ability of a pedestrian to cross the street. The ability of pedestrians to cross the street can impact community cohesion, with more pedestrian accessible streets perceived as an integral part of a cohesive neighborhood.

\textit{Advocates of signalization and of coordinated systems of signalized intersections}
cite positive social impacts among the several benefits of such a system over non-signalization. Coordinated systems utilize what is known as progression, which is one of the eleven current warrants stated in the MUTCD for the installation of traffic signals. According to the progression theorists, signals spaced and timed appropriately along a linear corridor create green “bands”, platoons of vehicles that, when driving the appropriate progression speed, will always have a green indication. Under the progression theory, if motorists stray too far from the appropriate progression speed, they will not be able to traverse the corridor without stopping; the signal progression, then, becomes a form of speed control. Theoretically, if signals are timed for a progression of 44 feet per second, then vehicles will travel 44 feet per second. In order to maintain progression, signals cannot be spaced too far apart or the platoons created by the signals will start to disperse, and fewer and fewer vehicles will be able to traverse the corridor without stopping. To minimize overall delay along the corridor, this “green band” must be maximized, with signals placed at distances to prevent the dissipation of the platoon. The need to maintain progression has resulted in the placement of signals at locations that would not meet crash history or pedestrian or vehicular volume warrants. Yet, these signals must still be maintained at a cost of thousands of dollars per year in both operation and maintenance costs and other costs due to the increase in delay to vehicles having to wait for the green indication instead of taking the first available gap.

This is not to say that progression is not a worthy justification for installing signals. The ability to reduce overall delay, and its associated costs, is always beneficial. Furthermore, progression provides benefits to pedestrians and communities as well.
Progression provides pedestrians with consistent and predictable protected opportunities to cross the street. With this ability comes enhanced community cohesion, as the street no longer is a barrier from one side to the other. A valid concern about removing signals along a coordinated corridor is the removal of these consistent opportunities to cross the street. It is well within the realm of possibilities that removing signals could remove crossing opportunities, and thus have a detrimental impact on community cohesion by created a physical barrier to interaction between opposite sides of the street.

To determine the potential for negative social impacts by removing traffic signals, it is necessary to study both speed and platoon characteristics. Such an analysis provides a snapshot of actual speed and platoon characteristics. These characteristics, in turn, provide valuable insight as to the social impacts of a conversion.

4.4.1 Impact on vehicular speeds

As previously stated, one justification of signalization is for speed control through progression. Typically, signals in a progression system are spaced less than one-half mile apart in order to preserve the theoretical progression band, or platoons. In order to pass through the signals without stopping, the vehicles in the platoon must travel the progression speed, which is commonly the posted speed limit. Thus, one would expect the operating speed of a platoon within a coordinated progression system to be the posted speed limit. For the purposes of establishing operating speeds, the 85th percentile speed is used. The 85th percentile is the maximum speed at which 85 percent of the vehicles are traveling.

Speed studies were conducted at three different distances downstream of a stop
condition. A stop condition is one where motorists on a particular street or roadway must stop and then accelerate to a desired speed. Traffic signals and stop signs represent stopped conditions. The results of these studies are shown in TABLE 4.1.

<table>
<thead>
<tr>
<th>Miles Downstream</th>
<th>posted SL MPH</th>
<th>85th%ile speed (MPH)</th>
<th>MPH over posted SL</th>
<th>85th%/ posted SL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.95</td>
<td>45</td>
<td>59</td>
<td>14</td>
<td>1.31</td>
</tr>
<tr>
<td>1.50</td>
<td>45</td>
<td>54</td>
<td>9</td>
<td>1.20</td>
</tr>
<tr>
<td>0.12</td>
<td>35</td>
<td>43</td>
<td>8</td>
<td>1.23</td>
</tr>
</tbody>
</table>

**TABLE 4.1. Operating speeds at varying distances downstream of a stopped condition.**

While the sample size is small, and by no means all-encompassing, it does reveal virtually no difference in speeds between a progression corridor (the 0.12 mile sample, which was located between signals with progression) and a non-progression one (the 1.50 and 1.95 mile samples), which verifies a long-held theory by many practitioners, based on publications from the 1960s from notable practitioners such as the Institute of Traffic Engineers (now called the Institute of Transportation Engineers) and Donald Drew, that speed is determined more by the physical characteristics of the road than the operational characteristics. In other words, drivers drive what they feel is the safe and prudent speed for conditions regardless of what operations managers want drivers to drive. The Institute of Traffic Engineers, the precursor to the Institute of Transportation Engineers (ITE), emphasize this point in their 1965 edition of the *Traffic Engineering Handbook*:

“A speed regulation notifies the driver of the maximum or minimum speed that is considered safe for the conditions that exist under normal circumstances on the trafficway, and is intended to establish the standard or speed limit within which the driver may be expected to safely react to the driving problems he may encounter.” (ITE 1965), p. 530
Note that even in 1965 that speed limits were not intended to reflect desired operating speed, but to convey to the motorist information about the physical environment and the potential hazards that that motorist may encounter. In that same resource, ITE points out that it is the driver who makes the ultimate decision on what is the correct speed to drive, regardless of the presence of lack thereof of recommended or statutory speed guidance, based on factors both objective, such as prevailing conditions, and subjective, such as trip purpose. (That being said, one of the factors that drivers use to select the correct speed is the presence, or lack thereof, of signal progression.)

“The driver’s attitude which causes him to adopt a certain speed is often motivated by trip purpose.” (ITE (1965), p. 530)

“It is generally agreed, that with no traffic controls, the driver would adopt the speed at which he reasonably desires to travel under the prevailing conditions. Further, it is sometime assumed that a certain percentage (usually 15 percent) will normally exceed a safe and reasonable speed.” (ITE (1965), p. 531)

“Motorists tend to pay little attention to speed regulations which they consider unreasonable unless there is a high enforcement activity.” (ITE (1965), p. 531).

Drew, in his seminal work Traffic flow theory and control (1968), reinforces this concept of operating speed being related to design with several caveats:

“Assumed design speed should be a reasonable one...” (Drew (1968), p. 51)

“...low design speed cannot be arbitrarily assigned to all secondary roads, since many may be located in terrain which encourage high speeds.” (Drew (1968), p.52)

“The choice of design speed is based principally on what drivers will accept.” (Drew (1968), p. 52)

In an urban corridor, such as Jefferson, there are many factors that impact not only how fast drivers drive, but also the characteristics of the traffic stream itself. As stated by ITE in 1965:
“Driver variations and desires affect the flow of vehicles in traffic streams. Each driver wishes to select his own speed, which results in variations in desired speeds and in overtaking and passing maneuvers. Drivers also have different destinations, which result in lane changes and diverging and crossing maneuvers. Drivers also possess different degrees of ability and caution, as well as reaction times, resulting in variations in following distances and headways.

“Vehicle factors, such as acceleration, size of rear window, availability of automatic transmissions, etc., all affect starting times and resulting headways of vehicles when starting from a stopped position.

“Roadway factors affecting stream flow include such factors as the number of lanes and their width, effects of curbs and other obstructions along the road, spacing and design of ramps, grades, sight distances, and marginal and intersectional interferences.

“Traffic factors affecting stream flow include volumes in relation to capacities, vehicle classification, travel speeds, the extent of entering, merging, weaving and crossing traffic flows, and parked and stalled vehicles.” (ITE (1965), p. 170)

In the same volume, ITE also points out that:

“The proper timing of progressive traffic signal systems penalizes (and thus controls) the driver who travels too fast or too slow for the predetermined speed of the timed progression by forcing him to stop for red signals.” (ITE (1965), p. 545)

Despite their age, these seminal works by Drew and ITE, and their conclusions, have not been refuted or invalidated over the subsequent decades. Thus, until such a definitive repudiation exists, these conclusions should be considered valid in the present. In fact, given the added complexity of the present urban transportation environment (due to a wider variety of vehicle types, drivers, and driver tasks), one would expect a much greater variation in driver selected speeds, thus making it problematic and difficult to select a “correct” speed for signal progression (due to the greater variation in driver selected speeds), as any speed selected has a significant likelihood of excluding many vehicles. For all practical purposes, traffic flow in an urban environment is more random than organized, and should be analyzed as such. Since true progression is virtually impossible to achieve in an urban environment due to the inherent characteristics of the
urban environment and its high level of entropy (i.e. numerous access points, denser and more varied land uses, greater variety in trip purpose and mode of transport), converting a signalized intersection on a typical high entropy urban corridor to unsignalized will have minimal, if any, impact on progression, as whatever progression existed would be minimal (due to the entropy in the urban transportation environment), if any effective progression can exist at all at times of peak entropy. It should be noted that the effectiveness of any progression system inversely relates to the level of entropy on the corridor and in the system; the more closed the system, the lower the entropy, and the more effective the progression. These low-entropy corridors tend to have higher volumes (and correspondingly higher volumes of commercial and large vehicles) and speeds than their high-entropy counterparts; thus, the signals along these types of corridors tend to meet volume warrants (and are often part of a well-programmed progression system) and should not be considered for removal.

The small verification study illustrates this point of a lack of progression in an urban area during periods of high entropy. Note how the 85th percentile speed roughly two blocks downstream of a stopped condition in a corridor with progression, where acceleration could have an impact on the operating speed, is eight miles per hour higher than the posted speed limit, which is typically assumed by motorists to be and often used by practitioners as the progression speed. Interestingly, this eight miles per hour differential represents a higher percentage of overage of the posted speed limit than the nine miles per hour differential observed at a location 1.5 miles downstream of a stopped condition in a non-progression corridor (during an equivalent high entropy period). Even
at 1.95 miles downstream in a high entropy non-progression corridor, the differential between the actual operating speed and the posted speed is not much greater than the differential in the high-entropy progression corridor. Thus, based on the prevailing wisdom of the past half century, empirical observations by traffic engineers in various locations, and the small verification study, it can be reasonably concluded that signal progression is not an effective method of speed control at high entropy locations, and removing signals will probably not have any noticeable impact on speeds at these locations. (Since corridors with excellent progression typically have access control, they are low-entropy and progression has more value in controlling speeds; thus, removing a signal from such a facility may have a noticeable effect on speeds and such a decision should only be made judiciously.)

It could be argued that a corridor like East Jefferson passing through a depopulated area may not necessarily be a high entropy corridor despite numerous access points, as the land uses aren’t varied or dense (e.g. vacant), and there is not a great variety of trip purposes or modes of transport. Because the access points and potential land uses are already in place, the characteristics of this corridor could readily change into a high entropy one if these currently vacant properties are developed as intended. Thus, because the corridor has the potential for high entropy, it should be treated as such with respect to progression and speeds. If these potentials were to change by policy (change of zoning and potential land use) and design (elimination of access points), then the high entropy assertion would be dubious and have to be reexamined.
4.4.2 Impact on pedestrian crossing opportunities

At unsignalized locations located outside the zone of influence of a signal (typically 0.25 miles or more downstream of a signal), vehicles are assumed to arrive randomly, with appropriate sized gaps for pedestrians to cross arriving randomly. In contrast, at a signalized location, appropriate sized gaps for the pedestrian cross arrive at predictable intervals. For the removal of a signal to not be detrimental to pedestrian crossings, the number of appropriate gaps for the random unsignalized condition for a specific period of time should be the same as the number of appropriate gaps created by the signal for that same specific period of time. This specific period should, for simplicity’s sake, be equal to the cycle length. (If the cycle length is 60 seconds, then the appropriate time period for comparison for unsignalized operation should also be 60 seconds.) The number of gaps created in one cycle is one (as there are no gaps when the signal is green, and one gap when the signal is red); thus, the number of appropriate pedestrian gaps for a random arrival distribution in one cycle length equivalent is one. (To provide the same opportunity to cross, the same number of gaps must be provided). For example, a common urban cycle length is 60 seconds; thus, for signal removal to not have a detrimental impact on the ability of pedestrians to cross the street, assuming a random arrival, a pedestrian should have one appropriate gap per minute. Humphreys (1989) used the following equation to determine the maximum volume threshold for a given crossing width for a frequency of one appropriate gap per 60 seconds with a random arrival distribution:
\[ V_c = \frac{29000}{D} (2.322 - \log D) \]  

\text{(EQ 4.24)}

Where

\[ V_c = \text{Critical volume per hour} \]
\[ D = \text{crossing distance, ft} \]

Solving the equation for a crossing distance of 25 feet yields a critical volume of 1072, meaning that for this crossing distance, if more vehicles than the critical volume cross the unsignalized crossing point, then the number of acceptable gaps will be less than one per minute, and the ability of pedestrians to cross will be compromised if a signal is removed. For a distance of 70 feet, the critical volume drops to 173; thus, an unsignalized crossing with a random arrival distribution is only feasible for smaller crossing distances. Since unsignalized alternative intersections necessitate the construction of a median, the distance will be reduced, resulting in a higher maximum threshold. Furthermore, with the construction of medians, vehicles will only be arriving from one direction for each crossing, thereby increasing the likelihood of actual volumes being below the maximum critical volume threshold. For this particular analysis, the number of through traffic lanes in each direction will be reduced to two (for a total crossing distance of 25 feet when curb and gutters are included), and an appropriate gap frequency of one per 60 seconds will be used, yielding a unidirectional maximum critical volume threshold of 1072 vehicles per hour.

In this form, \text{EQ 4.24} has very limited application, being useful only for situations with 60 gaps per hour. Furthermore, it really cannot be used to determine if a particular volume and a particular width will provide enough gaps. When determining if a signalized
intersection can be replaced with an unsignalized such an analysis is crucial; after all, if
the unsignalized condition cannot accommodate the pedestrian volumes, then it can be
reasonably and justifiably argued that the signal should not be removed at all. For
example, replacing a fully functional signal with a pedestrian activated one will not
provide savings on operation and maintenance costs. Delay may actually be increased
with such a replacement as pedestrian activated signals cannot not easily be assimilated
into a progression system that maximizes platoons along a corridor. Because of the
random, uncoordinated nature of pedestrian activated signals, the number of vehicles
having to stop, and with it the associated costs, may actually increase. It becomes crucial,
then, to know if the removal of a signal will provide adequate safe crossings for
pedestrians. Such an equation can be derived from **EQ 4.24**.

**EQ 4.24** can be rewritten as follows:

\[
V_c = - \frac{29000}{D} (\log D - \log 210)
\]

(EQ 4.25)

Since

\[
\log \frac{D}{210} = \log D - \log 210
\]

(EQ 4.26)

Substituting **EQ 4.26** into **EQ 4.25** yields:

\[
V_c = - \frac{29000}{D} \log \frac{D}{210}
\]

(EQ 4.27)

Often it is easier to do computations with natural logs rather than common logs as natural
logs express an exponential distribution.
Substituting \( \text{EQ 4.28} \) into \( \text{EQ 4.27} \) yields:

\[
V_C = -\frac{29000}{D} \log e \ln \frac{D}{210}
\]  
\( \text{(EQ 4.29)} \)

Simplifying \( \text{EQ 4.29} \),

\[
V_C = -\frac{12600}{D} \ln \frac{D}{210}
\]  
\( \text{(EQ 4.30)} \)

The value of 210 in \( \text{EQ 4.30} \) is the solution of \( \text{EQ 4.31} \) for the value of 60, the 60 gaps per hour used for \( \text{EQ 4.24} \):

\[
210 = \frac{12600}{P} \text{ for } P = 60
\]  
\( \text{(EQ 4.31)} \)

Substituting the ratio of \( \text{EQ 4.30} \) into \( \text{EQ 4.31} \) results in a mathematical equation that can be used to verify adequate pedestrian crossing gaps at an unsignalized location for any pavement width or traffic volume, as expressed in \( \text{EQ 4.32} \).

\[
V_C = -\frac{12600}{D} \ln \frac{D}{12600 \frac{P}{P}}
\]  
\( \text{# # (EQ 4.32)} \)

Where

\begin{align*}
P &= \text{the number of pedestrians per hour} \\
V_C &= \text{the number of vehicles per hour} \\
D &= \text{crossing distance in feet}
\end{align*}
It should be noted that **EQ 4.32** expresses a negative exponential distribution, the established distribution pattern for gap occurrences. As this research pertains to the feasibility and practicality of replacing signalized intersections with unsignalized ones, an important component is the ability of pedestrians to cross the major street if the crossing is unprotected, specifically if there are sufficient gaps to cross. Specific design and operational parameters for each proposed unsignalized alternative intersection, i.e. crossing width and pedestrian volumes, are input into **EQ 4.32** and the calculated maximum traffic volume, $V_c$, is compared with observed volumes to verify that pedestrian volumes can be accommodated if the signals are removed. If the calculated volumes exceed the observed volumes, then pedestrians can be accommodated. A worst case scenario will be used for this calculation. As stated previously, three different scenarios for each location will be evaluated: the existing signalized conditions; the proposed unsignalized geometrics using the existing signalized traffic volumes; the proposed geometrics using volumes adjusted with **EQ 4.6** to take into account reduced demand caused by the geometric changes. The highest through street volumes in each direction from any of the three evaluation scenarios represent the worst case for pedestrians and will be used for the pedestrian verification analysis. To aid in future design and analysis, a table of vehicular values for selected pedestrian volumes and crossing widths can be generated. *(TABLE 5.1)*

As **EQ 4.24** assumes a random arrival distribution, it is then necessary to determine the validity of that assumption. For an urban signalized corridor like Jefferson, how well does the platoon maintain its cohesiveness? More importantly, given that any
intersection where the signal is removed will be between two other signals, if a signal is removed, can the assumption of random arrival be justified? To test this assumption, 31 gap studies were conducted downstream of signalized intersections, from distances of 200 feet to 850 feet. Studies were conducted at various times and various locations in order to detect unique pattern clusters. Each study was analyzed for the goodness-of-fit to a random distribution pattern, specifically a shifted negative exponential distribution pattern. Of these 31 studies, 9 satisfied the random distribution pattern. The results of these studies are shown in **TABLE 4.2**.

As can be seen from the table, there is no specific pattern with respect to traffic volumes per lane, number of appropriate gaps, or median type. The random distributions also showed no spatial correlations, occurring at the minimum and maximum of the range and other distances within the range, with dataset 15a at 200 feet downstream of the signal and 16a and 17a at 850 feet downstream of the signal. It should be noted that datasets with the same number but different suffixes (such as 1a and 1b) were collected at the same location on the same day but at different times, with the “b” set collected immediately after the “a” set. Note how 2a and 2b, 15a and 15b, and 17a and 17b have different results, while the other paired datasets have the same results. This is possibly due to a temporal effect on distributions, with distributions changing to and from random at a given downstream location throughout the day. However, due to the limited data, such an effect can neither be confirmed nor denied, but should be considered for future research. If distribution patterns do change to and from random throughout the day, it could have a significant impact on signal timing and progression theory, as well as data
collection techniques. The one point of commonality among the locations where random distribution was observed is the presence of more intense land uses, such as commercial or institutional, between the signal and the data collection location. (All locations were within urbanized areas of at least 30,000 people and along corridors with minimal access control.) Since most urban corridors have such intense land uses, it is reasonable to assume many of the downstream gap distributions will be random, and thus the use of a random distribution is an acceptable assumption. For locations where a more organized, less random distribution exists, EQ.4.24 will yield a conservative result, i.e. a lower volume

### TABLE 4.2. Chi-squared test for random distribution 200-850 ft downstream of a signal. Boxed datasets represent random distributions.

<table>
<thead>
<tr>
<th>Dataset</th>
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threshold for signalization than might actually exist. In other words, **EQ 4.24**, with its random arrival assumption, might call for pedestrian signalization when it is not needed.

**4.5 Summary of methodology**

The purpose of the methodology is to provide a mathematical model for the analysis of the potential replacement of traditional signalized intersections with alternate unsignalized ones. This model consists of two parts, both of which should be satisfied to consider total removal of signals- a net benefit value (NBV) analysis and a pedestrian accessibility analysis (PAA). The NBV consists of five benefits in annualized dollars—travel time, delay, maintenance and operation, runoff, and stopping—and one cost—construction, and if the benefits are greater than the costs, then the traditional signalized intersection is a viable candidate for replacement with an alternate unsignalized one. Applying the California Diversion Equation (CDE) microscopically at the intersection level, two different scenarios, a worst-case scenario, where no traffic is diverted after the conversion from a traditional signalized intersection to an alternate unsignalized one, and a best-case one, where the maximum amount of traffic is diverted, are calculated for the NBV benefits. The costs reflect the difference between the annualized cost of constructing an alternate unsignalized intersection versus a traditional signalized one.

The PAA can be used as an objective surrogate for the subjective community cohesion, with greater pedestrian accessibility yielding greater cohesion. The PAA is a measure of the ability of pedestrians to successfully cross at an unsignalized crossing. The PAA is a mathematical relationship between pedestrian flow, vehicular flow (for the same time period, an hour), and pedestrian exposure (the length of the crossing). The
relationship assumes random arrival of pedestrians; in other words, only one pedestrian crosses in each available gap in vehicular traffic, and thus the number of pedestrians per hour and the number of gaps per hour are the same. This assumption results in conservative results, i.e. conditions where pedestrians are safely accommodated even though the equation does not say such a condition exists. Thus, the status quo may be maintained for locations where it need not be.

As the methodology uses objective, mathematical equations, it can be used in any location under consideration for conversion. Values for the cost of time, construction costs, signal maintenance and operation, stormwater impact fees, etc., can be changed to reflect local conditions if necessary, which gives the methodology the utility to be applied in any locale. By including factors such stopping, runoff, and pedestrian accessibility, the methodology incorporates quality-of-life issues such as air and noise pollution (due to the repeated acceleration and deceleration of vehicles for signals), water pollution (due to the imperviousness of pavement and pollutants from the pavement being carried off in a storm event), and neighborhood cohesion and aesthetics (by reducing pavement widths and ensuring pedestrian accessibility from one side of the street to the other). Traffic operations, while a part of the overall methodology (i.e. delay), are not the only factor considered; thus, a more holistic approach to evaluating operational and geometric changes to the transportation network.
5.1 Design and geometric considerations

Much of the United States and Canada was surveyed in one-mile sections. (Kavanagh and Bird, 1989) Originally, these surveys were made using a Gunter’s chain, which is 66 feet in length, with 80 chains making a mile. (Kavanagh and Bird, 1989) Because fractional chains were challenging, measures were made in multiples of the chain. When the sections were divided, they were divided in quarter sections, which were subsequently divided into halves and quarters. Many cities and towns were platted into grids of streets 330 feet on center, with the 330 feet representing 5 chains, or 1/16 mile.

For a street with alternative intersections on consecutive blocks, the location of the median crossovers for one intersection should be located at a point outside the zone of influence of either intersection. According to Marconi (1977), at a lower-speed intersection where a vehicle has to stop, the zone of influence is 200 feet upstream and 100 feet downstream of the intersection. In other words, motorists do not start slowing down until 200 feet upstream, and reach uninterrupted free flow speed 100 feet downstream, of the intersection. (This assumes no impedances or queueing, which for most low volume urban intersections, is the norm. This assumption is not valid for locations with numerous driveways and access points to impede flow, and the ability to obtain free-flow speed, downstream, or for higher volume streets where queueing extends the zone of influence beyond 200 feet upstream) The median crossover for a particular lower speed urban intersection in a MUT configuration, where no geometric modifications are made to the intersection other than the addition of the median
crossovers, needs to be further than 100 feet downstream of and 200 feet upstream of that intersection, to prevent the median crossovers from interfering with operations of the intersection. In addition, the median crossovers need to be more than 200 feet upstream of adjacent intersections to keep them from interfering with those intersections as well. In order to meet this requirement, intersections must be spaced at least 400 feet apart. Along an access restricted urban corridor, or a suburban or rural corridor with few intersections, it is highly likely that this condition can be met. However, this is not the case in an established typical dense urban corridor, where streets are often spaced 5 chains, or 330 feet, apart. Because of this constraint, the ubiquitous use of the MUT along dense corridors (i.e. every intersection) is neither practical nor desirable.

Because both the ILAC and RCUT involve modifications to the intersections themselves, which, in turn, change the operational characteristics not only of the intersection modified but of the transportation network itself (due to changes in route choice resulting from the changes to the intersection), it is hard to definitely determine whether Marconi’s (1977) observations would apply to the median crossovers that are an integral part of the operational concepts of these intersection designs. A traditional MUT intersection functions as three different independent intersections, often with different types of traffic control. Through traffic at an MUT, from any direction, is not impeded in its ability to go through the main intersection; since there are no physical barriers preventing left turns at the main intersection. Geometrically, the main intersection under a MUT operational regime is no different from a traditional intersection, with all movements possible. The restrictions on movements is not physical, but statutory. In
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essence, the MUT is not one intersection but three distinct independent intersections, with motorists who volunteer to be statutorily compliant utilizing the two minor intersections to complete the left turn maneuver, and motorists choosing to not comply with the statute completing the left turn movement at the main intersection. By contrast, the RCUT and ILAC configurations are one integrated intersection, not multiple independent ones, as the use of the median cross-over is not voluntary, but mandatory due to physical geometric constraints. Unlike the MUT, where all movements can be served without the median crossovers, such is not the case with the RCUT and the ILAC. Although they look similar to the MUT, operationally they function more like a roundabout. In essence, then, both the RCUT and ILAC are a modified roundabout, and not a modified traditional intersection like an MUT. It can thus be reasonably argued that existing standards and research pertaining to MUTs and traditional intersections are not applicable.

If traditional standards for median cross-over locations are not applicable and should not be used, what standards should? First and foremost, each median crossing location is unique and should be designed as such, taking into account factors such as adjacent land uses, parking, driveway locations, pedestrian access needs, bus stops, and bicycle lanes, just to name a few. Second, it is imperative to have some recommended guidelines for the designer to use to help narrow down the possible locations of the median crossovers. Schrader (1990) notes that the distance required for a typical vehicle traveling approximately 30 miles per hour (a common speed on urban streets) to stop on dry pavement is 100 feet; Marconi (1977) also uses this value as the threshold for the
downstream zone of influence of an intersection. With the advent of fuel injection, onboard computers, and other advances in automotive technology, this downstream distance seems reasonable. For example, a 2017 Mazda 3 hatchback with an automatic transmission and a 4-cylinder 2.5 liter engine can accelerate from a dead stop to 60 miles per hour in less time (6.82 seconds) than it takes for a vehicle traveling at 30 miles per hour to travel one typical city block (330 feet). (Capparella, 2017) In other words, this vehicle can accelerate to typical urban speeds of 25 to 30 miles per hour within 100 feet, Marconi’s downstream zone of influence. Given that this is a typical compact car commonly found in urban areas, and not a high performance car or one with a larger and better performing engine, it is reasonable to conclude that most vehicles in the urban traffic stream have similar acceleration performance and can reach typical urban speeds within 100 feet.

In order to provide reasonable stopping distance, and to eliminate the various component parts of an integrated ILAC or RCUT intersection from influencing each other, the median crossovers for these types of intersections should not be less than 100 feet from the main part of the intersection. Marconi (1977) concluded that the upstream zone of influence of an intersection is 200 feet; thus an intersection (either street or driveway) should not be located less than 200 feet upstream of another intersection. Marconi conducted his research in 1977; the concept of RCUTs, ILACs, and other similar types of alternative intersection geometrics gelled subsequent to the publication of Marconi’s research. Thus, the concept of an intersection has changed. At an intersection, there are both merging (i.e. vehicles entering a traffic stream) and diverging movements that impact
the traffic stream. Because friction has a negative acceleration impact on motion (i.e. it wants to slow things down), it takes less distance to slow a vehicle down to a dead stop (as friction is contribution to the deceleration) from a particular speed than to accelerated to that speed from a dead stop (as friction is working against the acceleration). Thus, diverging movements (slowing down) do not have as much of an impact on the traffic stream (especially through movements) as do merging movements. Alternative intersections such as RCUTs and ILACs by their very design segregate merging and diverging movements. Because merging and diverging movements have different influences on the traffic stream, the upstream zone of influence distance observed by Marcon (1977) should be applied to each particular type of movement (i.e. diverging or merging). Applying this logic, a merging movement for an intersection should not be located within the upstream zone of influence of the merging movement for an adjacent intersection, and a diverging movement for an intersection should not be located within the upstream zone of influence of a diverging movement for an adjacent intersections. In other words, merges for adjacent intersections should not be less than 200 feet apart; likewise, diverges for adjacent intersections should not be less than 200 feet apart.

Placing median crossovers 1/3 and 2/3 of the distance between consecutive blocks that are a minimum of 5 chains apart will satisfy both of the aforementioned constraints. (FIGURE 5.1) While these specifications should be used when practical, that is the exceptional circumstance; in most locations, it is not. Because stopping distance as noted earlier is 100 feet, the minimum distance a median crossover should be from either a diverging or merging point, within the same or any other intersection, is 100 feet.
Furthermore, no points of access, either street or driveway, should be allowed either within the integrated alternative intersection or 100 feet downstream of the upstream median crossover and 100 feet upstream of the downstream median crossover.

This preferred location of crossovers should be used when possible, especially along corridors with access control where existing driveway locations are not an issue. East Jefferson, however, is not one of those corridors. Because of the numerous driveway and access points along the corridor, the proposed median crossovers could not be spaced uniformly in accordance with the ideal orthodoxy; instead, the median crossovers were located based on context sensitive design (CSD) principles. By using CSD instead of a prescriptive orthodox design, the crossovers were located to minimize potential conflict with existing access points; in other words, the access points dictated where the median crossovers should be located instead of vice versa.

Another significant issue to consider when analyzing the feasibility of using unsignalized alternative intersections as a replacement for a traditional signalized one is if it can be constructed for a reasonable cost, i.e. its constructability. The first constraint with respect to constructability is the adequacy of the existing right-of-way; specifically,
can an alternative intersection be constructed without removing any structures? The adequacy of right-of-way depends on the characteristics of the vehicles using the facility, with larger vehicles requiring more right-of-way. Traffic composition varies with land use, and land uses can and often do change over time. However, the variability of future land uses can be restrained by design; a corridor designed to accommodate large commercial vehicles will tend to have more intense commercial and industrial uses than those corridors not designed as such. The design parameters for a corridor should be set to best serve planned future land uses; thus, if a particular corridor is anticipated to have a residential nature, then designing it to easily accommodate large commercial vehicles, such as those one would find in an industrial area, would be counterproductive in accomplishing this future vision. Since there is a chance a large commercial vehicle will be using any given intersection (e.g. local delivery or emergency incidents), ideally, the design should be able to accommodate this worst case scenario. For a typical urban intersection, this worst case would be to design the intersection to safely accommodate a WB-50 semi-trailer combination, as this represents the largest vehicle typically found in most urban areas. Since the WB-50 has a larger sweep than the longer WB-60, using the WB-50 provides a more conservative design approach, as the design for a WB-50 will also accommodate the WB-60.

Because the corridor studied is an older built-up corridor, it has some significant right-of-way constraints. While ideally the crossovers should be able to accommodate the WB-50, for the East Jefferson corridor, this would be impractical due to the right-of-way constraints. However, traffic counts along the corridor reveal few, if any, trucks of that size
making the movements that would require the use of the crossovers. Furthermore, other than a few industrial parcels accessible by routes designed to provide such access (e.g. the auto plant accessible by Conner and Saint Jean), Jefferson is overwhelmingly residential and low-density commercial; thus, it is not really necessary, now or in the future, to easily accommodate these large trucks, as they are the infrequent exception. (On the rare occasion that a large vehicle needs to access adjacent properties, the dense street grid allows for the easy altering of route to utilize the remaining signalized intersections efficiently and effectively.) The largest vehicle observed making any of the movements in question is a single unit (SU) truck, so all crossovers have been designed to accommodate the SU vehicle. It is assumed that drivers of vehicles larger than an SU desiring to make a movement that would require using a median crossover will have knowledge of the physical constraints and plan their routes accordingly.

The second issue with respect to constructability pertains to corner sight distance, particularly the sight distance looking to the left on the minor street approaches. Ideally, there should be no sight distance obstructions for this movement in this direction. However, if such a sight obstruction exists it should be able to be easily minimized or removed. For example, parked cars along the major street can obstruct the line of sight of the minor street to the left, but this obstruction can be mitigated or removed by either signage or a simple geometric modification to the intersection. If the obstruction is of a permanent nature that cannot be easily modified or removed, such as a structure, then modification of a signalized intersection to unsignalized should be reconsidered, due to the inherent safety hazard created by the modification. For the intersections along East
Jefferson considered for conversion from signalized to unsignalized, this is not an issue.

5.2 Social cost analysis: pedestrian gaps

As mentioned previously, one measure of the social costs of conversion from signalized to unsignalized control is pedestrian accessibility, specifically the ability of pedestrians to cross the street. **EQ 4.32** allows for the computation of a maximum or critical volume, $V_c$, at which $P$ pedestrians can cross a distance of $D$ feet in width. (**EQ 4.32** assumes a worst case scenario of random arrival of pedestrians with only one pedestrian crossing at a time. In reality, there will often be occurrences of simultaneous crossing of numerous pedestrians; thus the vehicular volumes can actually be greater than $V_c$.) If $V_c$ is less than the actual volume, then there will not be enough safe gaps to accommodate the pedestrians (for the worst case random arrival scenario), and pedestrian activated signals will be needed. (If the number of pedestrians is equal to or greater than the number of gaps created by full signalization for several hours, then full pre-timed signalization should be strongly considered.) In situations where pedestrian activated signals are needed, the analyst should adjust the benefits and costs analyses to account for this addition. The solution for **EQ 4.32** for selected pedestrian volumes and crossing distances is given in **TABLE 5.1**. A graphical representation of **EQ 4.32** for crossing length values of 20, 22, 24, 30, 33, and 36 feet is presented in **FIGURE 5.2**. It is recommended that the analyst consult **TABLE 5.1** or **FIGURE 5.2** to determine whether or not an intersection can be converted from signalized to unsignalized before proceeding to the complete net benefit value (NBV) analysis. For illustrative purposes, NBV analyses were performed on all intersections contemplated for conversion as part of this study,
regardless of the outcome of the TABLE 5.1 or FIGURE 5.2 analysis.

In order to successfully complete a pedestrian accessibility analysis (PAA) using either TABLE 5.1 or FIGURE 5.2, the analyst must know two of the three variables, hourly vehicular volume, hourly pedestrian volume, or crossing length. If one knows the hourly pedestrian volume and the crossing length, then the hourly vehicular volume must not exceed the value shown at the intersection of the hourly pedestrian row and the crossing length column in order for pedestrians to be successfully accommodated with an

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<td>1108</td>
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<td>1292</td>
<td>1139</td>
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<td>1341</td>
<td>1164</td>
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<td>90</td>
<td>1226</td>
<td>1060</td>
<td>926</td>
<td>647</td>
<td>552</td>
<td>475</td>
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<td>120</td>
<td>1045</td>
<td>895</td>
<td>775</td>
<td>526</td>
<td>442</td>
<td>375</td>
</tr>
<tr>
<td>150</td>
<td>904</td>
<td>767</td>
<td>658</td>
<td>432</td>
<td>357</td>
<td>297</td>
</tr>
</tbody>
</table>

TABLE 5.1. Critical volume for given pedestrian volumes and crossing distances.
unsignalized operation. For example, the vehicular hourly volume must not exceed 1716 for 20 pedestrians an hour to be accommodated at a 24’ wide unsignalized crossing. As mentioned, a graphical presentation of TABLE 5.1 is shown in FIGURE 5.2. In FIGURE 5.2, any data points below and to the left of a selected crossing length curve can be accommodated with an unsignalized crossing, while data points above and to the right of a selected crossing length curve cannot. For example, a crossing with 60 pedestrians per hour and 1000 vehicles per hour can be unsignalized if the length is 24 feet or below, and cannot be unsignalized for lengths of 30 feet or more. Interpolating between the crossing length curves, the maximum length of an unsignalized crossing for this combination of pedestrian and vehicle volumes is approximately 26 feet. FIGURE 5.2 is thus a useful tool

**FIGURE 5.2.** Graphical representation of critical vehicular volumes in vehicles per hour for a given hourly pedestrian volume and various crossing lengths in feet.
to establish the appropriate length of an unsignalized crossing for a given combination of pedestrian and vehicular volumes, which can then be incorporated into the design of any modifications to a signalized intersection being converted to an alternative unsignalized one.

One concern often used for the justification for unnecessary signals is that unsignalized crossings are less safe, although this is conjecture. The danger to a pedestrian crossing a street is exposure, and for a crossing of a given length and composition, the exposure is the same whether or not a crossing is signalized, as motorists can, and often do, run traffic signals. In most jurisdictions, pedestrians in crosswalks have the legal right-of-way and motorists must yield to pedestrians in a marked crosswalk, whether signalized or unsignalized. Furthermore, one of the inherent dangers of a signalized crosswalk is turning vehicles, especially turns on red, where a motorist looking for clear gaps to complete the turning maneuver do not notice the pedestrian in the crosswalk, resulting in a vehicle-pedestrian collision. There are solutions to make unsignalized crossings safer. Reducing the exposure distance by reducing pavement width is the most effective way, as less exposure means less likelihood of a vehicle-pedestrian collision. Using a textured and colored crosswalk helps provide sensory notification of a pedestrian crossing and alert the motorist of the potential pedestrian activity at the location. Special pavement markings, such as diagonal bars, are also useful at enhancing safety at unsignalized pedestrian crossing locations (and unlike when used at signalized locations, will not accidentally actuate video-detected signals). Finally, speed tables are also useful in not only delineating the pedestrian crossing, but also slowing motorists in
the pedestrian crossing zone.

5.3 Signalized intersections reviewed or analyzed

As mentioned previously, the East Jefferson corridor runs 6.0 miles from the I-375 freeway spur on the east edge of downtown Detroit east to the Grosse Pointe Park city limit. East Jefferson is 90 feet in width and accommodates nine lanes throughout this length. The corridor is mixed use, consisting of retail, industrial, institutional, and high-density residential land uses. There are 25 traffic signals on the corridor, the westernmost of which is Rivard Street and the easternmost of which is Alter Road. (The signalized portion of the corridor is slightly less than the entire corridor, a distance of 5.8 miles from the westernmost signal at Rivard Street to the easternmost signal at Alter Road.)

Subsequent to the commencement of this study, a median was added between the penultimate easternmost signal at Chalmers Street and the signal at Alter Road. Because of this significant capital investment, consideration of any geometric alterations along the 0.3 miles of the corridor east of Chalmers was dropped, including modifications to the intersection of Alter Road. Thus, Alter was dropped from the analysis, and the easternmost signal considered was Chalmers. (For the purposes of this analysis, the corridor consists of 24 signals in 5.6 miles. It should be noted that although the signal at Alter was not considered for removal, due to its close proximity to Chalmers, it should be considered as part of the same signal progression system as the signal at Chalmers.)

Operationally, there are five distinct subcorridors of the East Jefferson corridor. When discussing issues such as signal progression, one cannot address the entire corridor as a whole due to the distinct differences between these five subcorridors, but must
consider each of these subcorridors to be functionally independent of each other. However, due to the inherent safety hazards of inconsistent geometric design (as drivers are expecting consistency), the geometric design should provide a consistent transition from one operational subcorridor to the next as well as being consistent within each subcorridor. As one travels from west to east along East Jefferson, these five corridors can be identified as follows: City Center; Belle Isle Access; Depopulated Zone; Conner Creek Industrial; Suburban Transition. Current operational characteristics of and proposed geometric modifications for all intersections along the East Jefferson corridor are detailed in APPENDIX B.

5.3.1 City Center subcorridor

The City Center subcorridor is approximately 1.8 miles in length and runs between Rivard Street on the west and Concord Street on the east. Within this subcorridor are seven traffic signals in a 1.6 mile distance, approximately one traffic signal every three blocks. If the spacing of the signals in this subcorridor were consistent, two way progression might be possible; since the spacing varies from signal to signal, two-way progression, for all practical purposes, is not, making this subcorridor inefficient with respect to both operations and the physical environment (specifically, unnecessary pollution). The City Center subcorridor has very dense and intense land uses typically found in urban areas adjacent to a central business district, such as multifamily residential and commercial. Due to the density of this subcorridor, off-street on-site parking for businesses and residents is not as readily available as in the other subcorridors; thus, not only is on-street parking in high demand, parkers have to walk farther to get to their
destination. The result of this scarcity of parking is higher volumes of pedestrians than are typically found elsewhere along East Jefferson.

The results of the analysis of the seven signals in this subcorridor are shown in **TABLE 5.2.** Of these seven, four, Rivard, Chene, McDougall/Walker, and Mount Elliott, were not considered for operational modification due to their importance to maintaining connectivity in the overall street network. These four streets are all either major collectors or arterials that are major points of access to and from the East Jefferson corridor, and adding any impedances to their current functionality and operation would have a detrimental impact on the city transportation network. Two of the intersections, Saint Aubin (north leg) and Dubois, can be converted from traditional signalized intersections to alternative unsignalized ones without any deleterious effects on either traffic flow or the ability for pedestrians to cross the street. The seventh signalized intersection, Joseph Campau, is a “maybe”. While conversion would be beneficial to motorists, it would not be beneficial to pedestrians, as there would be an insufficient number of gaps to service the pedestrian demand; thus, any conversion would have to include the installation of pedestrian-activated signals, such as the HAWK signals found in the vicinity of Wayne State campus. In such a case, the annualized cost of the signals would be added, not subtracted, from the annualized cost of the conversion, as it could no longer be considered a benefit. If all three signals are removed, the spacing of the remaining four would be more conducive to the possibility of achieving two-way progression through this subcorridor, even with a reduction of through laneage on East Jefferson from three to two. (See **APPENDIX A** for detailed progression analysis.)
### TABLE 5.2. Changes to signalized intersections in City Center subcorridor

<table>
<thead>
<tr>
<th>Signal #</th>
<th>Street</th>
<th>Can be converted?</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Rivard</td>
<td>✓</td>
<td>Not considered as this is a major connector</td>
</tr>
<tr>
<td>2</td>
<td>St Aubin (north leg)</td>
<td>✓</td>
<td>Converted to an RCUT</td>
</tr>
<tr>
<td>3</td>
<td>Dubois</td>
<td>✓</td>
<td>Converted to an RCUT</td>
</tr>
<tr>
<td>4</td>
<td>Chene</td>
<td>✓</td>
<td>Not considered as this is a major connector</td>
</tr>
<tr>
<td>5</td>
<td>Joseph Campau</td>
<td>✓</td>
<td>Insufficient pedestrian gaps; would need to provide pedestrian signalization such as HAWK</td>
</tr>
<tr>
<td>6</td>
<td>McDougall/Walker</td>
<td>✓</td>
<td>Not considered as this is a major connector</td>
</tr>
<tr>
<td>7</td>
<td>Mount Elliott</td>
<td>✓</td>
<td>Not considered as this is a major connector</td>
</tr>
</tbody>
</table>

**SUBCORRIDOR: CITY CENTER**

#### 5.3.2 Belle Isle Access subcorridor

The Belle Isle Access subcorridor is the shortest of the five subcorridors and contains the fewest number of signals. This subcorridor is approximately 0.4 miles in length and runs between Concord Street on the west and Sheridan Street on the east. Within this subcorridor is one traffic signal, at East Grand Boulevard, which is the entrance to Belle Isle Park. What makes this subcorridor unique is the geometric configuration, namely alternative intersections, both signalized and unsignalized; this is the only segment of East Jefferson where alternative geometric designs currently exist. This less than one-half mile subcorridor contains elements of the MUT, ILAC, and the Reverse RCUT (RRCUT), where the minor street left turns are direct and the major street left turns are indirect, as well as an interchange. With the conversion of the adjacent corridors to alternative geometric designs, this subcorridor will be better integrated into the East Jefferson corridor as a whole, which is beneficial for aesthetics, safety, and operations. Since the existing signal already incorporates alternative geometrics, and serves the only
access to Belle Isle, no changes to either the geometrics or operations are proposed for this intersection. (TABLE 5.3) Due to the proximity of the signal at East Grand to the signal at Mount Elliott, with the enhancements to the City Center subcorridor and the consistency between that subcorridor and this one, the signal at East Grand could feasibly be included in whatever signal progression is created for the City Center subcorridor, and the two subcorridors would function as one.

5.3.3 Depopulated Zone subcorridor

The Depopulated Zone subcorridor is the longest of the five subcorridors and contains the greatest number of signals, eleven. This subcorridor is approximately 1.9 miles in length and runs between Sheridan Street on the west and Defer Place on the east. Ironically, while this subcorridor has the highest density of traffic signals, it also has the lowest density land uses, as much of the surrounding neighborhoods have depopulated (many blocks are more than 50 percent vacant). Because of this phenomenon, most of the minor streets at the existing signalized intersections are very low volume (less than 500 vehicles per day) and the signals are not warranted. Due to the overall dearth of people and activity in along this subcorridor, parking supply far exceeds demand, and few pedestrians.

<table>
<thead>
<tr>
<th>Signal #</th>
<th>Street</th>
<th>Can be converted?</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>East Grand Bl</td>
<td>Yes</td>
<td>Not considered as this is a major connector, the only access to a major public recreation facility, and already has an alternative geometric design</td>
</tr>
</tbody>
</table>

TABLE 5.3. Changes to signalized intersections in Belle Isle Access subcorridor
The results of the analysis of the ten signals in this subcorridor are shown in TABLE 5.4. Unlike the other subcorridors, all of the signalized intersections in this subcorridor can be converted from traditional signalized intersections to alternative unsignalized ones without any deleterious effects on either traffic flow or the ability for pedestrians to cross the street. Six of these eleven (Seyburn, Van Dyke, Burns, McClellan, Cadillac, and Garland Marquette) are proposed to be converted to full RCUTs. Two of the existing signalized intersections (Harding and Lillibridge) are proposed to be converted to partial RCUTs.

<table>
<thead>
<tr>
<th>Signal #</th>
<th>Street</th>
<th>Can be converted?</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>Baldwin</td>
<td>✓</td>
<td>Converted to an RRCUT in conjunction with conversion of Signalized Intersection 10 to an RCUT as both serve same property.</td>
</tr>
<tr>
<td>10</td>
<td>Seyburn</td>
<td>✓</td>
<td>Converted to an RCUT in conjunction with conversion of Signalized Intersection 9 to an RRCUT as both serve same property.</td>
</tr>
<tr>
<td>11</td>
<td>Van Dyke</td>
<td>✓</td>
<td>Converted to an RCUT.</td>
</tr>
<tr>
<td>12</td>
<td>Parker</td>
<td>✓</td>
<td>Converted to an ILAC. Due to proximity to Signalized Intersection 11, can use that intersection's geometry.</td>
</tr>
<tr>
<td>13</td>
<td>Burns</td>
<td>✓</td>
<td>Converted to an RCUT. Due to positive offset, can provide direct access to both legs.</td>
</tr>
<tr>
<td>14</td>
<td>Hubbard</td>
<td>✓</td>
<td>Converted to an ILAC.</td>
</tr>
<tr>
<td>15</td>
<td>McClellan</td>
<td>✓</td>
<td>Converted to an RCUT.</td>
</tr>
<tr>
<td>16</td>
<td>Cadillac</td>
<td>✓</td>
<td>Converted to an RCUT.</td>
</tr>
<tr>
<td>17</td>
<td>Garland/Marquette</td>
<td>✓</td>
<td>Converted to an RCUT. Due to positive offset, can provide direct access to both legs.</td>
</tr>
<tr>
<td>18</td>
<td>Harding</td>
<td>✓</td>
<td>Converted to an RCUT. Due to negative offset, can provide direct access only to S leg.</td>
</tr>
<tr>
<td>19</td>
<td>Lillibridge</td>
<td>✓</td>
<td>Converted to an RCUT. Due to space constraints, can provide direct access only to S leg.</td>
</tr>
</tbody>
</table>

TABLE 5.4. Changes to signalized intersections in Depopulated Zone subcorridor
allowing direct left turns to only one of the minor street approaches (south) due to geometric limitations (a negative offset at Harding and physical space constraints at Lillibridge). A detailed narrative as to why the south legs were selected for direct access is provided in the corresponding sections of APPENDIX B, with general guidelines for the application of particular alternative designs presented in APPENDIX D. Two signalized intersections (Parker and Hubbard) are proposed to be converted to ILACs, as Parker is only one block from Van Dyke (thus, vehicles wanting to turn left into the neighborhood via Parker can use Van Dyke), and Hubbard has a negative offset that hinders the ability to turn left from Jefferson onto either leg. Because of the close proximity of Parker to Van Dyke, they can share geometrics. It should be noted that for the sake of consistency of the analysis, these two intersections were considered to be completely independent of each other in order to maximize the costs and minimize the benefits (e.g. a conservative approach). The proposed geometric configuration for the signalized intersection at Baldwin is an RRCUT; since Baldwin serves as the egress for a hospital (with Seyburn serving as the ingress), an RRCUT is the logical choice at the location as it will provide direct access for emergency vehicles leaving the hospital and responding to a call, a situation where seconds are crucial. (It should be noted that if the Depopulated Zone were to repopulate, the converted intersections can easily be resignalized.)

5.3.4 Conner Creek Industrial subcorridor

The Conner Creek Industrial subcorridor is 0.73 miles in length and transverses the Conner Creek Industrial Park between Defer Place on the west and Navahoe Street on the east. There are two signals in this subcorridor at Saint Jean (Signalized Intersection 20)
and Conner (Signalized Intersection 21). Because these two signals are more than one-half mile apart, they cannot be effectively coordinated with other (due to the dissipation of the progression platoon over such a long distance), and for all practical purposes, operate as independent signals. Due to their presence in an industrial corridor, both have a relatively high percentage of heavy truck traffic. Furthermore, both provide connections from the industrial area to other major routes such as Mack, Warren, and the Edsel Ford Freeway (I-94). Because of these characteristics, neither one is a viable candidate for removal and the status quo at these intersections will be maintained. (TABLE 5.5)

<table>
<thead>
<tr>
<th>Signal #</th>
<th>Street</th>
<th>Can be converted?</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>St Jean</td>
<td>✔️</td>
<td>Not considered as this is a major connector with relatively high truck traffic</td>
</tr>
<tr>
<td>21</td>
<td>Conner</td>
<td>✔️</td>
<td>Not considered as this is a major connector with relatively high truck traffic</td>
</tr>
</tbody>
</table>

TABLE 5.5. Changes to signalized intersections in Conner Creek Industrial subcorridor

5.3.5 Suburban Transition subcorridor

The easternmost subcorridor is the Suburban Transition subcorridor, in which the characteristics of East Jefferson transitions from urban to suburban. In actuality, this subcorridor extends 1.09 miles from Navahoe to the Detroit/Grosse Pointe Park boundary; however, since the section east of Chalmers has been recently reconstructed with raised medians and unsignalized ILAC configurations, consistent with what is proposed in this research for the remainder of the corridor, it has not been included in this analysis. Because of this, this subcorridor has been truncated at Chalmers for this
5.6. Changes to signalized intersections in Suburban Transition subcorridor analysis, yielding a subcorridor 0.75 miles in length with three traffic signals. The westernmost of these, Dickerson, can be converted to a full RCUT intersection due to the presence of a positive offset condition between the two legs of Dickerson. The middle signal, Coplin, is proposed to be converted to a partial RCUT due to a negative offset between the two minor street legs that geometrically prohibits the use of a full RCUT. Since the north leg of Coplin is a public street and the south leg of Coplin is a private drive, full access is shown to be provided to the north leg, as the public street cannot be easily restricted or moved like the private driveway. The negative offset intersection at Chalmers was modified in conjunction with the modifications east of Chalmers, with the north leg functioning as a traditional T-intersection and the south leg functioning as an ILAC. Although the signal was removed for the south leg, it was not for the north leg; thus, it was analyzed for the removal of the signal using this new configuration. As can be seen from **TABLE 5.6**, all three signals can be successfully removed.

### TABLE 5.6. Changes to signalized intersections in Suburban Transition subcorridor

<table>
<thead>
<tr>
<th>Signal #</th>
<th>Street</th>
<th>Can be converted?</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>Dickerson</td>
<td>✓</td>
<td>Converted to an RCUT. Due to positive offset, can provide direct access to both legs.</td>
</tr>
<tr>
<td>23</td>
<td>Coplin</td>
<td>✓</td>
<td>Converted to an RCUT. Due to negative offset, can provide direct access only to N leg, which is a public street. (S leg is private drive.)</td>
</tr>
<tr>
<td>24</td>
<td>Chalmers</td>
<td>✓ *</td>
<td>Already converted. N leg has full access, and S leg is an ILAC. (*Analyzed status quo.)</td>
</tr>
</tbody>
</table>

The methodology for analyzing the possible conversion of a traditional signalized
intersection to an alternative unsignalized one consists of two parts: a net benefit value (NBV) analysis and a pedestrian accessibility analysis (PAA). The PAA uses a mathematical equation using the pavement width, hourly pedestrian volume, and hourly vehicular volume to determine if an unsignalized crossing will successfully accommodate the number of pedestrians wanting to cross. The NBV is computed by adding five different benefit values of converting the intersection from a traditional signalized intersection to an alternative unsignalized one – delay ($B_{D}$); maintenance and operation ($B_{M&O}$); runoff ($B_{R}$); stopping ($B_{S}$); travel time ($B_{TT}$) – and subtracting the costs of the conversion. If both the PAA and the NBV yield positive results, then the intersection can be converted from a traditional signalized to an alternative unsignalized one without detriment, and should be seriously considered for such a conversion.

Seventeen unique intersection configurations, both operational and geometric, were successfully analyzed using this methodology, the results of which are shown in

<table>
<thead>
<tr>
<th>SUBCORRIDOR</th>
<th>SIGNALS</th>
<th>CROSSTINGS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Remaining</td>
<td>Removed</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>3*</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>11</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>TOTAL</td>
<td>7</td>
<td>17*</td>
</tr>
</tbody>
</table>

* Signalized Intersection 5 (Joseph Campau) can be converted to an alternate geometry and the signals removed only if a pedestrian activated signal is installed

TABLE 5.7. Results of analysis.
TABLE 5.7. Of these seventeen, one, Signalized Intersection 5, resulted in a negative output, i.e. the intersection cannot be converted unconditionally, as the PAA revealed that an unsignalized crossing would not be able to adequately accommodate all the pedestrians wanting to cross at this location. This problem can be resolved with the installation of pedestrian-activated signals that only stop traffic when a pedestrian needs to create an adequate gap to cross the street. If this modification to the alternate unsignalized geometrics can be made, then this intersection can be converted. (TABLE 5.7 reflects this.) While the NBV was positive for all seventeen intersections, it was marginal for Signalized Intersection 19. A slight change in the input values for the NBV could result in a negative value at this location; if this were to be, then the conversion should not be made at this intersection as well, even though the PAA was positive, as both the PAA and the NBV must be positive. Because of the fiscal and budgetary constraints of Detroit, these seventeen intersections would need to be prioritized using the ROI method to ensure the most efficient use of scarce financial resources. Such an analysis is presented in APPENDIX C.

If the corridor is modified as a whole, rather than just individual intersections, a total of 54 median crossovers, 27 for eastbound traffic and 27 for westbound traffic. (TABLE 5.7) Of those 54, only eighteen, ten eastbound and eight westbound, would not be needed as part of the intersection modifications. In other words, if all seventeen of the intersections that can be modified are, then two-thirds of the crossovers needed for the entire corridor to be converted would need to be constructed. Assuming that each intersection cost $1 million to modify, and each additional crossover costs $500,000, then
the cost to modify the entire corridor would be $26 million, or $4.33 million per mile. This estimated cost is a conservative one, as several intersections share modifications and thus costs.

If the proposed improvements are constructed for the entire corridor, approximately 14 acres of land would be converted from impervious to pervious with the construction of the medians, a reduction of the impervious area by one-fourth. Such a sizable reduction in impervious area is good for the environment by resulting in a sizable reduction in runoff from the street pavement, reducing not only the amount of runoff, but also potential toxins from both vehicles (such as gasoline, oil, antifreeze, and hydraulic fluid) and pavement treatments (such as bitumens, lime, and deicing compounds) typically found on street pavements.

The number of on-street parking spaces would be reduced with the full implementation of the proposed changes. Eleven of the proposed crossover locations would require removal of on-street parking on both sides of the street, approximately four on each side, for a total of 88 spaces removed, or approximately six percent of the total possible on-street parking inventory. Of these eleven locations, ten are in Subcorridor 3 (Depopulated Zone), where on-street parking supply far exceeds demand and is prohibited during certain periods of the day. In other words, only eight spaces, for all intents and purposes, will be removed, less than 1 percent of the possible on-street parking available along the corridor. It can be reasonably concluded, then, that the construction of the proposed improvements along the entire corridor will have a minimal effect on parking.
To maintain a good two-way progression system, signals must operate on the same cycle length (or multiples thereof) and have consistent spacing (or multiples thereof); the existing signals on Jefferson have neither. Because of this, it is highly improbable for a motorist to be able to drive from one end of the corridor to the other without stopping. The proposed improvements prescribe the removal of seventeen of the existing signals; an unlucky motorist who previously was stopped at each one would now save over eight minutes in travel time to drive six miles. Due to the spacing between the remaining signals, it is not practical to consider progression for the entire corridor; however, progression for a particular subcorridors is more easily achievable. (A complete progression analysis of the existing and proposed signalization is presented in APPENDIX A.) Since Jefferson would now function as two independent one-way streets, each direction can have its own progression, rendering the consistent spacing requirements for two-way progression moot. Additionally, pedestrians activated signals can be included as part of a progression system to minimize disruption to traffic flow where needed.

Construction of the proposed changes will be beneficial for the overall safety of both pedestrians and motorists along the corridor. With the construction of the median, pedestrians will have less exposure, crossing two narrower streets instead of one wide one. In addition, the median provides a safe-refuge for slower pedestrians such as children, the handicapped, and the elderly. Finally, pedestrians will only need to be concerned about traffic (and suitable gaps) from one direction, not two, reducing stress and anxiety for the pedestrian and thus reducing unnecessary risk-taking. For motorists, the proposed improvements eliminate most perpendicular conflict points (such as left
turns or through movements on the minor side streets) and potential head-on collisions (as there would be a physically barrier separating opposing directions of traffic); as these are the most severe types of crashes, reducing the possibility of these types of crashes occurring improves safety. One caveat must be stated – the construction of the proposed improvements is not a panacea; there will still be crashes, but not as severe, as the preponderance of crashes along a corridor such as being proposed are angle, sideswipe, or rear-end crashes, which generally result in fewer serious injuries or fatalities.

The proposed changes should improve the overall aesthetics of the corridor, a benefit to both businesses and residents alike. As stated previously, the easternmost segment of the corridor from Chalmers to Alter has already been modified, and with it improved aesthetics. An aesthetically pleasing location is a positive benefit for any business, as it encourages potential customers to visit and hopefully patronize. A recent conversion of a section of Livernois in the vicinity of the University of Detroit Mercy transformed a very cold and intimidating corridor into a warm and inviting one. Where Livernois has not been converted, vehicles travel at higher speeds and the street is barren and devoid of life; where it has been converted, vehicles are traveling slower, and the street is much more vibrant. Such a conversion benefits both businesses and residents by improving the quality-of-life.

At all of the seventeen intersections analyzed for conversion, the few heavy vehicles and bicycles present tend to be part of the major street through traffic stream. Eliminating signals, then, will improve the operational efficiency of these vehicles by requiring fewer stops and starts and allowing the operators to remain in higher gears. For
diesel trucks and buses, not having to stop and accelerate as frequently will have a positive effect on air pollution. For bicycles, removing direct and conflicting movements, such as left turns and minor street through movements, will improve safety. While conversion from traditional to alternative geometrics will make certain movements (i.e. left turns) more challenging for larger vehicles (and in some locations, not possible due to geometric restrictions), in a dense urban network, these challenges can be overcome by a simple change of routing to utilize remaining traditional signalized intersections.

It is important to note that not all of the intersections analyzed can be easily converted from a traditional signalized to an alternative unsignalized configuration, as one would require pedestrian-activated signals. This is an affirmation of the methodology. If every one of these seventeen different intersections yielded the same results, i.e. that the intersection can be successfully converted, one could rightfully question its goodness and usefulness, as one would not expect such a result. In other words, one expects that not every intersection analyzed will be able to be converted, and a methodology that does not yield results matching those expectation would raise justifiable doubts about its usefulness. Since the results of the methodology match what would reasonably be expected, that most, but not all, intersections can be converted, and some of those that may not be good candidates for conversion are a result of a negative PPA and others are a result of a potentially negative NBV (i.e. a slight variation in any of the input data would change a nominally positive NBV to a negative one), one can reasonably conclude that the methodology is sound.
CHAPTER 6 “CONCLUSIONS”

6.1 Objectives of this study

Transportation, the physical delivery of people and goods from one geographic point to another, is the lifeblood of a community, as it enables a community to interact, both culturally and economically, with the outside world; the greater the interaction, the more successful the community. One of the greatest empires the world has ever known, the Roman Empire, had an extensive transportation, both natural (rivers, lakes, and seas) and manmade which allowed for the efficient movement of people and goods from one end of the empire to the other. Julius Caesar famously built a bridge across the Rhine and marched his legions across as a demonstration to the Germanic tribes that not even the Rhine, a formidable natural barrier, can halt an army when a good transportation network is in place, similar to Xerxes’ display to the Greeks when he built a bridge across the Bosporus centuries before. Because a good transportation network is essential for the successful movement of people and goods from one geographic place to another, it is important that the transportation network operate as efficiently as possible.

Traffic signals are an important component in the maximization of efficiency in the transportation network. When used properly, traffic signals can maximize the efficiency of a transportation corridor. For example, Hall Road, between I-94 and the M-59 freeway, is a highly signalized corridor (more than 30 signals in less than 10 miles) in which the multiple signals are coordinated to allow vehicles to travel from one end of the corridor to the without stopping, while also providing sufficient gaps for vehicles on the intersecting streets to cross efficient, with these corridors having coordinated signals as
well. However, when used improperly, such as for intersections with low volumes or improperly coordinated, signals can, and do, impede the efficiency of the transportation network. Improperly used signals reduce the capacity of the network, resulting in diversions of vehicles onto other parts of the network and inefficient use of existing laneage and capacity. Improperly used signals also may contribute to an increase in crashes resulting from motorists running the signal to avoid having to wait or otherwise driving recklessly and carelessly. The purpose of this study, is to create a methodology to encourage the maintenance of properly used signals and the removal of improperly used ones.

Signal installation and removal is often highly subjective, based on political considerations and not engineering ones. This is problematic, as a methodology using subjective criteria may not be universally applicable, as the subjective criteria for one jurisdiction may not be the same as for another, resulting in a methodology of very limited value. To overcome this limitation, the methodology presented in this study uses objective criteria, namely monetary benefits and costs. While some of the constants used in calculating these monetary values are either local to southeast Michigan or are a generic default value, these constants can be changed to reflect local conditions; thus, because of this adaptability, this methodology is universal and can be used for any community.

6.2 Utility and applicability of the methodology

When creating an analysis methodology, it is important that the methodology have as much utility and applicability as possible to maximize its usefulness. In other words, a
methodology that is limited in its scope of application, i.e. can only be used in a limited number of situations, is less useful than one that has a broad application and can be used in many situations. This methodology was created to be the latter, and be useful at any intersection, regardless of local conditions. Each of the seventeen intersections analyzed were unique, and the methodology was successfully used at each one. Furthermore, the results of the methodology were unique for each intersection. This methodology, then, should be applicable at any intersection in any locale, which makes it useful for practitioners to use to perform such an analysis prior to spending of any financial resources. (While theoretically applicable at any intersection, realistically, only certain signalized intersections, those with lower speeds, low volume side streets, few heavy vehicles, random arrivals, and an adequate footprint to make modifications within the existing rights-of-way, are practical candidates for conversion from traditional signalized to alternative unsignalized geometrics and operation.)

6.3 Strengths of the methodology

A good planning methodology is one where the results can be duplicated, regardless of the analyst. In other words, it should be objective and not subjective. The problem with subjective analysis methodologies is the results are skewed by the biases and opinions of the analyst; thus, two different analysts may come up with completely different answers. A common example of this is a survey where the person filling out the survey is asked to grade a particular criterion; depending on the viewpoint of the person filling in the survey, the answer to the same scenario or question will vary. When analyzing capital improvements or major changes to the transportation infrastructure, many of the
factors that should be considered, such as community cohesion or aesthetics, are subjective and dependent on the worldview of the analyst. This is problematic, as results can be predetermined, and the analysis skewed to achieve this predetermined outcome. For example, say that the analyst is an advocate for bike lanes; he or she can weight or bias subjective analysis factors in such a way to achieve that result. The problem becomes that there will always be doubt about the validity of the results; if bike lanes are actually the best solution, how is one to know whether that is actually the case or if the results are due to the bias of the analyst?

The most notable strength of this methodology is that it is more objective than many other methodologies with the use of mathematically calculated values that are easily duplicated by many different analysts as a surrogate for subjective factors that are more dependent on the biases and worldview of the analyst. In other words, the personal opinions, preferences, and biases of an individual analyst should not yield different results. For example, the ability of pedestrians to cross the street is a key element of community cohesion; the more pedestrian-friendly a community is, the more cohesive it will be. The pedestrian accessibility analysis is an objective measure of community cohesion. Multiple analysts with various biases can perform the pedestrian accessibility analysis at a location and come up with the same answers; the bias and opinion of the analyst is irrelevant to the results. Whereas different analysts may have different subjective opinions on community cohesion resulting in different answers, by using a mathematically objective surrogate for community cohesion the results will be the same.

The mathematically objective surrogate for aesthetics is pervious area, as more
pervious area means more green space, and more green space lends itself to improved aesthetics. Aesthetics are quite subjective; what one person may find aesthetically pleasing another may find ugly. The potential for aesthetic improvement with the provision of green space is objective, the amount of space provided does not change from analyst to analyst. By using a mathematically objective surrogate, then, the methodology will deliver consistent results, regardless of the personal preferences of the analyst. When analyzing any capital improvement, the ability to remove subjectivity and opinion is important in convincing often skeptical citizens and decision makers to make the investment, i.e. the improvements are cost effective and a good use of taxpayer money.

Another notable strength of the methodology is its simplicity. The equations used in the calculations are straightforward and do not require intimate knowledge of calculus, differential equations, or other higher mathematics. Many traffic operations personnel are not engineers, and do not have the requisite knowledge of higher mathematics. Simpler equations make the methodology usable and available to those who do not have this higher mathematics knowledge, which makes it useful, for the most part, to all.

A final important strength is the adaptability of the methodology to reflect local conditions. Every locality has different values for material and construction cost, stormwater management, and the value of time. This methodology allows for the easy substitution of values for these items in place of the default values that better reflect the local conditions. For example, the value of time is not the same in wealthier locales than in poorer ones, and this can easily be adjusted within the methodology to accommodate for this. Same for construction costs, which vary greatly from locale to locale; these costs
can be changed to reflect local conditions. Thus, for two intersections with the exact same operational and geometric conditions in two different locations, the methodology will yield different results due to the local variances.

6.4 Weaknesses of the methodology

No methodology is perfect; every one has its strengths and weaknesses. This methodology is no exception. Thus, it is important to address the weaknesses to understand the limitations of the methodology. By understanding the weaknesses the analyst will better understand how to use it properly. In addition, knowing the weaknesses helps illuminate potential improvements to the methodology.

The first weakness pertains to the diversion of traffic from the minor street of the intersection after the construction of the alternate geometry. With the increased impedance created by the alternate geometry, traffic will be diverted from the minor street to other streets in the network. However, the California Diversion Equation (CDE) is a macroscopic equation, so its use in a microscopic situation may not be entirely appropriate, due to limitations in the network. The CDE works if there are viable alternate routes that can be used, which is the case in a macroscopic analysis. However, on the microscopic level, such viable alternate routes may not exist. Because of this uncertainty, two different scenarios, a worst case where no traffic is diverted, and a best case where a viable alternate route exists, must be calculated. This is problematic when the two scenarios yield different results, the best case yielding that the conversion should be done, and the worst case where it should not. In the case of conflict, the absolute value of the amplitude of the difference between the calculated value and zero for each scenario is
calculated, and the one with the largest amplitude controls. In most cases, such a conflict will not exist, but there will always be the exception, requiring further calculations.

Another weakness of the methodology pertains to the pedestrian accessibility analysis. The equation used is a general, default equation originally used to determine the viability of unsignalized school crossings. Because children tend to walk slower than adults, the equation yields conservative results. Thus, there is a chance that the pedestrian analysis shows a false failure, resulting in the wrong decision to be made, and a signalized intersection maintained where an unsignalized alternative geometry intersection can be used.

A final weakness of the methodology concerns the data requirements to perform the most accurate analysis and reliance on random assumptions in lieu of data. Among the input data required are signal phasing and timing data (for the net benefit value), hourly pedestrian volumes (for the pedestrian accessibility analysis), hourly turning movement volumes (for both the net benefit value and pedestrian accessibility analysis), and data pertaining to the quantities and movements of bicycles, heavy vehicles, and transit. Many jurisdictions either do not have this data, or do not have the ability to collect the data; in such cases, assumptions must be used. When using assumptions, the GIGO (garbage in-garbage out) principle applies, with the results only being as good as the assumptions used in the input to generate those results. Caution is recommended when using assumptions.

6.5 Opportunities for further study

There are numerous opportunities and topics that could be studied further. For
example, how well does this methodology work on other corridors? This study explored one corridor in one city in one state; a very limited sample size. How effective is it in other corridors in the same city? Other cities in the same state? Other cities in other states? In other countries? Do the various components work in other locations, or do they have to be replaced with other components? How does the outcome of the methodology compare to microsimulation, and can one or more benefits be derived from microsimulation instead of static equations?

Before and after studies of the conversions is a necessary study to confirm the effectiveness and validity of the methodology. Did each of the components accurately predict what would happen upon conversion? Was the pedestrian equation a good surrogate for the impact on walkability and neighborhood cohesion? Did the conversion impact other elements, such as speeds, aesthetics, delays, safety, and overall quality-of-life issues not only on the individual intersectional level but also on the overall corridor level as well? Did the conversions deliver as promised?

One interesting study would be to test and calibrate the methodology with existing signalized alternative intersections where before and after data is available. This calibration could then be used to enhance the methodology. A second interesting study would be to test the validity of the pedestrian crossing equations by collecting data and performing gap studies at various unsignalized pedestrian crossings, and making whatever changes are necessary to get a truer picture of pedestrian crossings. This study mentioned distribution patterns, specifically the randomness of distribution in an urban corridor, even one with a progression system. Much research needs to be done on this topic, as
randomness changes the effectiveness of any signal progression system.

One important issue that deserves more in-depth discussion is that of speed control strategies along an arterial. Can we control speeds through progression? What is the impact of reducing lane width a particular incremental amount on speed? Does the number of and location of (left, middle, right) lanes make a difference in speeds? Do curbs or parking lanes provide a better result? What are the impact of bike lanes on speed? Is the best speed control strategy a combination of both progression and design?

A final topic that should be explored further is the effect of impedance on traffic patterns. Specifically, how much diversion occurs when alternative intersection geometries are constructed and installed? A better understanding of diversion and distribution of traffic in a network will provide important information and insight into the validity of a transportation model, and will allow for the refinement of such a model. In addition, this better understanding can also be applied to improving access control policies, to ensure maximum benefit and minimum detriment, with respect to both traffic operations and land access, from such policies.
APPENDIX A “CORRIDOR PROGRESSION ANALYSIS”

A.1 Purpose of progression analysis

One of the warrants for signalization is maintenance of progression along a corridor. Good progression is only possible if all the signals operate on the same cycle length or even multiples thereof and signals are properly spaced. Signals that are too close are problematic for progression as the traffic stream often has not reached steady state flow due to the differences in acceleration ability of different vehicles. Signals that are too far apart are problematic due to the breakdown of the integrity of the progression platoon. Due to these deleterious effects on progression of improper signal spacing, it is imperative to analyze the impacts any proposed changes in signalization will have on progress prior to recommendation or implementation of these changes.

A.2 Existing conditions

A.2.1 Timing

As it is currently timed, Jefferson has no progression, and its signals cannot be progressed into a unified system. This is due to a lack of a uniform cycle length (or multiples thereof) throughout the entire corridor. Currently, the progression on Jefferson is limited to each of the five subcorridors (City Center, Belle Isle Access, Depopulated Zone, Conner Creek Industrial, Suburban Transition) independent of the other subcorridors as the cycle lengths for each are not the same. In effect, then, East Jefferson is actually five different and unique progression corridors; thus, it is improbable that a motorists can run the gauntlet of all of the traffic signals along the corridor without stopping.
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A.2.2 Spacing

The differing cycle length problem can be rectified by setting all the cycle lengths along the corridor to the same value (e.g. 80 seconds) or multiples thereof (e.g. 40 seconds, 160 seconds, etc.). However, different cycle lengths are not the only impediment to successful progression of the signals on the East Jefferson corridor. As can be seen from \textbf{TABLE A.1}, the spacing between adjacent signals along the corridor is not conducive to effective progression based on the spreadsheet by Hummer (2016) and the 2003 TRB \textit{Access Management Manual}, as it is not consistent and varies greatly. Per the Hummer spreadsheet based on the McShane & Roess (1990) formula for progression efficiency, there are five locations where adjacent signals can be successfully coordinated for simultaneous progression for an operating speed of 35 miles per hour (the posted speed limit), i.e. where adjacent signals are in phase with each other (e.g. turn green at the same time) – Baldwin/Seyburn, Van Dyke/Parker, Burns N/Burns S, Harding S/Harding N, and Dickerson S/Dickerson N - all of which are 350 feet or less (approximately one standard city block) apart. However, that is not the case along the East Jefferson corridor; not only are the signals not all spaced one block apart, they are not spaced in even multiples of blocks apart due to the presence of half and other partial blocks along the corridor.

A second type of progression is what is known as alternate progression, where every other signal is in phase with each other. Based on the Hummer (2016) spreadsheet and the 2003 \textit{Access Management Manual}, (TRB, 2003) there are only two viable pairs where alternate progression is reasonable for an operating speed of 35 miles per hour –
Rivard/St. Aubin and McDougall/Mt. Elliott. Note that these are the only two locations where the calculated progression speed is within ten percent of the desired operating speed.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DIST FROM PREVIOUS SIGNAL (ft)</th>
<th>PROGRESSION EFFICIENCY(1)</th>
<th>CAN USE SIMULTANEOUS PROGRESSION?(2)</th>
<th>CAN USE ALTERNATE PROGRESSION?(3)</th>
<th>CALCULATED PROGRESSION SPEED(4)</th>
</tr>
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<tr>
<td>Rivard St Aubin</td>
<td>1980</td>
<td>1%</td>
<td>NO</td>
<td>YES</td>
<td>33.81</td>
</tr>
<tr>
<td>Dubois</td>
<td>1000</td>
<td>25%</td>
<td>NO</td>
<td>NO</td>
<td>17.10</td>
</tr>
<tr>
<td>Chene</td>
<td>570</td>
<td>36%</td>
<td>NO</td>
<td>NO</td>
<td>9.77</td>
</tr>
<tr>
<td>Jos Campau</td>
<td>940</td>
<td>27%</td>
<td>NO</td>
<td>NO</td>
<td>16.08</td>
</tr>
<tr>
<td>McDougall/Walker</td>
<td>1050</td>
<td>24%</td>
<td>NO</td>
<td>NO</td>
<td>17.95</td>
</tr>
<tr>
<td>Mt Elliott</td>
<td>1950</td>
<td>2%</td>
<td>NO</td>
<td>YES</td>
<td>33.30</td>
</tr>
<tr>
<td>E Grand</td>
<td>2550</td>
<td>-13%</td>
<td>NO</td>
<td>NO</td>
<td>43.52</td>
</tr>
<tr>
<td>Baldwin</td>
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<td>15%</td>
<td>NO</td>
<td>NO</td>
<td>24.43</td>
</tr>
<tr>
<td>Seyburn</td>
<td>350</td>
<td>41%</td>
<td>YES</td>
<td>NO</td>
<td>6.02</td>
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<tr>
<td>Van Dyke</td>
<td>670</td>
<td>34%</td>
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<td>NO</td>
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</tr>
<tr>
<td>Parker</td>
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<td>43%</td>
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<td>NO</td>
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<tr>
<td>Burns (N)</td>
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<td>14%</td>
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<td>NO</td>
<td>24.77</td>
</tr>
<tr>
<td>Burns (S)</td>
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<td>46%</td>
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<td>Hibbard</td>
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<td>McClellan</td>
<td>970</td>
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<td>Cadillac</td>
<td>1090</td>
<td>23%</td>
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</tr>
<tr>
<td>Garland/Marquette</td>
<td>1050</td>
<td>24%</td>
<td>NO</td>
<td>NO</td>
<td>17.95</td>
</tr>
<tr>
<td>Harding (S)</td>
<td>590</td>
<td>36%</td>
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<tr>
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<td>48%</td>
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<tr>
<td>Lillibridge</td>
<td>1300</td>
<td>18%</td>
<td>NO</td>
<td>NO</td>
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</tr>
<tr>
<td>St Jean</td>
<td>700</td>
<td>33%</td>
<td>NO</td>
<td>NO</td>
<td>11.99</td>
</tr>
<tr>
<td>Conner</td>
<td>3390</td>
<td>-33%</td>
<td>NO</td>
<td>NO</td>
<td>57.84</td>
</tr>
<tr>
<td>Dickerson (S)</td>
<td>1410</td>
<td>15%</td>
<td>NO</td>
<td>NO</td>
<td>24.09</td>
</tr>
<tr>
<td>Dickerson (N)</td>
<td>230</td>
<td>44%</td>
<td>YES</td>
<td>NO</td>
<td>3.98</td>
</tr>
<tr>
<td>Coplin</td>
<td>950</td>
<td>27%</td>
<td>NO</td>
<td>NO</td>
<td>16.25</td>
</tr>
<tr>
<td>Chalmers</td>
<td>1450</td>
<td>14%</td>
<td>NO</td>
<td>NO</td>
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<tr>
<td>Alter</td>
<td>1580</td>
<td>11%</td>
<td>NO</td>
<td>NO</td>
<td>26.99</td>
</tr>
</tbody>
</table>

(1) efficiency = 0.5 - (L/(S*C)) from McShane & Roess.  S=51 fps (35 mph); C=80 sec
(2) Simultaneous can be used if efficiency>= 40% per Hummer
(3) Alternate can be used if -10%<=efficiency<=10%
(4) Calculated progression speed from Table 6-1 of 2003 TRB Access Management Manual

TABLE A.1. Spacing and progression possibilities of existing signalization.
speed. (It should also be noted that progression efficiencies of zero represent the optimum for alternate progression.)

A.2.3 Two-way progression

As stated previously, progression on the entire East Jefferson corridor is not possible as it currently is signalized and timed due to differing cycle length. For the sake of evaluation and comparison with the proposed signalization scheme, the cycle length for the entire corridor will be set at 80 seconds, the longest cycle length currently used among the signals to be removed. With an 80 second cycle length, the signal at Rivard becomes the critical signal, with the shortest amount of green time for Jefferson at 40 seconds. Thus, the maximum width of any possible progression band is 40 seconds.

Using an 80 second cycle length, the potential optimum two-way progression (i.e. maximum progression band in both the eastbound and westbound directions) is 13 seconds eastbound and 13 seconds westbound, representing approximately 33% of the maximum possible progression bandwidth of 40 seconds. A time-space diagram of this progression was plotted using Synchro and is shown in FIGURE A.1. As these progression bands represent less than 50 percent of the available maximum green time (e.g. the maximum possible), the existing corridor has overall poor progression. As stated previously, the inconsistent spacing of existing signals makes efficient progression, for all intents and purposes, if not impossible to achieve, highly unlikely.
FIGURE A.1. Two-way progression for existing signalization. An 80 s cycle length yields both eastbound and westbound bands of 13 s, approximately 33% of available green.
A.2.4 Eastbound predominate progression

Depending on the travel characteristics of the corridor, often, at particular time periods, traffic volumes are imbalanced between the two travel directions. For example, on a corridor connecting a business district with neighboring residential districts, volumes are much larger inbound versus outbound in the morning and vice versa in the evening. During these time periods, two way progression may not be as important as progression for the dominant direction. Thus, it is important to optimize progression for each of the travel directions, regardless of the impact on the other direction. Typically, optimized one-way progression bands are wider than those for optimized two-way progression. In addition, spacing of the signals is not as restricting for one-way progression as for two-way progression.

The optimized progression bands for the eastbound predominate condition for the existing signalization are shown in the time-space diagram of FIGURE A.2. As can be seen from FIGURE A.2, the potential maximum eastbound progression band is 38 seconds, representing 95 percent of the possible maximum potential eastbound progression band. In order to achieve this, westbound progression is sacrificed. In the case of the existing signalization, no possible westbound progression of any magnitude can be achieved, as the possible maximum westbound progression band is zero.

A.2.5 Westbound predominate progression

The maximum possible progression band width for the westbound predominate condition for the existing signalization is shown by the Synchro time-space diagram in
FIGURE A.2. Eastbound progression for existing signalization. An 80 s cycle length yields a 38 s band eastbound. However, no westbound band exists.
FIGURE A.3. Westbound progression for existing signalization. An 80 s cycle length yields a 39 s band westbound. However, no eastbound band exists.
FIGURE A.3. As can be seen from FIGURE A.3, the maximum possible progression band for the westbound predominate scenario for existing conditions is 39 seconds, approximately 98 percent of the maximum possible. As with the eastbound predominate scenario, this excellent possible progression can only be achieved by sacrificing progression in the opposite direction, which, like the previous predominate scenario, is not possible. In other words, to achieve optimized progression westbound, it is not possible to have any possible progression eastbound, as the maximum possible progression bandwidth is zero.

A.3 Proposed conditions

A.3.1 Timing

Under the proposed signalization scheme, all signals will have an 80 second cycle length. This cycle length was selected for several reasons. First, it is the cycle length used along the City Center subcorridor, the same subcorridor with the highest concentration of signals that will remain, thus alleviating any need for substantial changes along this subcorridor. Second, it is the same cycle length selected for the existing signalization analysis, allowing for easy comparison and contrast of the results of the existing and proposed signalization scenarios; not only is the cycle length the same, the splits are the same as well. Third, it provides ample walk time for pedestrians to cross while still providing adequate levels-of-service (D or better) for vehicular movements. (It should be noted that with the exception of the signals at East Grand and Conner, all proposed signals reflect a reduction of laneage along East Jefferson from three in each direction to two.)
A.3.2 Spacing

At three locations along the corridor, the spacing between adjacent signals will not change – McDougall/Walker to Mount Elliott, Mount Elliot to East Grand, and Saint Jean to Conner. At all other locations, the spacing between adjacent signals will increase with the implementation of the proposed modifications. **TABLE A.2** shows the signal spacing, progression efficiency, and calculated progression speed for the proposed corridor signalization configuration. Negative efficiency represents segments where the signal progression platoon breaks down due to distance; most of the segments experience this phenomenon. Because of this, most of the signals cannot be part of an effective progression system and will thus operate as de facto independent signals. However, under the proposed system, 6490 feet of the corridor, from Chene to East Grand, can be

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>DIST FROM PREVIOUS SIGNAL (ft)</th>
<th>PROGRESSION EFFICIENCY(^{(1)})</th>
<th>CAN USE SIMULTANEOUS PROGRESSION?(^{(2)})</th>
<th>CAN USE ALTERNATE PROGRESSION?(^{(3)})</th>
<th>CALCULATED PROGRESSION SPEED(^{(4)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rivard</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chene</td>
<td>3550</td>
<td>-37%</td>
<td>NO</td>
<td>NO</td>
<td>60.57</td>
</tr>
<tr>
<td>McDougall/Walker</td>
<td>1990</td>
<td>1%</td>
<td>NO</td>
<td>YES</td>
<td><strong>33.98</strong></td>
</tr>
<tr>
<td>Mt Elliott</td>
<td>1950</td>
<td>2%</td>
<td>NO</td>
<td>YES</td>
<td><strong>33.30</strong></td>
</tr>
<tr>
<td>E Grand</td>
<td>2550</td>
<td>-13%</td>
<td>NO</td>
<td>NO</td>
<td>43.52</td>
</tr>
<tr>
<td>St Jean</td>
<td>11120</td>
<td>-223%</td>
<td>NO</td>
<td>NO</td>
<td>189.60</td>
</tr>
<tr>
<td>Conner</td>
<td>3390</td>
<td>-33%</td>
<td>NO</td>
<td>NO</td>
<td>57.84</td>
</tr>
<tr>
<td>Alter</td>
<td>5620</td>
<td>-88%</td>
<td>NO</td>
<td>NO</td>
<td>95.85</td>
</tr>
</tbody>
</table>

\(^{(1)}\) efficiency = 0.5 - (L/(S*C)) from McShane & Roess. \(S=51\) fps (35 mph); \(C=80\) sec  
\(^{(2)}\) Simultaneous can be used if efficiency\(\geq\) 40% per Hummer  
\(^{(3)}\) Alternate can be used if \(-10%\leq\)efficiency\(\leq\)10%  
\(^{(4)}\) Calculated progression speed from Table 6-1 of 2003 TRB Access Management Manual

**TABLE A.2.** Spacing and progression possibilities of proposed signalization.
progressed efficiently, as the Chene, McDougall, and Mt. Elliott signals can be coordinated to provide progression at 35 miles per hour; with the existing system, only 4500 feet of the corridor, from McDougall to East Grand, can provide efficient progression. *(TABLE A.1)* Thus, with the proposed signalization, the distance that can be efficiently coordinated for 35 miles per hour is 44 percent more than with the existing signalization, approximately 20 percent of the entire corridor. Despite the fact that most of the signals within the proposed scheme will be de facto independent signals, it should nonetheless provide better progression than the existing system.

*A.3.3 Two-way progression*

As with the existing conditions scenario, the cycle length for the entire corridor will be set at 80 seconds, the longest cycle length currently used among the signals to be maintained. As stated previously, with an 80 second cycle length, the signal at Rivard becomes the critical signal, with the shortest amount of green time for Jefferson at 40 seconds; thus, the maximum width of any possible progression band is 40 seconds. Using an 80 second cycle length, the potential optimum two-way progression (i.e. maximum progression band in both the eastbound and westbound directions) is 13 seconds eastbound and 13 seconds westbound, representing approximately 33% of the maximum possible progression bandwidth of 40 seconds. A time-space diagram of this progression was plotted using Synchro and is shown in FIGURE A.4. These are the same bandwidths as with the existing conditions; thus, the proposed signalization scheme will not have a deleterious effect on progression in the corridor.
FIGURE A.4. Two-way progression after modifications. An 80 s cycle length yields both eastbound and westbound bands of 13 s, same as for the existing conditions.
A.3.4 Eastbound predominate progression

The optimized progression bands for the eastbound predominate condition for the proposed signalization are shown in the time-space diagram of FIGURE A.5. As can be seen from FIGURE A.5, the potential maximum eastbound progression band is 40 seconds, representing 100 percent of the possible maximum potential eastbound progression band. This potential maximum band is actually an improvement over what is possible for the existing conditions. As with the existing conditions, westbound progression is sacrificed in order to accomplish this, with a maximum possible westbound progression band of zero. Since eastbound maximum possible progression has been improved, and westbound possible maximum progression has not been degraded, it can reasonably be concluded that the proposed changes to signalization will not have a deleterious effect on eastbound predominate progression in the corridor.

A.3.5 Westbound predominate progression

The maximum possible progression band width for the westbound predominate condition for the proposed signalization is shown by the Synchro time-space diagram in FIGURE A.6. As can be seen from FIGURE A.6, the maximum possible progression band for the westbound predominate scenario for existing conditions is 38 seconds, approximately 95 percent of the maximum possible. This is slightly less (1 second or approximately 3 percent) than the possible maximum progression band for the corresponding existing conditions scenario. For all practical purposes, the magnitude of the decrease is small enough that, for all practical purposes, the westbound progression
FIGURE A.5. Eastbound progression after modifications. An 80 s cycle length yields a 40 s band eastbound, 100% of available green. However, no westbound band exists.
bandwidths for both the existing and proposed conditions are the same. Unlike with the either eastbound predominate scenario or the existing westbound predominate scenario, this excellent possible progression can be achieved without totally sacrificing progression in the opposite direction. Under the proposed signalization, a small eastbound possible maximum progression band of 7 seconds can be obtained when the westbound band is maximized. The proposed signalization improves the possible maximum progression obtainable for the westbound predominate scenario.

A.4 Summary of corridor progression analysis

For an 80 second cycle length, the proposed changes to signalization along the corridor will not have a negative impact on progression. For two-way progression, the possible maximum bands for both eastbound and westbound remain the same. For an eastbound predominate scheme, the maximum possible eastbound progression band increases (while the corresponding westbound band is non-existent for both existing and proposed conditions). There is a slight drop (1 second) in the westbound band for the proposed westbound predominate scenario, but this is more than offset by the creation of a 7 second eastbound band. (No eastbound band existed at all for the westbound predominate scenario for existing conditions). While these results are for only one possible cycle length, this is the cycle length currently used in the subcorridor where the most signals will be retained, so using this particular cycle length would minimize changes to signal timing for the signals to be retained in the proposed signalization scheme.
FIGURE A.6. Westbound progression after modifications. An 80 s cycle length yields a 38 s band, 1 s less than existing conditions. However, unlike the existing conditions, a small (7 s) eastbound band exists.
APPENDIX B “DETAILED DESCRIPTION OF INTERSECTIONS”

B.1 City Center subcorridor

B.1.1 Signalized Intersection 1: Rivard Street

Rivard Street is the first signal encountered as one travels east on Jefferson from the I-375 freeway spur, and is the western boundary of the corridor. (FIGURE B.1) This intersection is a four-legged intersection, and Rivard serves as a direct connection from Jefferson and its parallel streets, Larned and Lafayette, to the riverfront. Because of this connectivity, and the fact that it only one block from the I-375 freeway spur, this intersection is not a good candidate for conversion, as such a conversion would be

FIGURE B.1. Intersection 1, Rivard, and the westernmost part of the East Jefferson Corridor.
disruptive for accessing the riverfront and would impact the operations of the junction of Jefferson and the freeway spur, and diverted movements would be forced to travel through this junction. For consistency with the raised medians proposed for the rest of the corridor, raised medians would also be constructed on the approaches to this intersection; the intersection would thus remain operationally the same as it is currently.

**B.1.2 Unsignalized intersections: Riopelle Street, Orleans Street**

For the sake of consistency, the proposed alternative unsignalized geometric configuration will be continued through current unsignalized intersections as well. While many of the unsignalized intersections will be converted to alternative geometries as part of the conversions of the signalized intersections (“collateral unsignalized intersections”, or CUIs), there are also several unsignalized intersections along the corridor that will not be affected by the signalized conversions (“orphan unsignalized intersections”, or OUIs). To enhance consistency and uniformity along the corridor, the proposed alternative unsignalized geometric configuration will be continued through these OUIs as well. The first two unsignalized intersections impacted by the geometric changes as one travels east of Rivard on Jefferson are the intersections of Jefferson and Riopelle Street and Jefferson and Orleans Street, both of which are CUIs. **(FIGURE B.2)** Direct left turns from westbound Jefferson will be allowed at Riopelle; thus, this intersection will function as an RCUT, with only the minor street left turn movements being indirect. In contrast, the no direct left turns will be permitted to or from Orleans Street; all left turns will be indirect. Thus, unlike the intersection of Jefferson and Riopelle, the intersection of Jefferson and
Orleans will function as an ILAC.

B.1.3 Signalized Intersection 2: Saint Aubin Street (North leg)

Saint Aubin Street is a three-legged intersection approximately five blocks east of Rivard. There are technically two Saint Aubins: one servicing the residential area north of Jefferson (North leg), and one connecting Jefferson to the riverfront (South leg). Unlike Rivard, the two pieces of Saint Aubin are not continuous, with an offset at Jefferson, with only the Saint Aubin north of Jefferson signalized. Because Saint Aubin (North leg) is a minor collector street, it has low traffic volumes and no heavy vehicles, and thus this signal

![Diagram](image)

**FIGURE B.2.** Intersection 2, Saint Aubin (North leg), with proposed design and analysis output.
is an excellent candidate for removal. The proposed design for this intersection, an RCUT, and the results of the analysis for this intersection are shown in FIGURE B.2. As can be seen from the figure, under the “best case” scenario the volume of traffic on the diverted movement, southbound left, drops to zero (shown in italics) from 28 for the “worst case” and signalized scenarios (non-italicized) due to the additional 1,750 feet that a motorist would have to travel to complete the movement. (The movement shown by the solid line, 122’, is the distance a southbound left-turning vehicle has to travel under the existing signalized condition; the movement shown by the dashed line, 1872’, is the distance a southbound left-turning vehicle has to travel with the proposed operational configuration. The difference between the two is 1750’.) Given that southbound motorists can turn left at Larned, which runs parallel to Jefferson and eventually intersects it via Mount Elliott, a volume of zero is intuitive, as a motorist wanting to go east can complete the movement easily and directly at Larned rather than travel an additional 1,750 feet to complete the movement at Jefferson.

Two important design concepts need to be noted regarding the location of the median crossover servicing the southbound left turns at this location. First, it is located at an unsignalized intersection, giving motorists the option of either proceeding straight or making a U-turn. All proposed median crossovers presented in this study located at unsignalized intersections that provide the ability to proceed straight also provide the ability to make a U-turn; thus they are slightly offset from the street they provide direct access to accommodate these maneuvers. The U-turn crossover for the southbound
movement from Saint Aubin (North leg) to eastbound Jefferson is slightly to the west of Riopelle; a motorist in this crossover can either complete the U-turn onto eastbound Jefferson or veer slightly to proceed south on Riopelle. This offset improves safety by lowering speeds of those vehicles completing either of these maneuvers, as the intersection for these movements behaves like a roundabout with curvature to the vehicle paths for these movements. Second, the distance from Saint Aubin (North leg) to the crossover at Riopelle exceeds the recommended optimal crossover spacing of 660 feet (Reid, Sutherland, Ray, Daleiden, Jenior & Knudsen, 2014). In an existing, built-up urban corridor, the optimal spacing may not be the most cost effective or practical due to restrictions such as structures, driveways, etc.; thus, spacing other than optimal should be expected. Optimal spacing should be used when practical and when the use of the optimal spacing does not have a deleterious effect on cost or the existing local environment.

The first step of the analysis, the pedestrian accessibility analysis (PAA), indicates that unsignalized operation can work for widths of 30 feet or less. The proposed design is for two travel lanes of 10 foot each and one parking lane of 8 foot for each direction; therefore, at this location conversion to unsignalized will not impede the ability of pedestrians to cross the street, and the net benefit value (NBV) analysis should be completed. The NBV analysis was completed for both the “worst case” scenario, where the traffic demand on the impeded movements is assumed to be the same as prior to the installation of the impedances, and the “best case” scenario, where it is assumed that the
impedances discourage motorists from making the affected movements, who then find alternate routes in the network. Under both scenarios, the NBV is greater than one; thus, this conversion as designed should be undertaken.

### B.1.4 Unsignalized intersection: Saint Aubin Street (South leg)

The intersection of Jefferson and Saint Aubin Street (South leg) is an OUI that will be converted to an ILAC-type operation to maintain uniformity along the corridor, as there will be no direct left turns at this intersection. **(FIGURE B.2)** Saint Aubin (South leg) is one of numerous streets connecting Jefferson to the Detroit River waterfront. Because it is a low volume local street, it is devoid of commercial traffic. Thus, eliminating left turns at this intersection will have a minimal impact on traffic operations along the corridor or in the adjacent neighborhood.

### B.1.5 Signalized Intersection 3: DuBois Street

DuBois Street is a minor three-legged intersection approximately 1000 feet east of the signal at Saint Aubin and 500 feet west of the signal at Chene. DuBois runs south for a couple of blocks from Jefferson to the riverfront area, serving as a very low volume secondary access to this area. It is a narrow street which is not used by larger commercial vehicles due to its small size and the presence of on-street parking. Because of these characteristics, this signal is an excellent candidate for replacement with an alternate unsignalized intersection. Like Signalized Intersection 2, the proposed design for this intersection is an RCUT, and the results of the analysis for this intersection are shown in **FIGURE B.3.** As can be seen from the figure, under the “best case” scenario the volume
of traffic on the diverted movement, northbound left, drops to zero due to the additional 1,161 feet that a motorist would have to travel to complete the movement. (Note that the values for the “worst-case” and existing volume and travel distance conditions are in regular type, the “best case” in italics, the existing travel path shown with a solid red line, and the proposed travel path shown with a broken blue line. This nomenclature is used

<table>
<thead>
<tr>
<th></th>
<th>&quot;Worst case&quot;</th>
<th>&quot;Best case&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_{SD}$</td>
<td>19,390</td>
<td>25,546</td>
</tr>
<tr>
<td>$B_{PM&amp;O}$</td>
<td>1,000</td>
<td>1,000</td>
</tr>
<tr>
<td>$B_{SR}$</td>
<td>6,482</td>
<td>6,482</td>
</tr>
<tr>
<td>$B_{S}$</td>
<td>188,203</td>
<td>212,446</td>
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<tr>
<td>$B_{TT}$</td>
<td>(10,380)</td>
<td>884</td>
</tr>
<tr>
<td>$\Sigma B$</td>
<td>204,695</td>
<td>246,358</td>
</tr>
<tr>
<td>B-C</td>
<td>$162,195$</td>
<td>$203,858$</td>
</tr>
</tbody>
</table>

FIGURE B.3. Intersections 3 & 4, Dubois & Chene, with proposed design and analysis output. (NOTE: Since Intersection 4 is not to be changed, the output pertains only to Intersection 3.)
in all of the design figures shown in Appendix B.) Given the close proximity of Dubois to Chene, a more prominent parallel street that provides connections at not only Jefferson but also Larned and Lafayette (for northbound motorists wanting to turn west towards downtown Detroit) and a direct connection to the neighborhoods north of Jefferson, a complete diversion of the left-turning movement upon the installation of an impedance is logical, due to the superiority of this and other alternate routes within the network. Furthermore, there is a direct connection from the riverfront area to downtown via Atwater Street, rendering the need to turn left onto Jefferson to access downtown moot.

The PAA for Signalized Intersection 3 indicates that unsignalized operation can work for widths of 36 feet or less, and since the proposed design is for two travel lanes of 10 foot each and one parking lane of 8 foot for each direction, at this location conversion to unsignalized will not impede the ability of pedestrians to cross the street, and the NBV analysis should be completed. As with Signalized Intersection 2, the NBV for both the “worst case” and “best case” is greater than one; thus, this conversion as designed should be undertaken.

B.1.6 Signalized Intersection 4: Chene Street

Chene Street is a major collector that provides a direct connection from the neighborhoods on the north side of Jefferson to the riverfront on the south side. North of Jefferson, Chene is a four lane divided facility that intersects Larned and Lafayette; south of Jefferson, Chene is a wide collector and serves as a primary access to the riverfront. Because of Chene’s functionality, it is busier than many of the intersections
along the corridor. Furthermore, it is a four-legged intersection, which is not as conducive to conversion to an alternative unsignalized geometric configuration. Thus Signalized Intersection 4, like Signalized Intersection 1, will be maintained as a signalized intersection with raised medians. The proposed geometric configuration for Signalized Intersection 4 is shown in FIGURE B.3.

B.1.7 Signalized Intersection 5: Joseph Campau Street

Joseph Campau Street is a minor collector street that connects the riverfront with Jefferson, with its northern terminus at Jefferson, and is located midway between the signals at Chene and McDougall Streets, approximately 1000 feet from each. Joseph Campau provides access to a burgeoning entertainment district featuring microbreweries, restaurants, and live entertainment. Because on-street parking is at a premium in this district, many patrons will park on the north side of Jefferson, the side opposite the district, thus necessitating crossing Jefferson. The pedestrians counted crossing Jefferson at this intersection, 150 per hour, are by far the highest number counted at any signalized intersection along the corridor. The observed pedestrian characteristics, namely crossing Jefferson to access the entertainment establishments in the Joseph Campau district, reinforce the validity of using pedestrian accessibility as a surrogate for community cohesion.

As stated previously, the PAA should be performed before the NBV analysis. Signalized Intersection 5 is an illustrative example why. The NBV indicates that this intersection should be converted to the unsignalized RCUT as shown in FIGURE B.4.
However, the PAA indicates that pedestrian accessibility is a problem, as there is no width for which the number of pedestrians crossing at this location can be serviced adequately. In other words, the PAA indicates that a pedestrian-activated signal is needed. Adding such a signal will reduce all but the travel time and runoff benefits and will increase the cost; it is possible that these detriments will cause the NBV to be less than zero. Doing the analysis in reverse order can yield an erroneous NBV, and necessitates an unnecessary expenditure of time, labor, and money in correcting this error. Because pedestrians cannot be accommodated properly without some type of signalization, then this intersection should not be converted to an alternative unsignalized one.

![Figure B.4. Intersection 5, Joseph Campau, with proposed design and analysis output.](image-url)
B.1.8 Unsignalized intersection: McDougall Street (South leg)

The intersection of Jefferson and McDougall Street (South leg) is a CUI that will be converted to an ILAC-type operation in conjunction with the modification to Signalized Intersection 5. **(FIGURE B.4)** Even though, as stated previously, Signalized Intersection 5 will require pedestrian activated signals, that type of signal can be used with the proposed RCUT design. If a more traditional design is used, then the intersection of Jefferson and McDougall (South leg) becomes an OUI, and to maintain uniformity along the corridor, the proposed ILAC operational configuration will be maintained. McDougall (South leg) is one of numerous streets connecting Jefferson to the Detroit River waterfront; thus, it is a low volume local street, and devoid of commercial traffic. Thus, eliminating left turns at this intersection will have a minimal impact on traffic operations along the corridor or in the adjacent neighborhood.

B.1.9 Signalized Intersections 6 & 7: McDougall (North leg)/Walker Street and Mount Elliott Street

Signalized Intersections 6 and 7 are both four-legged intersections with higher classification streets that provide a direct connection from the neighborhoods north of Jefferson to the commercial and riverfront districts on the south side of Jefferson. **(FIGURE B.5)** The easternmost of the two is McDougall (North leg)/Walker, a four legged intersection with the leg on the north side of Jefferson being McDougall and that on the south side being Walker. Like Signalized Intersection 4, the north leg of Signalized Intersection 6 (McDougall) is four lane divided with connections to Larned and Lafayette,
while the south leg is a wide collector that serves as a primary access to the waterfront.

Because of the higher number of large commercial vehicles, and the high number of left
turns at the intersection, Intersection 6 is not a good candidate for conversion to an
alternative unsignalized one, and was not analyzed further.

Signalized Intersection 7 is similar to both Signalized Intersections 4 and 6 in that
it is a four-legged intersection whose north approach is multilane divided and south
approach is a wide collector. Mount Elliott is the southern part of a major interregional
transportation corridor that stretches from the Detroit River to The Thumb via Mound, 18
½ Mile, and M-53. Because of its importance, it is heavily trafficked and has frequent
commercial vehicles. Additionally, as it is the eastern terminus of Larned, it has a very

FIGURE B.5. Intersections 6 & 7, McDougall (North leg)/Walker & Mount Elliott, and
the unsignalized intersections between them.
heavy left-turning movement onto eastbound Jefferson as the Larned traffic continues eastward. Thus, Signalized Intersection 7 is not a good candidate for conversion to an alternative unsignalized one and was dropped from further consideration.

**B.1.10 Unsignalized intersections: Adair Street, Leib Street**

Between Signalized Intersections 6 and 7 are two OUIs: Adair Street and Leib Street. (FIGURE B.5) Although Signalized Intersections 6 and 7 are not proposed to be converted to unsignalized operation, for the sake of geometric consistency along the Jefferson corridor, the alternative geometric configurations will be continued between these intersections through the intersections of Jefferson and Adair Street and Jefferson and Leib Street. Since Adair Street is a small local street, the former intersection will be converted to ILAC operation, as there are a dearth of commercial vehicles wanting to turn left on to or off of Adair, and thus an ILAC operational configuration will have a minimal impact on the overall traffic operations on Jefferson. In contrast to Adair, Leib is a divided minor collector servicing the same neighborhood on the south side of Jefferson between it and the Detroit River; because of its superiority, it is more likely to be used by commercial vehicles servicing this area. Because of this, the intersection of Jefferson and Leib will be converted to an RCUT to allow direct left turns from Jefferson onto Leib. As this intersection is between Signalized Intersections 6 and 7, traffic wanting to turn left off of Leib onto Jefferson can utilize one of these two signals; thus, the impact to traffic flow and accessibility is negligible.
B.1.11 Unsignalized intersections: Iron Street, Meldrum Street (North leg), Meldrum Street (South leg), Jefferson Court, Beaufait Street, Bellevue Street, Concord Street

Between Signalized Intersections 7 and 8 are seven OUIs: Iron Street, Meldrum Street (North leg), Meldrum Street (South leg), Jefferson Court, Beaufait Street, Bellevue Street and Concord Street. Although Signalized Intersections 7 and 8 are not proposed to be converted to unsignalized operation, for the sake of geometric consistency along the Jefferson corridor, the alternative geometric configurations will be continued between these intersections through the six aforementioned OUIs. The first two of intersections, Iron and Meldrum (North leg), will be converted to ILAC operation. (FIGURE B.6) Both of these intersecting streets are low volume local streets with few, if any, commercial vehicles. Because of these existing operational and traffic stream characteristics, conversion to ILAC operation will have little, if any impact on, degradation to, existing traffic operations, characteristics, and patterns.

The intersection of Jefferson and Meldrum Street (South leg) will be converted to an RCUT operation, with direct left turns from Jefferson permitted at the intersection. (FIGURE B.6) Although this intersection could function as an ILAC, it is beneficial to provide periodic median openings to minimize the degradation of access to adjacent properties and neighborhoods, and it is logical to provide these openings at unsignalized intersections to minimize disruption to traffic operations and access. When determining if a particular unsignalized intersection is suited to such a treatment, the location of adjacent driveways and streets is important, as it would be dangerous to locate a median
opening at an intersection where through movements perpendicular to the major street movement is possible. For example, if a driveway is located across from a local street, a driver would have the opportunity to go straight across multiple lanes of traffic into the local street via the median opening, creating numerous potential conflicts with motorists on the through street, and degrading both traffic operations and overall safety. At the intersection of Jefferson and Meldrum (South), no such potential conflict condition exists, as there are no driveways or other entrances on the north side of Jefferson across from Meldrum (South).

Characteristically, Jefferson Court and Beaufait Street are similar to Iron Street and
Meldrum Street (North) - low volume local streets with few, if any, commercial vehicles. Because of these existing operational and traffic stream characteristics, conversion to ILAC operation will have little, if any impact on, degradation to, existing traffic operations, characteristics, and patterns, and thus these two intersections are ideal candidates for conversion to unsignalized ILACs. (FIGURE B.6) As they are located between the RCUTs at Meldrum Street (South) and Bellevue Street, these two ILACs will utilize the direct left turns at these latter RCUT locations for the indirect left and through movements. These crossovers will be utilized by traffic from multiple intersections, making them more cost efficient than unique and separate crossovers for each intersection. By increasing their utilization, their economic justification is improved. For decision makers with budget constraints, enhanced economic justification of an improvement is often a key consideration when making the necessary fiscal decisions to embark on the construction of the project.

Bellevue Street is unique among the streets running perpendicular to Jefferson Street, and it is because of this uniqueness that although the intersection of Bellevue and Jefferson could be an ILAC, it should be an RCUT. (FIGURE B.6) Operationally, Vernor Highway is an extension of the Fisher Freeway, which was constructed along the route of Vernor Highway through the central part of Detroit; in effect, then, the Fisher Freeway replaced and superseded Vernor Highway. At Gratiot Avenue, Vernor Highway and the Fisher Freeway operate in tandem as a continuous east-west corridor, with through traffic passing from Vernor Highway to the Fisher Freeway and vice versa. Bellevue Street is the
easternmost street that has direct access to this corridor, for it is at Bellevue that Vernor transforms from a two-way street to a one-way eastbound street. Traffic on Bellevue can directly access the Fisher Freeway via westbound Vernor; all streets east of Bellevue cannot as westbound traffic is prohibited on Vernor at those locations. For traffic coming the properties and streets on the south side of Jefferson between Mount Elliott and Bellevue, it is potentially easier and quicker to access the Fisher/Vernor corridor by going eastbound on Jefferson and then turning left onto Bellevue and following Bellevue to Vernor than to attempt to turn left across seven lanes of traffic onto westbound Jefferson and then turn right onto Mount Elliott. To help facilitate this movement, a direct left is appropriate and necessary at Bellevue.

In addition to aiding accessibility to the Fisher/Vernor corridor, there are other reasons why a direct left turn onto Bellevue is logical. First, Bellevue is approximately 1000 feet, or three blocks east of Mount Elliott, the last intersection where the direct left for eastbound traffic is allowed. Locating the direct turns further apart would not only degrade the benefit of the conversion to unsignalized alternative intersections, due to increased travel time, the extra inconvenience would make the change more difficult for policy makers to justify to their constituents. Without the support of the decision makers, the probability of the changes being implemented is diminished. Second, as there are no driveways on the south side of Jefferson at this location, the safe operation of this median crossing will not be compromised by these influences. For the reasons enumerated, an RCUT operational configuration is appropriate for this intersection. Since this intersection
is a three legged intersection, only one movement, left turns from Bellevue to Jefferson, will be affected by the conversion to an RCUT. Because Vernor becomes one-way eastbound at Bellevue, and there are numerous, and better connections, between Vernor and Jefferson east of Bellevue, Bellevue going towards Jefferson is neither significant nor unique like Bellevue going away from Jefferson; thus, this particular movement is of low importance. In other words, an RCUT at this intersection is only impacting the least important movement.

Characteristically, Concord Street is similar to Jefferson Court, Beaufait Street, Iron Street and Meldrum Street (North) – a low volume local street with few, if any, commercial vehicles. Because of these existing operational and traffic stream characteristics, conversion to ILAC operation will have little, if any impact on, degradation to, existing traffic operations, characteristics, and patterns, and thus this two intersections is an ideal candidates for conversion to unsignalized ILACs. **(FIGURE B.7)** Unlike the proposed modifications for previously enumerated OUIs, some of the proposed modifications for the Concord Street intersection, specifically a median crossover U-turn facility for left turning traffic from Jefferson, already exists, and will be utilized, thus reducing costs and improving the monetary efficiency of the ILAC at this location. To accommodate the left turning traffic from Concord, a new median crossover U-turn facility is proposed for a location approximately one-third of the distance between Concord and Bellevue in accordance with the preferred location guidelines shown in **FIGURE 5.1.**
B.2 Belle Isle Access subcorridor

B.2.1 Unsignalized intersections: Canton Street, Helen Street

In addition to the seven OUIs between Signalized Intersections 7 and 8 are two existing unsignalized intersections that will not be changed – Canton Street and Helen Street. (FIGURE B.7) Canton Street current operates as an ILAC, with the left turns from Canton completing the indirect U-turning movement between Canton and Concord and the left turns from Jefferson utilizing a median crossover at Helen. The intersection of Jefferson and Helen currently functions as an RCUT, as eastbound vehicles utilizing the

FIGURE B.7. Intersection 8, East Grand Boulevard, and adjacent intersections.
median crossover at Helen have the opportunity to proceed straight onto Helen; vehicles turning left from Helen utilize the same U-turn crossing point between Canton and Concord used by the left-turning traffic from Canton. Because of its close proximity to the exit from East Jefferson to Belle Isle Park, this current U-turn location is potentially hazardous; in order to enhance safety, the indirect left turning movement from Canton and Helen will be moved to the west away from the park entrance by sharing the median crossover U-turn between Concord and Bellevue with the Concord left turning traffic. It should be noted that the existing opposing crossovers at Helen Street are not at least 100 feet apart as recommended by the Michigan Department of Transportation (Reid, Sutherland, Ray, Daleiden, Jenior & Knudsen, 2014). These crossovers predate the formalization of these standards and are not heavily used (thus the probability of vehicles using both crossovers simultaneously and causing a sight distance obstruction is very low); thus, they will be maintained as-is. Wherever practical (based on intersection and driveway location restrictions), new crossovers adhere to this recommendation.

B.2.2 Signalized Intersection 8: East Grand Boulevard

Signalized Intersections 4, 6, 7, and 8 are all four-legged intersections with higher classification streets that provide a direct connection from the neighborhoods north of Jefferson to the commercial and riverfront districts on the south side of Jefferson. Signalized Intersection 8, East Grand, like Signalized Intersections 4, 6, and 7, is four legged intersection whose north approach is a multilane divided street. (FIGURE B.7) Unlike the other three, the south approach of Intersection 8 is also a multilane-divided street, as it is
the Belle Isle Causeway that provides access to Belle Isle, a major park. East Grand Boulevard is the first of two streets that formed the outer boundary of the urbanized area, and functions as a loop around the core of the urbanized area. Because it is the entrance to Belle Isle, East Grand has heavy turning movements at Jefferson, on the north end of the causeway; these movements are heavy enough that this intersection is a signalized MUT. Because of these conditions, this intersection was not considered for signal removal.

**B.2.3 Unsignalized intersections: Field Street, Sheridan Street**

Between Signalized Intersection 8, East Grand Boulevard, and the eastern terminus of the Belle Isle Access subcorridor are two unsignalized intersections: Field Street and Sheridan Street. The Field Street intersection currently functions as an ILAC, and will continue to function as such with one modification – the median crossover for eastbound Jefferson traffic turning left onto Field will be moved to the west to Sheridan Street. *(FIGURE B.8)* There are two advantages to this modification. First, it moves this maneuver away from the merge point of traffic from Belle Isle onto Jefferson eastbound and through traffic on eastbound Jefferson, thus enhancing safety. Second, it provides for the more efficient use of infrastructure, as a median crossover at Sheridan can be shared by traffic wanting to access Field and traffic wanting to access Sheridan from eastbound Jefferson, reducing the per vehicle cost of the infrastructure by increasing utilization.

Because of the safety benefit of relocating the median crossover to access Field, and the aesthetic and safety benefit and maintaining a consistent overall geometry along
the corridor, the OUI intersection at Sheridan will be converted to an unsignalized partial RCUT. (FIGURE B.8) Specifically, the direct left turn from Jefferson onto Sheridan would be created; however, a convenient median crossover for traffic turning left from Sheridan on Jefferson would not. Although it would be operationally possible to construct such a crossover between Sheridan and Field, as there are no driveways to interfere with placement of such a crossover in this location, it would not be fiscally prudent to make

![Intersections 9 & 10, Baldwin and Seyburn Streets, with proposed design and analysis output.](image-url)
such a capital expenditure to service the handful of vehicles wanting to make this maneuver.

**B.3 Depopulated Zone subcorridor**

**B.3.1 Unsignalized intersection: Townsend Street**

The third unsignalized intersection between Signalized Intersections 8 (East Grand) and 9 (Baldwin), and the first within the Depopulated Zone subcorridor as one travels from west to east, is the intersection of Townsend Street. This intersection is a CUI, i.e. an unsignalized intersection that will be modified as part of the modification of a nearby signalized intersection, in this case, Signalized Intersection 9, Baldwin Street. As a result of the modifications to Signalized Intersection 9, the unsignalized intersection at Townsend will be converted to an ILAC operational regime. *(FIGURE B.8)*

**B.3.2 Signalized Intersections 9 & 10: Baldwin Street, Seyburn Street**

Signalized Intersections 9 and 10 are one block apart and service the same complex, a medical facility on the north side of Jefferson. Because of this commonality, ideally these two intersections should be analyzed in tandem; the proposed geometrics reflect this tandem concept. *(FIGURE B.8)* For Signalized Intersection 9, a reversed RCUT (RRCUT), a derivative of the MUT where side street traffic can turn left but main street left-turning traffic must proceed through the intersection and complete the movement via a downstream median crossover, is proposed. Since Seyburn and Baldwin service the same facility, the two streets in combination can function as a quasi-couplet, with Seyburn servicing the inbound traffic (while still permitting outbound) and Baldwin the outbound
traffic (while still permitted inbound). To emphasize this operational concept, Signalized Intersection 10, Seyburn, is a proposed RCUT. The result, then, is the RRCUT at Signalized Intersection 9 will encourage the outbound traffic from the complex to use Baldwin, and the RCUT at Signalized Intersection 10 will encourage the inbound traffic to use Seyburn.

Since both the PAA and NBV models are intended to be used on independent intersections, then both Signalized Intersections 9 and 10 were analyzed assuming that any conversion of one will be made independent of the other, as such an analysis allows an exploration of the value of each type of geometry. Furthermore, as the cost of constructing both will be more expensive than constructing only one, analyzing each independently gives decision makers more flexibility and options in converting these intersections, as they could choose to do one without the other due to budgetary constraints.

Signalized Intersection 9 is technically a four-legged intersection, with the south leg a stub accessing the adjacent properties. Subsequent to the initial data collection, the south leg has been blocked and is not used. Although there are traffic counts to and from this leg, that data was collected prior to the abandonment of this approach; this leg now has no traffic. Functionally, Signalized Intersection 9 is a three-legged intersection. Since this change occurred while this study was ongoing, the original traffic counts were used in the analysis. Because Baldwin services a medical facility, larger vehicles use it, and thus it is necessary to provide easy access out to Jefferson. This is the rationale for using an RRCUT, to expedite movement out of the facility. Regardless of whether or not Signalized
Intersection 10 is modified, vehicles can use Seyburn as access to the facility; thus the impact on the ability to enter the facility is, for the most part, moot.

The PAA of both intersections reveals that neither proposed alternate unsignalized operation will impact the ability of pedestrians to cross the street as long as the crossing distance is 30 feet or less. As stated previously, the maximum proposed distance from curb to median is 28 feet, which is less than the maximum calculated for the pedestrian and vehicular volumes by EQ 4.32, TABLE 5.1, and FIGURE 5.2. The NBV analysis for both intersections indicates a positive net benefit with conversion, for both the “worst case” and “best case” scenarios. It should be noted that there are some possible safety issues with the RRCUT when compared with a similar RCUT, namely that the RRCUT has one additional conflict point for each left turning movement. With an RCUT, the only conflict point is where the left turning vehicles have to cross the opposing main street through traffic. With an RRCUT, the left turning vehicles are in conflict with both directions of main street through traffic. More conflict points means more potential for crashes; thus caution and discretion is advised and encouraged when using the RRCUT.

B.3.3 Signalized Intersections 11 & 12: Van Dyke Street, Parker Street

Signalized Intersection 11, Van Dyke Street, and Signalized Intersection 12, Parker Street, are three legged intersections with the minor street approaching from the north. (FIGURE B.9) As both intersections have private driveways where a south approach would be, they both technically could be considered four-legged; however, private driveways, while taken into consideration when determining the geometric configuration, were not
taken into consideration when conducting the operational analysis. The rationale for this dichotomy is that while the location of a driveway will not change during the design life, twenty years, its operation will change, depending on the land use. Thus, it is logical to include the driveways in the geometric considerations but not the operational ones.

FIGURE B.9. Intersections 11 & 12, Van Dyke and Parker Streets, with proposed design and analysis output.

Like Signalized Intersections 9 and 10, Signalized Intersections 11 and 12 are one block apart and service the same area on the north side of Jefferson. Because of this commonality, ideally these two intersections should be analyzed in tandem; the proposed geometrics reflect this tandem concept.  (FIGURE B.9) For Signalized Intersection 11, an RCUt is proposed, and an ILAC is proposed for Signalized Intersection 12.  Although Van
Dyke eventually ends at Port Austin in The Thumb as M-53, most inbound traffic on Van Dyke turns on I-94 or Gratiot, with that portion between Gratiot and Jefferson primarily being used for local traffic; thus, volumes are low enough at the Jefferson terminus to warrant removal of the existing signal and conversion to an alternative unsignalized configuration. Parker is a local street that runs approximately two blocks to Lafayette; due to this status, there is little traffic and few turns onto Parker from Jefferson. Thus, an ILAC is an appropriate configuration.

As with Signalized Intersections 9 and 10, both Signalized Intersections 11 and 12 were analyzed assuming that any conversion of one will be made independent of the other. The PAA of both intersections reveals that neither proposed alternate unsignalized operation will impact the ability of pedestrians to cross the street as long as the crossing distance is 24 feet or less at Van Dyke and 36 or less at Parker. As can be seen from FIGURE B.9, the proposed design shows the pavement narrowing to 24 feet in the vicinity of the downstream median U-turn crossing; if pedestrian crossings are restricted to this area, then the proposed configuration will satisfactorily satisfy pedestrian crossing demand without the need for pedestrian-activated signalization. Forcing pedestrians to cross downstream away from the intersection proper is the strategy used for accommodating pedestrians at roundabouts; thus, it is not unreasonable to apply the same strategy for a different type of alternate intersection geometry such as an RCUT. Although both intersections have the same number of pedestrians crossing, they have different main street vehicular volumes, which is why the crossing distance for Van Dyke is different than
that for Parker. Since the maximum volumes used in the PAA are the volumes downstream of the intersection, consisting of main street through, side street right, and diverted movements, and the minor street and diverted movements at Van Dyke are much greater than those at Parker, the maximum volume for Signalized Intersection 11 will be higher than that for Signalized Intersection 12, resulting in a shorter crossing distance.

B.3.4 Unsignalized intersections: Seminole Street, Iroquois Street

Between Signalized Intersection 12, Parker Street, and Signalized Intersection 13, Burns Street, are two unsignalized intersections – Seminole and Iroquois Streets. The westernmost of these, the intersection of Jefferson and Seminole, is an OUI that is proposed as an RCUT geometric configuration. (FIGURE B.10) Since an RCUT is operationally preferable to an ILAC because no major street movements are affected, an RCUT should be installed when modifying an OUI where practical; Seminole is one of those locations. The adjacent intersection to the east, Iroquois, is a CUI that is proposed to have an ILAC-type configuration as part of the modifications to Signalized Intersection 13, Burns Street.

B.3.5 Signalized Intersection 13: Burns Street

Signalized Intersection 13, Burns Street, is a four-legged offset intersection, with the minor street, Burns, the offset one. (FIGURE B.11) From the perspective of the main street, Jefferson, this is a positive offset intersection, as the left turns do not conflict. The current signal operation scheme is a modified simple two-phase one; while both legs of Burns proceed simultaneously, left turns are held on Jefferson between the two legs of
FIGURE B.10. OUI and CUI east of Parker.

Burns at the far offset, and then proceed with Jefferson traffic. While each of the two legs operates as a simple two-phase, together they operate as a three-phase: major street left-through-right, minor street through-right, and a minor street lagging protected left turn.

Because of its positive offset geometric configuration, Signalized Intersection 13 is an ideal candidate for conversion to a dual, or full, RCUT configuration. With a full RCUT,
all main street traffic in both directions is permitted to make a direct turn at the intersection, with only side street left and through movements being forced to make the

**FIGURE B.11.** Intersection 13, Burns Street, with proposed design and analysis output.
indirect movements. The PAA of this intersection reveals that the proposed alternate
unsignalized operation will not impact the ability of pedestrians to cross the street as long
as the crossing distance is 24 feet or less. As stated previously, the maximum proposed
distance from curb to median is 28 feet, consisting of two 10 feet wide driving lanes and
one 8 feet wide parking lane. Eliminating parking within the offset (between the two legs
of Burns) would enhance the safety of the intersection by improving the sight distance for
vehicles on Burns; it would also reduce the crossing distance to 20 feet and satisfy the
pedestrian accessibility requirement. The NBV analysis for this intersection indicates a
positive net benefit with conversion for both the “worst case” and “best case” scenarios.

B.3.6 Unsignalized intersections: Fischer Street, Crane Street

Fischer Street is a CUI that will be converted to ILAC operation as part of the
conversion of Signalized Intersection 13, Burns. (FIGURE B.11) Left turns onto Fischer will
be accommodated with the median U-turn crossing used by northbound lefts and
throughs from Burns, while left turns off of Fischer are accommodated using the left-turn
onto southbound Burns, as there is sufficient room for vehicles to complete a U-turn at
this location. Crane Street is an OUI that will also be converted to ILAC operation. (FIGURE
B.12) Due to its location between Signalized Intersections 13 and 14, movements at the
Crane intersection will utilize the geometric improvements constructed for these two
intersections. Left turning traffic coming from Crane will be accommodated by using the
median left turn crossing at Burns as a median U-turn crossing, while left turning traffic
onto Crane will be accommodated via the median U-turn crossing for Signalized
Intersection 14, Hibbard.

B.3.7 Signalized Intersection 14: Hibbard Street

Signalized Intersection 14, Hibbard Street, like Signalized Intersection 13, is a four-legged offset intersection where only one of the cross-streets is offset. Like at Signalized Intersection 13, the minor street at Signalized Intersection 14, Hibbard, is the offset one. 

(Figure B.12) Unlike Signalized Intersection 13, from the perspective of the main street, Jefferson, Signalized Intersection 14 is a negative offset intersection, as the left turns do conflict. The current signal operation scheme is a simple two-phase one, where both legs of Hibbard proceed simultaneously, as the left turns from Hibbard onto Jefferson do not conflict. Unlike Signalized Intersection 13, which has a large enough offset that vehicles can be safely stored between the two legs of the offset, the offset at Signalized Intersection 14 is too short to safely accommodate vehicle storage; thus, this intersection is geometrically one intersection, albeit a wide one, and operates as such.

Because of its negative offset geometric configuration, Signalized Intersection 14, unlike Signalized Intersection 13, is not an ideal candidate for conversion to a dual, or full, RCUT configuration, as the median left-turn lanes would overlap and intersect. As both legs of Hibbard and left turns onto are low volume with less than one vehicle per minute approaching the intersection, an ILAC is the appropriate remedy for the existing geometric constraints and can be constructed within the existing footprint. As can be seen from Figure B.12, when volume adjustments are applied to Signalized Intersection 14 resulting in the “best case scenario”, the maximum volume for any movement diverted by the ILAC
configuration is one vehicle every fifteen minutes. Given the redundant connectivity of the street network in this location, i.e. there are many different alternate paths available to a motorist resulting from the dense street grid, the “best case scenario” is reasonable.

FIGURE B.12. Intersection 14, Hibbard Street, with proposed design and analysis output.
The PAA of this intersection reveals that the proposed alternate unsignalized operation will not impact the ability of pedestrians to cross the street as long as the crossing distance is 22 feet or less. Due to the relatively close proximity of the western U-turn crossing, it would be reasonable to use the approaches to the crossing where the pavement narrows to 20 feet to accommodate pedestrians. The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios.

**B.3.8 Unsignalized intersections: Holcomb Street, Fiske Drive, Belvidere Street, Lodge Drive**

Holcomb Street is a CUI that will be converted to ILAC operation as part of the conversion of Signalized Intersection 14, Hibbard. *(FIGURE B.12)* Left turns onto Holcomb will be accommodated with the median U-turn crossing used by northbound lefts and throughs from Hibbard, while left turns off of Holcomb are accommodated using the median U-turn crossing for southbound Hibbard traffic. Fiske Drive is an OUI that will also be converted to ILAC operation. *(FIGURE B.12)* Due to its location between Signalized Intersections 14 and 15, movements at the Fiske intersection will utilize the geometric improvements constructed for these two intersections. Left turning traffic coming from Fiske will be accommodated by using the median left turn crossing at Signalized Intersection 15, McClellan, *(FIGURE B.13)* as a median U-turn crossing, while left turning traffic onto Fiske will be accommodated via the western median U-turn crossing for Signalized Intersection 14, Hibbard. *(FIGURE B.12)* Belvidere and Lodge are both CUIs
that will be converted to ILACs as part of the conversion of Signalized Intersection 15. **(FIGURE B.13)** Left turns from Belvidere and onto Lodge will be accommodated using the median U-turn crossing for Signalized Intersection 15, while left turns onto Belvidere and from Lodge will be accommodated using the direct left turn at McClellan.

**B.3.9 Signalized Intersection 15: McClellan Avenue**

Signalized Intersection 15, McClellan Avenue, is a traditional three-legged intersection. **(FIGURE B.13)** McClellan, the intercepted street, is a major collector that connects Jefferson to the parallel Mack and Warren corridors and Gratiot Avenue, also known as M-3, a trunk route that connects Detroit with communities to the northeast, terminating in Port Huron. Despite its connectivity, McClellan is a narrow street with on-street parking that passes through a residential area, and is not used as a commercial corridor. Because of the low number of trucks on McClellan, and the three-legged geometry of Signalized Intersection 15, an RCUT is the best alternative intersection design for this location, as only one movement, left-turns from McClellan onto eastbound Jefferson, would be redirected; left turns from eastbound Jefferson onto McClellan would continue to be a direct movement, as well as right turns from both southbound McClellan onto westbound Jefferson and westbound Jefferson onto northbound McClellan.

As can be seen from **FIGURE B.13**, when volume adjustments are applied to the Signalized Intersection 15 resulting in the “best case scenario”, the maximum volume for the one movement diverted by the RCUT configuration, left turns from McClellan onto eastbound Jefferson, is one vehicle per hour. Given the redundant connectivity of the
FIGURE B.13. Intersection 15, McClellan Street, with proposed design and analysis output.

street network in this location, i.e. there are many different alternate paths available to a motorist resulting from the dense street grid, the “best case scenario” is reasonable.
Due to the combination of relatively high vehicular and pedestrian volumes, the PAA of this intersection reveals that the proposed alternate unsignalized operation will not impact the ability of pedestrians to cross the street as long as the crossing distance is 20 feet or less, the minimum required to use an alternative geometric configuration at this location. Due to the lack of close proximity of either U-turn crossing, it would not be reasonable to use the approaches to either of these crossings where the pavement narrows to 20 feet to accommodate pedestrians. In order to provide for pedestrians, parking should be removed on both sides of Jefferson between McClellan and Lodge, allowing a reduction in pavement width to 20 feet, thus satisfying the requirement for pedestrian accommodation. An additional benefit of removing parking in this location is improving sight distance for vehicles entering the traffic streams on Jefferson from McClellan and Lodge. The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios.

**B.3.10 Unsignalized intersections: Parkview Drive, Parkview Street, Pennsylvania Street**

Parkview Drive (FIGURE B.13) is an OUI that will be converted to ILAC operation to maintain geometric and operational consistency along East Jefferson coinciding with the conversions of Signalized Intersections 15 and 16. Left turns onto Parkview Drive will be accommodated with the median U-turn crossing used by left turning traffic from McClellan, while left turns off of Parkview Drive will be accommodated with a median crossing at Parkview Street. Parkview Street (FIGURE B.14) is also an OUI that will also be converted to ILAC operation, utilizing the same median crossings as Parkview Drive.
Pennsylvania Street is a CUI that will be converted to ILAC operation utilizing the RCUT geometric infrastructure for Signalized Intersection 16, Cadillac Boulevard, with left turns from Pennsylvania accommodated using the median U-turn crossing for the left turns from Cadillac Boulevard, and left turns onto Pennsylvania accommodated using the direct left turn at Cadillac. (FIGURE B.14)

**FIGURE B.14.** Intersection 16, Cadillac Boulevard, with proposed design and analysis output.
B.3.11 Signalized Intersection 16: Cadillac Boulevard

Signalized Intersection 16, Cadillac Boulevard, like Signalized Intersection 15, is a traditional three-legged intersection. (FIGURE B.14) Cadillac, the intercepted street, is a minor collector that connects Jefferson to the parallel Mack, Warren, and Harper corridors, with one block of Cadillac serving as part of the Warren-Forest couplet. Like McClellan, Cadillac is a narrow street with on-street parking that passes through a residential area, and is not used as a commercial corridor. Because of the low number of trucks on McClellan, and the three-legged geometry of Signalized Intersection 16, an RCUT is the best alternative intersection design for this location, as only one movement, left - turns from Cadillac onto eastbound Jefferson, would be redirected; left turns from eastbound Jefferson onto Cadillac would continue to be a direct movement, as well as right turns from both southbound Cadillac onto westbound Jefferson and westbound Jefferson onto northbound Cadillac.

As can be seen from FIGURE B.14, when volume adjustments are applied to the Intersection 16 resulting in the “best case” scenario (the italicized left turn value, whereas the non-italicized value represents the signalized and “worst case” scenarios), the maximum volume for the one movement diverted by the RCUT configuration, left turns from Cadillac onto eastbound Jefferson, is, for all intents and purposes, zero. This is due to the introduction of an impedance or resistance to this particular desired movement and the presence of other routes with less impedance to accommodate this desired travel path; in other words, motorists are following the path of least resistance. Given the
redundant connectivity of the street network in this location, i.e. there are many different alternate paths available to a motorist resulting from the dense street grid, and the existing low volumes on Cadillac approaching Signalized Intersection 16, the “best case scenario” is reasonable. (It should be noted that, due to entropy, there will always be one or two random cars, even when the predicted volume is zero. However, because of the nature of entropy itself one has no way of predicting when these events will occur. In all likelihood, at any given moment in time, zero is the appropriate value.) Because of this diversion to alternate routes, the modifications necessary to convert this intersection to an alternative unsignalized geometry do not have to be as robust and expensive. Mathematical models are predictions, and entropy impacts the accuracy of the predictions; thus, caution should be used when using any predicted values as design parameters. Prudence dictates a more conservative design philosophy.

Due to the combination of relatively high vehicular and pedestrian volumes, the PAA of this intersection reveals that the proposed alternate unsignalized operation will not impact the ability of pedestrians to cross the street as long as the crossing distance is 30 feet or less, the minimum required to use an alternative geometric configuration at this location. Note that while the vehicular volumes are greater at Signalized Intersection 16 (1766 vph) compared to Signalized Intersection 15 (1696 vph), the allowable crossing is also greater (30 ft versus 20 ft), which, at first blush, seems counterintuitive. Recall that there are three unique variables expressed in EQ 4.32, tabulated in TABLE 5.1, and graphically presented in FIGURE 5.2 – pedestrian volume, vehicular volume, and
maximum crossing distance. If either of the volume variables are held constant, then an inverse relationship will exist between the other volume variable and the maximum crossing distance, e.g. an increase in volume will cause a decrease in maximum crossing distance, and vice versa. This is not the case with Signalized Intersections 15 and 16, as both the volume variables change, with the increase in vehicles at Signalized Intersection 16 more than offset by a decrease in pedestrians, the decrease allowing for a longer maximum crossing distance. Because the proposed design calls for a pavement width less than the maximum crossing distance, pedestrians could cross anywhere. That being said, placing the crosswalk east of the intersection will enhance safety for pedestrians by reducing the number of potential vehicle-pedestrian conflicts caused by turning vehicles, as pedestrians would not be in conflict with either vehicles turning left onto Cadillac or vehicles turning from Cadillac. The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios.

B.3.12 Unsignalized intersections: Hurlbut Street, Waterworks Park, Bewick Street

Hurlbut Street (FIGURE B.14) is an OUI that will be converted to RCUT-type operation to maintain geometric and operational consistency along East Jefferson coinciding with the conversions of Signalized Intersections 16 and 17. Left turns onto Hurlbut will be accommodated with a direct left turn median crossing at the intersection, located on the eastern extremity of the intersection to allow not only left turns into Hurlbut, but U-turns as well. Left turns from Hurlbut onto eastbound Jefferson will be accommodated via the median U-turn crossing between Pennsylvania and Parkview used
by left turning traffic from Cadillac onto Jefferson eastbound.

The intersection of East Jefferson and Waterworks Park is a CUI that will also be converted to RCUT-type operation. (FIGURE B.15) While Waterworks Park is gated and not open to the public at large, it is used by vehicles, including maintenance vehicles, accessing the waterworks. Thus, a direct left turn from Jefferson is desirable. In addition to providing access to the waterworks, this median crossover will also accommodate the left turns from the CUI at Bewick Street to eastbound Jefferson, as well as the left turns from Garland Street, the north leg of Signalized Intersection 17, to eastbound Jefferson; in short, three different movements will be sharing it, maximizing cost efficiency.

Bewick Street is a CUI that will be converted to ILAC operation in conjunction with the conversion of Signalized Intersection 17. (FIGURE B.15) As stated previously, left turns from Bewick to eastbound Jefferson will use the direct left-turn crossing at Waterworks Park. Left turns from eastbound Jefferson to Bewick will use a similar direct left-turn crossing at Garland Street.

B.3.13 Signalized Intersection 17: Garland Street/Marquette Drive

Like Signalized Intersection 13, Burns Street, Signalized Intersection 17 is a four-legged offset intersection where the minor street approaches, Garland Street on the north side of Jefferson and Marquette Drive on the south side of Jefferson, are offset. (FIGURE B.15) Like Signalized Intersection 13, from the perspective of the main street, Jefferson, Signalized Intersection 17 is a positive offset intersection, as the left turns do not conflict. Because of its geometric similarities to Signalized Intersection 13, it functions the same as
that intersection as well.

Like Signalized Intersection 13, Signalized Intersection 17 is an ideal candidate for conversion to a dual, or full, RCUT configuration due to its positive offset geometric configuration. The PAA of this intersection reveals that the proposed alternate unsignalized operation will not impact the ability of pedestrians to cross the street as long as

**PEDESTRIAN ACCESSIBILITY**

\[
V = 1699 \\
P = 4 \\
V_C > V \text{ for } D \leq 33 \text{ ft}
\]

**INTERSECTION 17 DATA**

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**FIGURE B.15.** Intersection 17, Garland Street/Marquette Drive, with proposed design and analysis output.
as the crossing distance is 33 feet or less. Since the maximum proposed distance from curb to median is 28 feet, the ability of pedestrians to cross the street will not be adversely impacted with the conversion of Signalized Intersection 17 to an unsignalized dual RCUT configuration. The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios.

**B.3.14 Unsignalized intersection: Saint Clair Street**

The intersection of East Jefferson and Saint Clair Street is a CUI that will be converted to RCUT operation as part of the conversion of Signalized Intersection 17 to an alternate unsignalized one. (FIGURE B.15) As shown in the figure, the left-turn and through movements from Marquette will utilize a median crossing at Saint Clair Street; this same crossing, then provides direct left-turn access from eastbound Jefferson onto northbound Saint Clair. Left turns from westbound Jefferson onto southbound Saint Clair, as well as the through and left-turn movements from the north leg of Saint Clair, will utilize the direct left-turn installed at Marquette as part of the conversion of Signalized Intersection 17. (FIGURE B.15) Northbound through and left-turning traffic on Saint Clair will complete those movements using a direct left-turn median crossing at Montclair Street installed as part of the conversion of Signalized Intersection 18, Harding Street. (FIGURE B.16)

**B.3.15 Signalized Intersection 18: Harding Street**

Like Signalized Intersection 14, Hibbard Street, Signalized Intersection 18, Harding Street, is a four-legged offset intersection where the minor street approaches are
negatively offset. (FIGURE B.16) Thus, like Signalized Intersection 14, Signalized Intersection 18 is not a viable candidate for conversion to a dual, or full, RCUT configuration. In a perfect urban grid, the network is equally balanced and distributed on

![PEDESTRIAN ACCESSIBILITY](image)

**INTERSECTION 18 DATA**

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FIGURE B.16. Intersection 18, Harding Street, with proposed design and analysis output.
both sides of a corridor, giving equal opportunity for access on either side. Along such a corridor in a perfect grid system, a series of alternating direct left-turn median crossings would maximize efficiency and minimize impact on access to and from the adjacent neighborhoods, as direct access points to a given neighborhood would be no further than two blocks apart and no motorist on any minor street would have to travel more than two additional blocks to complete any movement. Along most of the Jefferson corridor, the perfect urban grid system does not exist, especially on the south side of the corridor, where there are fewer streets than on the north side. In addition, many of the few that do exist do not interconnect, presenting additional geometric and operational constraints. A notable exception is the short section between Marquette and Montclair, where such a balanced grid exists. Through this section, the ideal configuration of alternating direct left-turn median crossings (partial RCUTs) should be used. Because Signalized Intersection 18, Harding falls within this short section, it, too, should be converted to a partial RCUT, to maintain consistency through this section (FIGURE B.16), as opposed to an ILAC configuration as specified for Signalized Intersection 14, Hibbard. At Marquette, the direct-left is from westbound Jefferson to southbound Marquette. The next intersection in the series, Saint Clair, has the direct-left from eastbound Jefferson to northbound Saint Clair. Since Harding follows Saint Clair, the direct left at this location is the same as at Marquette, from westbound Jefferson to southbound Harding. (FIGURE B.16)

The PAA of this intersection reveals that the proposed alternate unsignalized operation will not impact the ability of pedestrians to cross the street as long as the
crossing distance is 33 feet or less. Since the maximum proposed distance from curb to median is 28 feet, the ability of pedestrians to cross the street will not be adversely impacted with the conversion of Signalized Intersection 18 to an unsignalized dual RCUT configuration. The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios.

**B.3.16 Unsignalized intersections: Meadowbrook Street, Montclair Street, Lemay Street, Fairview Street**

There are four CUIs between Signalized Intersections 18 and 19 – Meadowbrook, Montclair, Lemay, and Fairview Streets. As mentioned previously, a balanced grid exists along the short stretch of Jefferson between Marquette and Montclair – for the most part. There is one exception to this balance – Meadowbrook Street. *(FIGURE B.16)* Meadowbrook exists approximately midway between Marquette and Montclair only on the south side of Jefferson, as the distance between Marquette and Montclair is slightly longer on the south side of Jefferson than on the north side due to intersection offsets. Because the CUI of Meadowbrook and Jefferson is a three-legged intersection, and Meadowbrook is a short, low volume street that is part of a grid network, an ILAC is the appropriate operational stratagem for this intersection, as few vehicles are impacted and access to adjacent properties is maintained via the grid.

Montclair Street is a positive offset intersection. It is unique in that it is a CUI for two different signalized intersections: Signalized Intersections 18 and 19. Due to its location and the nature of its offset, one leg is integral to the modifications to one
signalized intersection and the other leg is integral to the modifications to the other signalized intersection. As can be seen from **FIGURE B.16**, the offset unsignalized intersection of Montclair has a dual, or full, RCUT operational scheme, with the direct left onto the north leg part of the modifications to Signalized Intersection 18, Harding Street, and the direct left onto the south leg part of the modifications to Signalized Intersection 19, Lillibridge Street.

Between Montclair and Saint Jean Streets, the urban grid exists only on the north side of Jefferson, with Signalized Intersection 19, Lillibridge, the only intersection in this segment of Jefferson having four legs; all the others are three-legged. The first of these three-legged intersections is Lemay Street. Characteristically, Lemay Street is like Meadowbrook Street. It is a low-volume local street that is part of a developed grid network that ends at Jefferson in a three-legged unsignalized intersection, with the difference being Lemay is on the north side of Jefferson instead of the south. (**FIGURE B.17**) Because of these strong similarities, the proposed operational stratagem for Meadowbrook, an ILAC, is also applicable to Lemay, and should be utilized at this location.

Fairview Street is one of the several low-volume local streets that comprise the dense urban grid on the north side of this section of Jefferson. Because of its location approximately two blocks east of the direct left turn onto Montclair Street northbound, and because between that left turn and Fairview Street is a direct left-turn in the opposite direction onto southbound Montclair Street, it is logical to extend the alternating direct left turn pattern established at Marquette Drive to Fairview Street. Due to its geometrics,
i.e. a three-legged intersection to the north of Jefferson, and its location with respect to the locations of the alternating direct left turns, the CUI at Fairview and Jefferson is an ideal candidate for an RCUT operational scheme, with a direct left turn from Jefferson onto northbound Fairview. (FIGURE B.17)

![Intersection 19, Lillibridge Street, with proposed design and analysis output.](image-url)
**B.3.17 Signalized Intersection 19: Lillibridge Street**

Signalized Intersection 19, Lillibridge Street, is the only four-legged intersection on Jefferson between Montclair and Saint Jean Streets. *(FIGURE B.17)* The northern leg of Lillibridge is a narrow local urban street, the midway of the dense grid network of six streets that exists on the north side of Jefferson between Montclair and Saint Jean Streets. The southern leg provides access to the residential, commercial, and industrial properties between Jefferson and the waterfront, and is the only access point to these properties between Montclair and Saint Jean Streets. Thus, between the two legs of Lillibridge, maintenance of access to the southern leg is more critical than to the northern one due to the lack of a dense urban grid network on the south side of Jefferson.

Due to geometric constraints, a full RCUT is not feasible at Signalized Intersection 19; thus either a partial RCUT or an ILAC is required. As stated previously, the south leg of Lillibridge is the only street on the south side of Jefferson between Montclair and Saint Jean Streets; due to this factor, a partial RCUT configuration with direct access from westbound Jefferson to southbound Lillibridge is the appropriate unsignalized alternative geometric and operational configuration for this intersection. *(FIGURE B.17)* Furthermore, such a direct left turn movement would continue the alternating direct left turn configuration along the Jefferson corridor west of Signalized Intersection 17, Garland/Marquette, as it would be the companion to the eastbound-to-northbound direct left turn at Fairview Street.

The PAA of Signalized Intersection 19 reveals that the proposed alternate
unsignalized operation will not impact the ability of pedestrians to cross the street as long as the crossing distance is 30 feet or less. Since the maximum proposed distance from curb to median is 28 feet, the ability of pedestrians to cross the street will not be adversely impacted with the conversion of Signalized Intersection 19 to an unsignalized partial RCUT configuration. The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios, although it should be noted that the positive net benefit for the worst case scenario is marginal, at best; a difference of a few additional vehicles, in all likelihood, would change the net benefit to negative for this scenario. For the sake of discussion, if that were to be the case, should the conversion be made?

If the worst case scenario resulted in a negative net benefit, but the best case scenario resulted in a positive net benefit, it could reasonably be argued that either maintenance of the status quo or conversion is the appropriate response. Subjectively, the analyst could use the existing network and traffic characteristics as a basis for determining which scenario is more likely. In the case of Signalized Intersection 19, since Lilibridge is part of a grid network, it can be reasonably concluded that the best case scenario is more likely, as alternate routes are readily available, and based on that scenario, the conversion should be made. The problem with this subjective approach is that the results are open to debate, as different analysts may draw different conclusions. A better, and more objective approach, is a second derivation of the NBV method, by comparing the absolute values of each net benefit. If the absolute value of the net benefit
of the best case scenario is greater than the absolute value of the net benefit of the worst case scenario, then the conversion should be made; if not, the status quo should be maintained. If the net benefit of the worst case scenario for Signalized Intersection 19 had been negative and the net benefit of the best case scenario had been positive, the absolute values would be compared, with the highest value determining whether to convert or maintain the status quo. Since the net benefits for both the worst case and best case scenarios for Signalized Intersection 19 are both positive, this is a moot point.

**B.3.18 Unsignalized intersections: Beniteau Street, Defer Place**

Between Signalized Intersection 19, Lillibridge Street, and Signalized Intersection 20, Saint Jean Street, are two CUIs – Beniteau Street and Defer Place. (FIGURE B.17) Both of these streets are part of the dense urban grid network on the north side of Jefferson between Montclair and Saint Jean Streets. Both streets are narrow, local, low volume streets that service the neighborhood north of Jefferson and terminate at Jefferson. Because of these characteristics, and their proximity to Signalized Intersection 20 where the status quo will be maintained, both of these intersections are recommended to be converted to ILAC operation.

**B.4 Conner Creek Industrial subcorridor**

**B.4.1 Signalized Intersection 20: Saint Jean Street**

Signalized Intersection 20, Saint Jean Street, is a four legged intersection that operationally will be maintained as is. (FIGURE B.18) There are several important considerations to support maintaining the status quo at this location. First, Saint Jean is a
multilane facility; typically, multiple lanes on the minor street means higher traffic volumes on the minor street, a condition that is not conducive to the successful conversion to an alternative unsignalized intersection. Second, the adjacent land uses along Saint Jean are commercial and industrial, notably a major automobile assembly facility. Because of this, there is a high volume of large commercial vehicles that use Saint Jean, many of which are left turns onto or from Jefferson. Redirecting these movements with an alternative geometric design would greatly impede the ability to use these properties, especially the automobile assembly facility, to their fullest potential, jeopardizing their long-term use and viability. Third, Saint Jean is a major transit corridor

FIGURE B.18. Intersection 20, Saint Jean Street, and adjacent streets.
providing service to the automobile assembly facility and other adjacent industrial and commercial properties, most notably federal and state social services centers. Redirecting turns at this intersection will impact the ability of transit to service these properties and of the transit dependent to access these important employment and social services centers. Finally, Saint Jean is a minor arterial that provides direct access from the Conner Street/I-94 interchange to the waterfront. Because this intersection is the intersection of two arterials, it satisfies the network warrant as stated in the MUTCD. For this and the other aforementioned reasons, Signalized Intersection 20 should be maintained as such. It should be noted that, for the sake of geometric consistency, the raised-median cross-section established for the corridor west of Saint Jean will be maintained through this intersection. (FIGURE B.18) In addition to the obvious safety benefit of such a strategy, i.e. maintaining consistency and thus lessening confusion and the potential for crashes for motorists, there is also a direct quantifiable economic benefit in the reduction of runoff by reducing the amount of impervious surface.

B.4.2 Unsignalized intersections: Glover Street, Ly caste Street, Hart Street, Terminal Street

Between Signalized Intersection 20, Saint Jean Street, and Signalized Intersection 21, Conner Street, are four OUIDs – Glover Street, Hart Street, Lycaste Street, and Terminal Street. (FIGURE 5.20) As stated previously, for the sake of geometric consistency and safety, and because of the net economic benefit from reduced runoff that grass medians provide, the geometric cross-section established for the corridor west of Signalized
Intersection 20, Saint Jean, will be continued east of Saint Jean through these intersections as well. Unlike Saint Jean, these intersections will be reconfigured with alternative geometries as they are local low-volume streets. Glover Street is an approximately one-half block long dead end street that services one property, a small commercial business; because of this, the most restrictive geometry, the ILAC, is appropriate, as there are very few vehicles that will be impacted. In effect, since Glover functions as a driveway, the right-in/right-out restriction of an ILAC is not unduly burdensome, as right-in/right-out is a common commercial driveway treatment, especially in Metro Detroit, where raised medians through commercial areas are common.

The intersection of Lycaste Street can be classified as either a three-legged or a four-legged intersection, as one of the legs is a private driveway accessing a federal border patrol facility on the south side of Jefferson. The actual public street is a one-way northbound away from Jefferson, and is the primary entrance to the previously mentioned automobile fabrication facility. This intersection used to be signalized, but has subsequently converted to unsignalized operation; thus, it can be reasonably concluded that the traffic entering the intersection from the private driveway on the south side is minimal at best, and obviously not enough to warrant the maintenance of a traffic signal. Since the north leg is the primary entrance to a major industrial facility, then direct access to this leg from both directions of Jefferson needs to be maintained. Thus, a partial RCUT operational configuration with a direct left turn from eastbound Jefferson to northbound
Lycaste is the appropriate alternative geometric configuration for this location. (FIGURE B.18) Since it is a direct left-turn, such a configuration will not impede the ability of commercial vehicles to access the automobile fabrication facility, therefore will not degrade the usefulness of this industrial complex.

Hart Street is a dead-end local street on the south side of Jefferson that provides access to the federal border patrol facility that is also access by the private driveway on the south side of the Lycaste intersection. Because there is a need for commercial vehicles to service this and adjacent properties, then it is appropriate to provide direct access to Hart from both directions on Jefferson. An RCUT-type operational scheme, then, is the correct one to apply at this intersection, with a direct left-turn from westbound Jefferson onto Hart. (FIGURE B.18) Commercial vehicles leaving Hart can access the regional arterial and freeway network via Signalized Intersection 21, Conner Street by turning right onto eastbound Jefferson.

Terminal Street, both north and south of Jefferson, is a dead end street that provides access to industrial property. In the case of the north leg of Terminal, it provides delivery and service access to the automobile fabrication facility. The two legs of Terminal have a slight positive offset; thus a full RCUT is geometrically and operationally possible. Because both the north and south legs provide access to industrial properties, and both are dead ends, it is important to maintain direct access to both sides from both directions on Jefferson. Since a full RCUT is possible, and access from Jefferson is critical, the appropriate alternative configuration for this location is a full RCUT type of one, with
direct left turns from both directions of Jefferson to allow commercial vehicles to easily access the properties. (FIGURE B.19) For commercial vehicles leaving Terminal Street, a right turn onto Jefferson from either direction will provide a connection to the Conner Street/Interstate 94 interchange and the regional arterial network via Saint Jean Street for commercial vehicles exiting the north leg of Terminal and via Conner Street for commercial vehicles exiting from the south leg of Terminal.

B.4.3 Signalized Intersection 21: Conner Street

Signalized Intersection 21, Conner Street, like Signalized Intersection 20 is a four legged intersection that operationally will be maintained as is. (FIGURE B.19) Like at Signalized Intersection 20, there are several important considerations to support maintaining the status quo at this location. First, Conner is a multilane facility; typically, multiple lanes on the minor street means higher traffic volumes on the minor street, a condition that is not conducive to the successful conversion to an alternative unsignalized

FIGURE B.19. Intersection 21, Conner Street, and adjacent streets.
intersection. Second, the adjacent land uses along Conner are commercial and industrial, notably a major automobile assembly facility. Because of this, there is a high volume of large commercial vehicles that use Conner, many of which are left turns onto or from Jefferson. Making these movements indirect with an alternative geometric design would greatly impede the ability to use these properties, especially the automobile assembly facility, to their fullest potential, jeopardizing their long-term use and viability. Third, Conner is a major transit corridor providing service to the automobile assembly facility and other adjacent industrial and commercial properties, most notably a college campus. Restricting direct turns at this intersection will impact the ability of transit to service these properties and of the transit dependent to access these important employment and social centers. Finally, Conner is a major arterial that provides direct access from the Coleman Young Airport and the adjacent industrial area to the Detroit River waterfront via the Conner Street/I-94 interchange. Because Signalized Intersection 21 is the intersection of two arterials, it satisfies the network warrant as stated in the MUTCD. For this and the other aforementioned reasons, Signalized Intersection 21 should be maintained as such. It should be noted that, for the sake of geometric consistency, the raised-median cross-section established for the corridor west of Conner will be maintained through this intersection. (FIGURE B.19) In addition to the obvious safety benefit of such a strategy, i.e. maintaining consistency and thus lessening confusion and the potential for crashes for motorists, there is also a direct quantifiable economic benefit in the reduction of runoff by reducing the amount of impervious surface.
B.5 Suburban Transition subcorridor

B.5.1 Unsignalized intersections: Navahoe Street, Algonquin Street, Kitchener Street, Continental Street, Gray Street

Between Signalized Intersection 21, Conner Street, and Signalized Intersection 22, Dickerson Avenue, are five OUIs – Navahoe Street, Algonquin Street, Kitchener Street, Continental Street, and Gray Street. Since Signalized Intersection 21 is a four legged intersection that operationally will be maintained as is but will have raised medians consistent with the rest of the corridor for safety, aesthetic, and economic benefits, these five unsignalized intersections will, also. Navahoe Street is a low-volume local street that runs south of Jefferson. The neighborhood that Navahoe services can also be easily accessed by Conner Street via Signalized Intersection 21 and other parallel streets, so

FIGURE B.20. East Jefferson corridor from Conner Street to Gray Street.
direct left-turn access is not critical. Furthermore, providing direct-left turn access will interfere with the proper operation of Signalized Intersection 21 due to its close proximity to the intersection. Thus, the appropriate operational treatment at Navahoe is an ILAC. (FIGURE B.20)

The intersection of Jefferson and Algonquin Street is a negative offset intersection. Because of this, a full RCUT configuration is not possible; only a partial one. The south leg of the intersection is the second of four parallel streets between Signalized Intersections 21 and 22 that service the neighborhood on the south side of Jefferson, while the north leg is the first of two parallel streets that service the neighborhood on the north side of Jefferson in this same segment. Because of this network configuration, it is logical to install the direct left-turn for access to the north leg, due to the fewer access points to the neighborhood on the north side. (FIGURE B.20)

The intersection of Jefferson and Kitchener Street is a three-legged intersection with the branch servicing the neighborhood on the south side of Jefferson. Kitchener is the third of the four parallel low-volume local streets between Signalized Intersections 21 and 22 on the south side of Jefferson. A RCUT-type operation with a direct left turn will help maintain access to the south side of Jefferson by providing an alternative to Signalized Intersection 21. In addition, a direct left-turn at the location is a logical companion to the direct left-turn to the north side of Jefferson at Algonquin one block to the west. (FIGURE B.20)
Continental Street is the fourth of the four low-volume local streets accessing the properties and neighborhood on the south side of Jefferson between Signalized Intersections 21 and 22. Because it is one block from the direct left-turn onto southbound Kitchener, a direct left-turn onto Continental would be redundant. Thus, an ILAC would be the appropriate treatment at this intersection. (FIGURE B.20)

Gray Street is the second of the two streets that access the area on the north side of Jefferson between Signalized Intersections 21 and 22. As the last direct left-turn proposed is to access the south side of Jefferson, it is logical to provide a direct left-turn to access the north side of Jefferson next. As Gray Street is only one of two streets providing access to this area, this is a logical location for a direct left-turn. Thus, this intersection will function with an RCUT-type operational regime. (FIGURE B.20)

B.5.2 Signalized Intersection 22: Dickerson Avenue

Signalized Intersection 22, Dickerson Avenue, is a negatively offset four-legged intersection. (FIGURE B.21) At Jefferson, both legs of Dickerson are collectors providing access to the surrounding neighborhoods. Even though Signalized Intersection 22 has a negative offset, the legs are spaced far enough apart to make a full unsignalized RCUT both geometrically and operationally feasible. Furthermore, a full RCUT at Signalized Intersection 22 will maintain the direct access from both directions of Jefferson to both legs of Dickerson. (FIGURE B.21) In addition, a full RCUT at Signalized Intersection 22 will continue the alternating direct left-turn pattern beginning at Algonquin Street.

The PAA of Signalized Intersection 22 reveals that the proposed alternate
unsignalized operation will not impact the ability of pedestrians to cross the street as long as the crossing distance is 24 feet or less. Because the maximum proposed distance from curb to median is 28 feet, parking will need to be eliminated to maintain the ability of pedestrians to cross the street with the conversion of Signalized Intersection 22 to an unsignalized full RCUT configuration. Eliminating parking in the intersection itself between the two legs of the offset will satisfy this need. In addition, such an elimination will enhance the operation and safety of the intersection by eliminating unexpected and potentially hazardous movements and conflicts within the intersection proper.

**FIGURE B.21.** Intersection 22, Dickerson Avenue, and adjacent streets.
The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios. For the “worst case” scenario, both the delay benefit and the travel time benefit are negative, while for the “best case” scenario, all benefits, including delay and travel time, are positive. The highest benefit for both scenarios is the stopping benefit. Interestingly, for the “best case” scenario, the delay, which had a negative benefit for the “worst case” scenario, has the second highest benefit value.

**B.5.3 Unsignalized intersections: Lenox Street, Drexel Street**

Between Signalized Intersection 22, Dickerson Avenue, and Signalized Intersection 23, Coplin Street, are two unsignalized intersections – Lenox Street and Drexel Street, both of which are three-legged intersections that service the properties and neighborhoods on the north side of Jefferson. ([FIGURE B.21]) The intersection of Lenox Street and Jefferson is an OUI that, for the sake of corridor consistency, safety, and aesthetics, will be converted to an ILAC operational scheme. Because of its proximity to the proposed full RCUT at Dickerson Avenue, a direct left-turn at Lenox would be redundant, as the adjacent neighborhood is accessible via Dickerson. As Lenox is a low-volume local street, the conversion of this intersection to ILAC operation would have minimal impact on the overall quality and efficiency of traffic flow.

The unsignalized intersection of Jefferson and Drexel is a CUI that would be modified as part of the modifications to Signalized Intersection 23, Coplin Street. ([FIGURE B.21]) While the intersection of Jefferson and Drexel Street is a du jure three-legged
intersection, with the branch on the north side of Jefferson, due to the presence of a commercial driveway to a shopping center across from Drexel, this intersection is a de facto four-legged intersection. Because of the need for commercial vehicles to access this commercial property, it is logical to provide a direct left-turn from Jefferson into this commercial driveway on the south side of the intersection. This intersection, then, would operate similar to a partial RCUT.

B.5.4 Signalized Intersection 23: Coplin Street

Signalized Intersection 23, Coplin Street, like the CUI at Drexel Street, is a du jure three-legged intersection with the branch on the north side of Jefferson that is a de facto four-legged one due to the presence of a commercial driveway on the south side of Jefferson. (FIGURE B.22) Due to geometric constraints, a full RCUT is not possible at Signalized Intersection 23, although a partial one is. The commercial driveway at Signalized Intersection 23 services the same shopping development as the commercial driveway at Drexel; thus they effectively work in tandem. With a direct left-turn into the commercial driveway at Drexel, one at Signalized Intersection 23 servicing the same commercial development would be redundant. Thus, it is logical to have the direct left-turn at Signalized Intersection 23 provide access to the north leg of the intersection, Coplin Street. Such a direct left-turn would function as the companion to the direct left-turn to the south at Drexel. In addition, a direct left-turn to the north at Signalized Intersection 23 will continue the alternating direct left-turn pattern beginning at Algonquin Street.

The PAA of Signalized Intersection 23 reveals that the proposed alternate
unsignalized operation will not impact the ability of pedestrians to cross the street as long as the crossing distance is 24 feet or less, the same condition as for Signalized Intersection 22. Because the maximum proposed distance from curb to median is 28 feet, parking will need to be eliminated to maintain the ability of pedestrians to cross the street with the conversion of Signalized Intersection 22 to an unsignalized partial RCUT configuration. Eliminating parking between Drexel and Signalized Intersection 23 will satisfy this need. As most pedestrians crossing from the neighborhood on the north will want to access the

**INTERSECTION 23 DATA**

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**PEDESTRIAN ACCESSIBILITY**

\[ V = 1452 \]
\[ P = 16 \]
\[ V_c > V \text{ for } D \leq 24 \text{ ft} \]

**FIGURE B.22.** Intersection 23, Coplin Street, and adjacent streets.
shopping district on the south side between Drexel and Signalized Intersection 23, this is a logical location to encourage pedestrian crossings.

The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios. For the “worst case” scenario, the travel time benefit are negative, while for the “best case” scenario, all benefits, including travel time, are positive. The highest benefit for both scenarios is the stopping benefit, with the delay benefit the second highest for both.

B.5.5 Unsignalized intersections: Piper Boulevard/Lakeview Street, Eastlawn Street, Newport Street, Lakewood Street

Between Signalized Intersection 23, Coplin Street, and Signalized Intersection 24, Chalmers Street, are four unsignalized negative offset intersections – Piper Boulevard/Lakeview Street, Eastlawn Street, Newport Street, and Lakewood Street. Piper Boulevard/Lakeview Street would be a partial RCUT CUI resulting from improvements to Signalized Intersection 23. (FIGURE B.22) Piper Boulevard is the south leg of this intersection, and has a divided throat at Jefferson separating the two directions of travel. Lakeview Street is the north leg of this intersection, and is a typical low volume local street. Due to the short length of the negative offset, it is not practical to convert this intersection to a full RCUT operational stratagem. Because this intersection is only one block east of Signalized Intersection 23, installing a direct left-turn to the north at this location would be redundant. Thus, it is logical to have the direct left-turn at this intersection provide access to the south leg of the intersection, Piper Boulevard. Such a direct left-turn would
function as the companion to the direct left-turn to the north at Signalized Intersection 23. In addition, a direct left-turn to the south onto Piper Boulevard will continue the alternating direct left-turn pattern beginning at Algonquin Street.

As stated previously, the intersection of Jefferson and Eastlawn is a negative offset intersection. Like Piper Boulevard/Lakeview Street, the intersection of Eastlawn would be a partial RCUT CUI resulting from improvements to Signalized Intersection 23. (FIGURE B.22) Due to the short length of the negative offset, it is not practical to convert this intersection to a full RCUT operational stratagem. Both the north and south Eastlawn legs are low-volume local streets that are part of a larger grid; thus, this is not a factor in deciding which direction should have the direct left-turn access. Because this intersection is only one block east of Piper/Lakeview, installing a direct left-turn to the south at this location would be redundant; thus, it is logical to have the direct left-turn at this intersection provide access to the north leg of the intersection. Not only would such a direct left-turn function as the companion to the direct left-turn to the south at Piper/Lakeview, it also would continue the alternating direct left-turn pattern beginning at Algonquin Street.

The intersection of Jefferson and Newport Street is a negatively offset OUI. (FIGURE B.23) Due to the short length of the negative offset, it is not practical to convert this intersection to a full RCUT operational stratagem. Both the north and south Newport legs are low-volume local streets that are part of a larger grid; thus, this is not a factor in deciding which direction should have the direct left-turn access. Because this intersection
is only one block east of Eastlawn, installing a direct left-turn to the north at this location would be redundant; thus, it is logical to have the direct left-turn at this intersection provide access to the south leg of the intersection. However, since direct left-turn access would be provided to the south at Lakewood, one block to the west (FIGURE 5.25), installing a direct left-turn to the south at Newport would also be redundant. Thus, to avoid redundancy in either direction, and for the sake of corridor consistency, safety, and aesthetics, will be converted to an ILAC operational scheme.

As previously stated, the intersection of Jefferson and Lakewood is a negatively offset unsignalized intersection. (FIGURE B.23) Also as previously mentioned, this intersection will have direct left-turn access to the south. Because of the small offset distance between the north and south legs of the intersection, a full RCUT is not geometrically feasible. Because this intersection is a CUI modified in conjunction with the already completed modifications to Signalized Intersections 24, Chalmers Street, the direct left-turn access must be provided to the south leg to maintain the already established access to the south leg of Chalmers via u-turn at Lakewood.

B.5.6 Signalized Intersection 24: Chalmers Street

Signalized Intersection 24, Chalmers Street, like the unsignalized intersections to the east, is a negatively offset intersection. (FIGURE B.23) Subsequent to the commencement of this study, this intersection was modified with signalization being removed for the south leg of Chalmers, removal of a driving lane, and the construction of raised median. After the modifications, the intersection operates as a signalized partial
RCUT, with direct left-turn access to the north leg of Chalmers, and indirect access to the south leg. Vehicles wanting to access the south leg from either the north or east legs of the intersection must make a U-turn at the end of the median, currently between

![Intersection Diagram]

**INTERSECTION 24 DATA**

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<td>B-C</td>
<td>$109,015$</td>
<td>$154,895$</td>
</tr>
</tbody>
</table>

**PEDESTRIAN ACCESSIBILITY**

- $V = 1447$
- $P = 4$
- $V_c > V$ for $D \leq 36$ ft

**FIGURE B.23.** Intersection 24, Chalmers Street, and adjacent streets.
Lakewood and Chalmers. To improve the safety and efficiency of these movements, the median will be extended through Lakewood, with the median crossover provided at the south leg of Lakewood to provide direct access to the south leg of Lakewood and indirect access to the south leg of Chalmers. Under the newly completed reconfiguration, motorists wanting to turn left from the south leg of Chalmers must turn east and make a U-turn at the median opening at the north leg of Chalmers. Because these modifications have been recently completed, a partial RCUT with the direct left-turn onto the north leg of Chalmers, which is the new operational scheme, was selected and analyzed for this intersection. Although signalization was maintained at the north leg of Signalized Intersection 24, this intersection was also analyzed for an unsignalized operation.

The PAA of Signalized Intersection 24 reveals that the proposed alternate unsignalized operation will not impact the ability of pedestrians to cross the street as long as the crossing distance is 36 feet or less. Since under the recent reconfiguration the crossing distance is, in effect, this distance, then no further modifications will need to be made to accommodate pedestrians. The NBV analysis for this intersection indicates a positive net benefit with conversion for both the “worst case” and “best case” scenarios. The existing signals can be removed and the entire intersection operate as unsignalized.

**B.6 Previously modified intersections east of Chalmers**

**B.6.1 Unsignalized intersections east of Signalized Intersection 24 (Chalmers)**

East of Signalized Intersection 24, Jefferson Avenue was modified after the commencement of this study. These modifications include eliminating lanes in both
directions to provide two lanes in each direction, the construction of raised medians with median openings and left turn bays at unsignalized intersections, and the installation of bike lanes in both directions. These modifications end just west of Signalized Intersection 25, Alter Road, to allow a one lane transition to the raised median geometry that begins just east of Alter at the boundary between the city of Detroit and the Grosse Pointe communities. Because of the newness of these modifications, the unsignalized intersections between Signalized Intersection 24, Chalmers Street, and Signalized Intersection 25, Alter Road, were not included in the analysis.

B.6.2 Signalized Intersection 25: Alter Road

Signalized Intersection 25, Alter Road, also was not included in this study. First, since raised medians with different dimensions now exist on either side of Signalized Intersection 25, the distance between these two raised median sections through the intersection proper serves as a transition zone between the two. As this transition functions effectively, there is really no strong reason to modify it. Second, as the last street in the city of Detroit, Alter functions as an important corridor connecting all of the radial corridors crossing the city boundary. For services that do not extend beyond the city limit, it provides a way to stay within the proper jurisdictional limits. For example, transit service provided by the city of Detroit does not cross the city limit, and Alter Road provides a corridor to allow buses to avoid crossing the boundary. Finally, Alter Road is an arterial that becomes Outer Drive, a perimeter route that circumnavigates the city of Detroit; thus, Signalized Intersections 25 meets signal warrants. For the reasons
enumerated, no changes are proposed for Signalized Intersection 25, and the status quo, i.e. signalized operation, should be maintained.
APPENDIX C “PRIORITIZATION OF INTERSECTION MODIFICATIONS”

C.1 NBV (B-C) versus ROI (B/C)

There are two common methods used for determining the economic viability of a project – net benefit value (NBV), which is the difference between the monetized benefits of a project and its cost, and return-on-investment (ROI), which is the quotient of the monetized benefits of a project and its cost. The NBV is expressed in monetary units (e.g. dollars, euros, pounds, zloty), with projects having a positive NBV considered viable and those having a negative NBV not. Because the ROI is the ratio of the monetized benefits over the monetized costs (with both expressed in the same monetary unit), it is a dimensionless number; an ROI greater than one is considered to have a positive return, while an ROI less than one is considered to have a negative one (with an ROI of one being neutral, neither positive nor negative). Because it is a ratio, small changes to the denominator (i.e. costs) can have a significant impact on the ROI, whereas these same changes will have a more muted impact on the NBV.

To illustrate this phenomenon, an example is appropriate. Suppose a project has a net benefit of $2K and a net cost of $1.5K. The NBV of this project is $0.50K ($2K - $1.5K) and the ROI is 1.333 ($2K/$1.5K). Unforeseen expenses have increased the total cost to $1.75K, thus reducing the NBV to $0.25K and the ROI to 1.143. Instead of costs increasing, suppose the benefits decreased from $2K to $1.75K. As with the previous example of the costs increasing by $0.25K, the NBV with the costs being held constant and the benefits reduced by $0.25K is $0.25K, the same as before. The ROI, however, is not the same, as
the ROI for the same change in NBV with the benefits being reduced is 1.167, higher than that for the benefits being held constant and the costs increasing. A similar phenomenon can be observed for an increase of the NBV by $0.25K to $0.75K; when the benefits are held constant and the costs reduced, the ROI is 1.60, while the ROI is 1.50 when the benefits are increased and the costs held constant. Because of this inherent sensitivity of the ROI, it is prudent to use the NBV in determining the viability of a project, as it is less susceptible to manipulation or error and to yield erroneous results.

An example of this digression between the NBV and ROI analysis for this study is shown in TABLE C.1. Both the NBV and ROI were calculated for each traffic analysis, unadjusted unsignalized (i.e. no diversion after conversion to an alternative unsignalized configuration) and adjusted unsignalized (i.e. diversion after conversion). Note how although the percentage of change in the NBV from the unadjusted to the adjusted conditions increases continuously throughout, the percentage of change in the corresponding ROI does not. The percentage change in the NBV at Chalmers for the two scenarios is greater than that at Hibbard, yet the corresponding percentage change in the ROI is smaller due to the absolute magnitude of the benefit being smaller, thus creating a smaller ratio value (as the costs at both locations are the same). Despite this limitation, the ROI can be useful in the determination of which projects to construct.

C.2 Using the ROI to prioritize viable projects

In a perfect world, all viable projects (i.e. projects with a positive NBV) are constructed, as there is ample funding available to do so. The world is not perfect, and
TABLE C.1. Percentage change in NBV and ROI for the “Adjusted” and “Unadjusted” scenarios.

| MINOR STREET | INT # | UNADJUSTED | | | | ADJUSTED | | | | | | ΔNBV | ΔROI |
|--------------|------|------------|---|---|---|---|---|---|---|---|
| Cadillac     | 16   | $222,552   | 6.237 | $236,433 | 6.563 | 6.24% | 5.24% |
| Parker       | 12   | $123,426   | 3.904 | $132,610 | 4.120 | 7.44% | 5.53% |
| McClellan    | 15   | $355,984   | 9.376 | $393,196 | 10.252 | 10.45% | 9.34% |
| Van Dyke     | 11   | $795,766   | 19.724 | $918,205 | 22.605 | 15.39% | 14.61% |
| Burns        | 13   | $243,831   | 6.737 | $297,141 | 7.992 | 21.86% | 18.62% |
| Du Bois      | 3    | $162,195   | 4.816 | $203,858 | 5.797 | 25.69% | 20.35% |
| Saint Aubin  | 2    | $151,271   | 4.559 | $198,204 | 5.664 | 31.03% | 24.22% |
| Hibbard      | 14   | $215,424   | 6.069 | $305,418 | 8.186 | 41.78% | 34.89% |
| Chalmers     | 24   | $109,015   | 3.565 | $154,895 | 4.645 | 42.09% | 30.28% |
| Baldwin      | 9    | $162,849   | 4.832 | $234,811 | 6.525 | 44.19% | 35.04% |
| Seyburn      | 10   | $139,817   | 4.290 | $212,508 | 6.000 | 51.99% | 39.87% |
| Marquette/Garland | 17 | $180,554 | 5.248 | $274,458 | 7.458 | 52.01% | 42.10% |
| Coplin       | 23   | $112,219   | 3.640 | $196,925 | 5.634 | 75.48% | 54.75% |
| Harding      | 18   | $92,716    | 3.182 | $264,260 | 7.218 | 185.02% | 126.87% |
| Jos Campau   | 5    | $56,067    | 2.319 | $244,368 | 6.750 | 335.85% | 191.04% |
| Dickerson    | 22   | $32,477    | 1.764 | $183,314 | 5.313 | 464.44% | 201.18% |
| Lillibridge  | 19   | $3,402     | 1.080 | $299,277 | 8.042 | 8697.32% | 644.58% |

many jurisdictions, like the City of Detroit, have fiscal and budgetary constraints that prevent them from constructing all projects deemed viable with the NBV analysis.

Decision-makers within these restrained jurisdictions endeavor to make the most efficient use of these limited funds, and the ROI is a good measure of this, with a higher ROI signifying a more efficient use of these funds. Since the City of Detroit is one such fiscally restrained jurisdiction, and the intersections studied are all within the jurisdiction of the City of Detroit, a ROI analysis was performed on the seventeen viable intersections for both the unadjusted and adjusted unsignalized conditions, the results of which are shown in TABLE C.2.
As a higher ROI indicates a higher monetary efficiency, a ranking of ROI from highest (most efficient) to lowest (least efficient) is useful to determine the order in which the projects should be undertaken; in other words, prioritizing them. For fiscally constraint jurisdictions like the City of Detroit, the prioritization rankings would be followed until the budgetary limit is reached, and those projects falling below this limit programmed for construction. Projects falling outside the limit, and therefore not programmed, should be reevaluated when new funds become available. This reevaluation should consist of a NBV analysis of all the intersections to determine viability for conversion from a traditional signalized configuration to an alternative unsignalized

### TABLE C.2. Total benefits and NBV and ROI analysis of viable intersections.

<table>
<thead>
<tr>
<th>MINOR STREET</th>
<th>INT #</th>
<th>UNADJUSTED UNSIGNALIZED</th>
<th>ADJUSTED UNSIGNALIZED</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ΣB$</td>
<td>ΣB$‐ΣC$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(NBV)</td>
<td>(ROI)</td>
</tr>
<tr>
<td>Saint Aubin</td>
<td>2</td>
<td>$193,771</td>
<td>$151,271</td>
</tr>
<tr>
<td>Du Bois</td>
<td>3</td>
<td>$204,695</td>
<td>$162,195</td>
</tr>
<tr>
<td>Jos Campau</td>
<td>5</td>
<td>$98,567</td>
<td>$56,067</td>
</tr>
<tr>
<td>Baldwin</td>
<td>9</td>
<td>$205,349</td>
<td>$162,849</td>
</tr>
<tr>
<td>Seyburn</td>
<td>10</td>
<td>$182,317</td>
<td>$139,817</td>
</tr>
<tr>
<td>Van Dyke</td>
<td>11</td>
<td>$838,266</td>
<td>$795,766</td>
</tr>
<tr>
<td>Parker</td>
<td>12</td>
<td>$165,926</td>
<td>$123,426</td>
</tr>
<tr>
<td>Burns</td>
<td>13</td>
<td>$286,331</td>
<td>$243,831</td>
</tr>
<tr>
<td>Hibbard</td>
<td>14</td>
<td>$257,924</td>
<td>$215,424</td>
</tr>
<tr>
<td>McClellan</td>
<td>15</td>
<td>$398,484</td>
<td>$355,984</td>
</tr>
<tr>
<td>Cadillac</td>
<td>16</td>
<td>$265,052</td>
<td>$222,552</td>
</tr>
<tr>
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<td>17</td>
<td>$223,054</td>
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<tr>
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<td>$135,216</td>
<td>$92,716</td>
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<td>Lillibridge</td>
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<td>$45,902</td>
<td>$3,402</td>
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<tr>
<td>Dickerson</td>
<td>22</td>
<td>$74,977</td>
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<tr>
<td>Coplin</td>
<td>23</td>
<td>$154,719</td>
<td>$112,219</td>
</tr>
<tr>
<td>Chalmers</td>
<td>24</td>
<td>$151,515</td>
<td>$109,015</td>
</tr>
</tbody>
</table>
one, a ROI analysis of those deemed viable, and a ranking of the ROI values to determine programming priority. The prioritization based on the unadjusted scenario is shown in TABLE C.3, and based on the adjusted scenario is shown in TABLE C.4. Note that only Signalized Intersections 11 (Van Dyke), 15 (McClellan), 13 (Burns), and 17 (Marquette/Garland) have the same prioritization rankings (1,2, and 6, respectively) for both scenarios. Note that only Signalized Intersections 11 (Van Dyke), 15 (McClellan), 13 (Burns), and 17 (Marquette/Garland) have the same prioritization rankings (1,2, and 6, respectively) for both scenarios.

When comparing the ROI prioritization of the two scenarios, the location that experiences the most significant change is Signalized Intersection 19 (Lillibridge), which

<table>
<thead>
<tr>
<th>MINOR STREET</th>
<th>INT #</th>
<th>UNADJUSTED UNSIGNALIZED</th>
<th>ADJUSTED UNSIGNALIZED</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ΣB$\cdot$ΣC$^*$ (NBV)</td>
<td>ΣB$/\Sigma$C$^*$ (ROI)</td>
<td>PRIORITY RANKUU</td>
</tr>
<tr>
<td>Van Dyke</td>
<td>11</td>
<td>$795,766$</td>
<td>19.724</td>
</tr>
<tr>
<td>McClellan</td>
<td>15</td>
<td>$355,984$</td>
<td>9.376</td>
</tr>
<tr>
<td>Burns</td>
<td>13</td>
<td>$243,831$</td>
<td>6.737</td>
</tr>
<tr>
<td>Cadillac</td>
<td>16</td>
<td>$222,552$</td>
<td>6.237</td>
</tr>
<tr>
<td>Hibbard</td>
<td>14</td>
<td>$215,424$</td>
<td>6.069</td>
</tr>
<tr>
<td>Marquette/</td>
<td>17</td>
<td>$180,554$</td>
<td>5.248</td>
</tr>
<tr>
<td>Garland</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Baldwin</td>
<td>9</td>
<td>$162,849$</td>
<td>4.832</td>
</tr>
<tr>
<td>Du Bois</td>
<td>3</td>
<td>$162,195$</td>
<td>4.816</td>
</tr>
<tr>
<td>Saint Aubin</td>
<td>2</td>
<td>$151,271$</td>
<td>4.559</td>
</tr>
<tr>
<td>Seyburn</td>
<td>10</td>
<td>$139,817$</td>
<td>4.290</td>
</tr>
<tr>
<td>Parker</td>
<td>12</td>
<td>$123,426$</td>
<td>3.904</td>
</tr>
<tr>
<td>Coplin</td>
<td>23</td>
<td>$112,219$</td>
<td>3.640</td>
</tr>
<tr>
<td>Chalmers</td>
<td>24</td>
<td>$109,015$</td>
<td>3.565</td>
</tr>
<tr>
<td>Harding</td>
<td>18</td>
<td>$92,716$</td>
<td>3.182</td>
</tr>
<tr>
<td>Jos Campau</td>
<td>5</td>
<td>$56,067$</td>
<td>2.319</td>
</tr>
<tr>
<td>Dickerson</td>
<td>22</td>
<td>$32,477$</td>
<td>1.764</td>
</tr>
<tr>
<td>Lillibridge</td>
<td>19</td>
<td>$3,402$</td>
<td>1.080</td>
</tr>
</tbody>
</table>

**TABLE C.3.** Prioritization of intersection conversions for the “Unadjusted” scenario.
jumps from the lowest prioritization ranking (17) for the unadjusted scenario to one of the highest (4) for the adjusted scenario. If one were to program the ten highest ranked intersections for conversion, Intersections 9, 11, 13, 14, 15, 16, and 17 would be programmed regardless of scenario. However, if the unadjusted scenario is used, Intersections 2, 3, and 10 would also be programmed, while if the adjusted scenario is used, Intersections 5, 18, and 19 would be programmed. When using the ROI to prioritize programming, it is imperative that the analyst use the best data possible and have clearly defined guidelines as to how to proceed when the different scenarios yield different outcomes.

| MINOR STREET | INT # | UNADJUSTED UNSIGNALIZED | | ADJUSTED UNSIGNALIZED | |
|--------------|------|-------------------------|-------------------------|-------------------------|
|              |      | ΣΒ$-ΣC$ | ΣΒ$/ΣC$ (NBV) | PRIORITY RANK | ΣΒ$-ΣC$ | ΣΒ$/ΣC$ (ROI) | PRIORITY RANK | |
| Van Dyke     | 11   | $795,766 | 19.724 | 1 | $918,205 | 22.605 | 1 | |
| McClellan    | 15   | $355,984 | 9.376  | 2 | $393,196 | 10.252 | 2 | |
| Hibbard      | 14   | $215,424 | 6.069  | 5 | $305,418 | 8.186  | 3 | |
| Lillibridge  | 19   | $3,402  | 1.080  | 17 | $299,277 | 8.042  | 4 | |
| Burns        | 13   | $243,831 | 6.737  | 3 | $297,141 | 7.992  | 5 | |
| Marquette/Garland | 17 | $180,554 | 5.248  | 6 | $274,458 | 7.458  | 6 | |
| Harding      | 18   | $92,716  | 3.182  | 7 | $264,260 | 7.218  | 7 | |
| Jos Campau   | 5    | $56,067  | 2.319  | 15 | $244,368 | 6.750  | 8 | |
| Cadillac     | 16   | $222,552 | 6.237  | 4 | $236,433 | 6.563  | 9 | |
| Baldwin      | 9    | $162,849 | 4.832  | 7 | $234,811 | 6.525  | 10 | |
| Seyburn      | 10   | $139,817 | 4.290  | 10 | $212,508 | 6.000  | 11 | |
| Du Bois      | 3    | $162,195 | 4.816  | 8 | $203,858 | 5.797  | 12 | |
| Saint Aubin  | 2    | $151,271 | 4.559  | 9 | $198,204 | 5.664  | 13 | |
| Coplin       | 23   | $112,219 | 3.640  | 12 | $196,925 | 5.634  | 14 | |
| Dickerson    | 22   | $32,477  | 1.764  | 16 | $183,314 | 5.313  | 15 | |
| Chalmers     | 24   | $109,015 | 3.565  | 13 | $154,895 | 4.645  | 16 | |
| Parker       | 12   | $123,426 | 3.904  | 11 | $132,610 | 4.120  | 17 | |

TABLE C.4. Prioritization of intersection conversions for the “Adjusted” scenario.
C.3 Strategies for addressing scenario ROI ranking differences

C.3.1 Average ranking

One method to address the ranking differences conundrum is by using the average ranking for each intersection for both scenarios. A benefit of averaging is that it promotes the programming of those intersections that have high rankings for both scenarios (such as the seven found in the top ten rankings for both scenarios). However, there are two significant drawbacks. First, there is much greater likelihood of a tie, which can create a funding issue dilemma, especially if the funding threshold falls at a tied rank. For example, if only the highest ten ranked projects in TABLE C.5 can be funded, then only nine can be programmed, with some funds left unused, as the tenth position is a tie between three

<table>
<thead>
<tr>
<th>MINOR STREET</th>
<th>INT #</th>
<th>UNADJUSTED UNSIGNALIZED</th>
<th>ADJUSTED UNSIGNALIZED</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ΣB$-ΣC$ (NBV)</td>
<td>ΣB/$ΣC$ (ROI)</td>
<td>PRIORITY RANKUU</td>
</tr>
<tr>
<td>Van Dyke</td>
<td>11</td>
<td>$ 795,766</td>
<td>19.724</td>
</tr>
<tr>
<td>McClellan</td>
<td>15</td>
<td>$ 355,984</td>
<td>9.376</td>
</tr>
<tr>
<td>Burns</td>
<td>13</td>
<td>$ 243,831</td>
<td>6.737</td>
</tr>
<tr>
<td>Hibbard</td>
<td>14</td>
<td>$ 215,424</td>
<td>6.069</td>
</tr>
<tr>
<td>Marquette/</td>
<td>17</td>
<td>$ 180,554</td>
<td>5.248</td>
</tr>
<tr>
<td>Garland</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cadillac</td>
<td>16</td>
<td>$ 222,552</td>
<td>6.237</td>
</tr>
<tr>
<td>Baldwin</td>
<td>9</td>
<td>$ 162,849</td>
<td>4.832</td>
</tr>
<tr>
<td>Du Bois</td>
<td>3</td>
<td>$ 162,195</td>
<td>4.816</td>
</tr>
<tr>
<td>Seyburn</td>
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<td>$ 139,817</td>
<td>4.290</td>
</tr>
<tr>
<td>Harding</td>
<td>18</td>
<td>$ 92,716</td>
<td>3.182</td>
</tr>
<tr>
<td>Lillibridge</td>
<td>19</td>
<td>$ 3,402</td>
<td>1.080</td>
</tr>
<tr>
<td>Saint Aubin</td>
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<td>$ 151,271</td>
<td>4.559</td>
</tr>
<tr>
<td>Jos Campau</td>
<td>5</td>
<td>$ 56,067</td>
<td>2.319</td>
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<td>$ 112,219</td>
<td>3.640</td>
</tr>
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<td>Parker</td>
<td>12</td>
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<td>3.904</td>
</tr>
<tr>
<td>Dickerson</td>
<td>22</td>
<td>$ 32,477</td>
<td>1.764</td>
</tr>
</tbody>
</table>

TABLE C.5. Prioritization of intersection conversions using the average ROI ranking.
different intersections (10, 18, and 19), all of which cannot be funded. The second significant drawback to using the average rank is manifested in the case of the tie, as the average by itself does not indicate the variation between the ranks (i.e. rank differential). In other words, an intersection with a high rank differential is treated the same as an intersection with a low one.

C.3.2 Average ranking plus rank differential

One good tiebreaker to determine priority is rank differential. Rank differential is a measure of the variation between the two scenarios, specifically, the absolute value of the difference in ROI ranks between the unadjusted and adjusted scenarios. Intersections with a lower rank differential should be given higher priority than those with a higher one, as a lower differential indicates less influence by variations in the traffic volume data. As

<table>
<thead>
<tr>
<th>MINOR STREET</th>
<th>INT #</th>
<th>UNADJUSTED UNSIGNALIZED</th>
<th>ADJUSTED UNSIGNALIZED</th>
<th>AVG RANK</th>
<th>RANK DIFFERENTIAL</th>
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</thead>
<tbody>
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<td></td>
<td></td>
<td>ΣB$/ΣC$ (NBV)</td>
<td>ΣB$/ΣC$ (ROI)</td>
<td>PRIORITY</td>
<td>ΣB$/ΣC$ (NBV)</td>
</tr>
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<td>Van Dyke</td>
<td>11</td>
<td>$795,766</td>
<td>19.724</td>
<td>1</td>
<td>$918,205</td>
</tr>
<tr>
<td>McClellan</td>
<td>15</td>
<td>$355,984</td>
<td>9.376</td>
<td>2</td>
<td>$393,196</td>
</tr>
<tr>
<td>Burns</td>
<td>13</td>
<td>$243,831</td>
<td>6.737</td>
<td>3</td>
<td>$297,141</td>
</tr>
<tr>
<td>Hibbard</td>
<td>14</td>
<td>$215,424</td>
<td>6.069</td>
<td>5</td>
<td>$305,418</td>
</tr>
<tr>
<td>Marquette/Garland</td>
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<td>$180,554</td>
<td>5.248</td>
<td>6</td>
<td>$274,458</td>
</tr>
<tr>
<td>Cadillac</td>
<td>16</td>
<td>$222,552</td>
<td>6.237</td>
<td>4</td>
<td>$236,433</td>
</tr>
<tr>
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<td>9</td>
<td>$162,849</td>
<td>4.832</td>
<td>7</td>
<td>$234,811</td>
</tr>
<tr>
<td>Du Bois</td>
<td>3</td>
<td>$162,195</td>
<td>4.816</td>
<td>8</td>
<td>$203,858</td>
</tr>
<tr>
<td>Seyburn</td>
<td>10</td>
<td>$139,817</td>
<td>4.290</td>
<td>10</td>
<td>$212,508</td>
</tr>
<tr>
<td>Harding</td>
<td>18</td>
<td>$92,716</td>
<td>3.182</td>
<td>14</td>
<td>$264,260</td>
</tr>
<tr>
<td>Lillibridge</td>
<td>19</td>
<td>$3,402</td>
<td>1.080</td>
<td>17</td>
<td>$299,277</td>
</tr>
<tr>
<td>Saint Aubin</td>
<td>2</td>
<td>$151,271</td>
<td>4.559</td>
<td>9</td>
<td>$198,204</td>
</tr>
<tr>
<td>Jos Campau</td>
<td>5</td>
<td>$56,067</td>
<td>2.319</td>
<td>15</td>
<td>$244,368</td>
</tr>
<tr>
<td>Coplin</td>
<td>23</td>
<td>$112,219</td>
<td>3.640</td>
<td>12</td>
<td>$196,925</td>
</tr>
<tr>
<td>Parker</td>
<td>12</td>
<td>$123,426</td>
<td>3.904</td>
<td>11</td>
<td>$132,610</td>
</tr>
<tr>
<td>Chalmers</td>
<td>24</td>
<td>$109,015</td>
<td>3.565</td>
<td>13</td>
<td>$154,895</td>
</tr>
<tr>
<td>Dickerson</td>
<td>22</td>
<td>$32,477</td>
<td>1.764</td>
<td>16</td>
<td>$183,314</td>
</tr>
</tbody>
</table>

TABLE C.6. Prioritization of intersection conversions using the average ROI ranking and rank differential.
traffic volumes are highly variable, minimizing the influence of their fluctuations is important in ensuring a more efficient use of financial resources. Such a prioritization of the intersections analyzed is presented in TABLE C.6. As can be seen from the table, of the three intersections tied for the tenth position (10, 18, and 19), Intersection 10 has the lowest rank differential (i.e. its rank is the most consistent between the two scenarios), and thus would be programmed in the tenth position. Of the three intersections, Intersection 19 would be programmed last, as it has the highest rank differential and thus is the most sensitive to variations in traffic volumes.
APPENDIX D “TEMPLATE FOR SELECTION OF INTERSECTION MODIFICATION”

D.1 Overview

When selecting a geometric design for an intersection, it is imperative to make it “context sensitive”, i.e. that the design takes into account the unique characteristics of the transportation network at the location in which it is to be used. These characteristics fall into three categories: operational, geographical, and social. The operational characteristics of the intersection were considered when determining which intersections should be considered for conversion from a traditional signalized one to an alternative unsignalized one. Once an intersection has been deemed to be worthy of consideration for conversion, the geographical and social characteristics of the locale should be evaluated and analyzed to determine what alternative geometric design should be used at that location.

D.2 Geographic characteristics

D.2.1 Intersection type

There are four basic intersection configurations that are commonly found in a typical urban grid, three variations of a four legged intersection (TYPE 1 - no offset, TYPE 2 - positive offset, and TYPE 3 - negative offset), and a three-legged intersection (TYPE 4).

(FIGURE D.1) Often, there are variations with skews or with additional legs; these types of intersections are more complex and may not be suitable candidates for conversion to alternative unsignalized. For example, a skew may be to such an extreme as to create a significant sight distance or turning restriction, or additional legs may create confusion or
potential geometric hazards. Therefore, it is strongly recommended that intersections other than these four basic configurations be analyzed in-depth utilizing potential geometries more suited to their unique characteristics.

**INTERSECTION TYPE**

![Intersection Types Diagram]

**FIGURE D.1.** Intersection types at typical candidate conversion locations.

**D.2.2 Network type**

In addition to the intersection characteristics, it behooves the analyst to consider the overall network characteristics at the candidate intersection to ensure the selection of the superior design, i.e. the one that will have the least detrimental impact on the operations of the transportation network and access to adjoining properties. This is especially critical at four legged intersections where there are several different design options to choose from, such as a TYPE 1 INTERSECTION as shown in **FIGURE D.1**. The three types of networks are: **TYPE 1 – Balanced**, **TYPE 2 – Unbalanced**, and **TYPE 3 – One Access Point** and are shown in **FIGURE D.2**. The network type helps determine how much, and what type of access should be provided to the minor street and the adjacent land.
**NETWORK TYPE**

![Network Types Diagram]

**FIGURE D.2.** Transportation network types at typical candidate conversion locations.

uses, and is most relevant for analyzing potential alternative geometric configurations for TYPE 1 and TYPE 4 INTERSECTIONS.

**D.3 Land use characteristics**

For the purposes of this template, there are seven basic land use categories as shown in **FIGURE D.3.** These seven are – **TYPE 1, RESIDENTIAL; TYPE 2, PARK/RECREATIONAL; TYPE 3, SCHOOL/EDUCATIONAL; TYPE 4, INDUSTRIAL/COMMERCIAL; TYPE 5, POLICE STATION/HOSPITAL; TYPE 6, STADIUM/ARENA/CIVIC; TYPE 7, FIREHOUSE.** A **TYPE 1-RESIDENTIAL** land use area is one where most properties are used as residences with few, if any, commercial or large vehicles accessing these properties. **TYPE 1** land uses require the least amount of access. A **TYPE 2-PARK/RECREATIONAL** land use is a park, campground, golf course, marina, or similar open area open to the public (i.e. more than the property owner or tenant). **TYPE 2** land uses are low density, have little need for extensive commercial vehicle access, and thus are not significantly degraded by
restrictions in direct access. In emergency situations (e.g. inclement weather, natural or manmade disasters), it is crucial to be able to evacuate a TYPE 2 land use (and many people) as quickly and efficiently as possible; thus, quick egress has higher priority than quick egress. Private and public schools, colleges and universities, libraries, vocational-technical centers, and any other place where education is the primary purpose are all TYPE 3-SCHOOL/EDUCATIONAL land uses. As with TYPE 2 land uses, ease of egress has more importance than ease of ingress for TYPE 3 for the same reasons, i.e. that ability to evacuate many people in as quickly as possible in case of emergency. A hybrid of TYPE 2 and TYPE 3 is TYPE 6-STADIUM/ARENA/CIVIC, e.g. sports venues, amphitheaters, auditoriums, courthouses, and government centers, non-commercial, industrial, or residential facilities where large groups of people gather and where the need for quick evacuation in times of emergency (egress) takes precedence over inbound access. TYPE 5-POLICE STATION/HOSPITAL and TYPE 7-FIREHOUSE are emergency services uses. For

**LAND USE TYPE**

1. Residential
2. Park/Recreation
3. School/Educational
4. Industrial/Commercial
5. Police Station/Hospital
6. Stadium/Arena/Civic
7. Firehouse

**FIGURE D.3. Land use categories.**
emergency services, getting to the scene of the emergency quickly and efficiently is critical to the response; thus, maximum egress access is crucial. Fire departments are the first responders, and thus are given a separate land use designation from other emergency services. Finally, as good access to commercial and industrial properties is vital to their viability, unlike the other non-residential land use types, for TYPE 4-INDUSTRIAL/COMMERCIAL land uses, ingress access takes precedence over egress access. For evaluation purposes, TYPE 4 land uses include all non-residential land uses (e.g. office, lodging, retail, factories) not included in other non-residential land use types.

D.4 Design selection

When selecting the best unsignalized alternative geometry at a candidate location, there are three location characteristics that need to be considered: intersection type, network type, and adjacent land uses. For the seven identified land uses, there are eight different land adjacent land use scenarios, as shown in FIGURE D.4. It should be noted that for Scenario 1, only land use TYPES 1, 2, 3, and 4 are shown, and that for Scenario 2, only land use TYPES 1 and 4 are shown. For Scenario 1, it is assumed that in the rare locations where the land use types are TYPE 5, 6 and 7, removal of signals would not be considered; the same assumption is used for Scenario 2, TYPES 2, 3, 5, 6, and 7.

In Scenario 7, TYPES 6 and 7 are assigned priority over TYPE 5, POLICE STATION/HOSPITAL. At first blush, it would seem counterintuitive to prioritize a stadium, auditorium, amphitheater, or other similar civic building over emergency services providers; however there are three primary rationales for this prioritization. First, a public
venue such as a stadium has thousands of people that need to be evacuated quickly, whereas the number emergency responders from a particular hospital or police station are a fraction of that. Second, in larger urban areas, there are multiple hospitals or police stations from which emergencies are responded to; thus, any impedance to response from only one particular hospital or police station may have zero net detriment to the overall response time due to this redundancy. Third, hospital and police responders often are elsewhere in the urban area upon receipt of the emergency dispatch and therefore are not responding from the hospital or police station where they are based. It should be
noted that this third rationale does not apply to firefighters, who typically are at the firehouse when dispatched for an emergency; thus, access prioritization for firehouses supersedes that for police stations and hospitals. As firefighters are the first responders for catastrophic emergencies where the potential loss of life can be large, firehouses are given the highest access priority, as shown in Scenario 8.

*D.4.1 Typical alternate geometries*

While there are many different alternate geometries, there are three that lend themselves for use at unsignalized locations where right-of-way acquisition is not practical – Indirect Left and Crossing (ILAC), sometimes called a “J-turn”; Redirected Crossing U-turns (RCUT), i.e. full ingress access; and Reversed Redirected Crossing U-turns (RRCUT), i.e. full egress access. Due to geometric constraints, often it is not possible to construct a full RCUT or RRCUT at a four-legged intersection; at these locations, a partial RCUT or RRCUT may be used. The eight variations of the ILAC, RCUT, and RRCUT for use at both three-legged and four-legged intersections are shown in FIGURE D.5.

*D.4.2 Input/output matrix*

To assist the practitioner in selecting an appropriate alternate geometry for a particular location based on the existing intersection type (FIGURE D.1), existing transportation network type (FIGURE D.2), and adjacent land uses (FIGURE D.3), an input/output matrix has been created and is shown in TABLE D.1. For example, for a location with a *TYPE 1* intersection type, a *TYPE 2* network type, and a *TYPE 3* adjacent land use scenario, a *TYPE 7* alternative geometry (*Partial RCUT*), is recommended. A couple of observations about
the Input/Output Matrix should be noted. First, the output, alternative geometry type,

**ALTERNATIVE GEOMETRY TYPE (3 LEGGED)**

1. ILAC
2. RCUT
3. RRCUT

**ALTERNATIVE GEOMETRY TYPE (4 LEGGED)**

4. ILAC
5. FULL RCUT
6. FULL RRCUT
7. PARTIAL RCUT
8. PARTIAL RRCUT

**FIGURE D.5.** Possible alternative geometry types at typical candidate conversion location.
TABLE D.1. Selection matrix for typical candidate conversion location.

is a recommendation based on the input provided. This matrix is a general template; if local conditions require an alternative geometry different than that recommended by the matrix, then the design dictated by local conditions should always supersede the one recommended by the matrix. Second, the matrix favors more direct access. The most restrictive alternative geometry, the ILAC, is recommended for the fewest input combinations. This is by design, as the ILAC has the greatest detrimental impact, and should only be used where these detriments are minimal.
REFERENCES


Grzwskowiak, J. (2007) Cities see the forest through the trees- officials work on preserving urban tree canopy. *American City & County, 122* (3), 16.


How to reduce car engine noise. Retrieved September 21, 2016, from

http://www.ehow.com/how_5196217_reduce-car‐engine‐noise.html


Hummer, J.E., Blue, V.J., Cate, J., & Stephenson, R. (2012). Taking advantage of the flexibility offered by unconventional arterial designs. ITE Journal, 82(9), 38-43.


Institute of Transportation Engineers (2003). *Making intersections safer: a toolbox of countermeasures to reduce red-light running.* Washington, DC.


*Proceedings of the 83rd Annual Meeting of the Transportation Research Board,* Washington, DC.


*Proceedings of the 93rd Annual Meeting of the Transportation Research Board*, Washington, DC.


United States Census (2010)


*Proceedings of the 83rd Annual Meeting of the Transportation Research Board*, Washington, DC.
ABSTRACT

A MODEL TO EVALUATE VARIOUS UNSIGNALIZED INTERSECTION GEOMETRIES AND OPERATIONS FOR IDENTIFICATION OF POSSIBLE LOCATIONS TO USE IN LIEU OF A TRADITIONAL SIGNALIZED INTERSECTION

by

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August 2018

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Major:  Civil Engineering

Degree:  Doctor of Philosophy

Since its peak in 1950, the City of Detroit has lost over a million people, nearly two-thirds of its population. Unfortunately, its transportation infrastructure has not adapted. In 2013, Detroit had over 1500 traffic signals, many more than is necessary for a city of 700,000. Signals, once installed, are often incredibly politically difficult to remove when they are no longer operationally needed, resulting in unnecessary expenses with the maintenance of these signals; in the case of a city like Detroit, these expenses can be sizeable. Thus, a methodology to mathematically evaluate the removal of these unneeded signals is beneficial for a depopulating city, like Detroit, to more efficiently operate its transportation network.

Unfortunately, in an existing urban area, it may not be practical to remove signals and replace them with traditional unsignalized intersections due to sight distance restraints. It may also not be practical to rebuild such intersections due to right-of-way
restrictions and acquisition cost. There are alternate intersection geometries that have typically been used at signalized intersections that may be used for unsignalized ones to allow the removal of these unneeded signals. This research tests a mathematical model that allows for the analysis of such a conversion that consists of two parts: a net benefit value analysis, consisting of the benefits provided of such a conversion to travel time, delay, maintenance and operation, runoff, and stopping versus the difference in construction costs between the unsignalized geometry and replacement of the signal; a pedestrian accessibility analysis, a mathematical modeling of the ability of pedestrians to cross without signals.

Seventeen potential locations along the East Jefferson corridor were analyzed using this model, with each location having unique proposed unsignalized geometries. For each location, two different scenarios were tested: a “worst case”, where traffic volumes do not change from the signalized condition, and a “best case”, where some traffic is diverted due to the changed geometrics as determined by the California Diversion Equation. Because of the uniqueness of each proposed design, this method is easily applicable to any location and is useful for any transportation practitioner to use.
AUTOBIOGRAPHICAL STATEMENT

Michael Howard Schrader was born on April 12, 1966, in Saint Louis, Missouri, to Howard and Esther Schrader, the youngest of their five children. He grew up in University City, Missouri, an inner ring suburb of Saint Louis, and graduated from Saint Louis University High School, a Jesuit college preparatory school, in May 1984. He entered the University of Missouri, Columbia, in August 1984, to study civil engineering. While an undergraduate, Michael worked summers for the Missouri Highway and Transportation Department, where he gained valuable practical experience.

Upon receiving his Baccalaureate of Science in Civil Engineering from the University of Missouri in December 1987, Michael worked as a structural engineer for McDonnell Douglas Corporation until he entered the University of Tennessee, Knoxville, in January 1989 to pursue a Master’s Degree in Civil Engineering with a transportation emphasis, which he obtained in May 1990. Subsequently, Michael has worked as a professional engineer for both the public and private sectors, and is a registered professional engineer in Illinois, Missouri, Arkansas, and Oklahoma. In January 2013, Michael began his pursuit of a Doctor of Philosophy in Civil Engineering from Wayne State University in Detroit, Michigan.

Michael is the proud husband of Victoria, the proud father of Jacqueline, Elizabeth, Genevieve, Xavier, Nikolai, Maximus, and Isaiah, the proud stepfather of Dyllan and Cade, and the proud grandfather of Matthew.