

**DEVELOPMENT OF A NEW JUGHANDLE DESIGN FOR FACILITATING HIGH-
VOLUME LEFT TURNS AND U-TURNS**

by

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The jughandle is a category of unconventional intersection that redistributes left turns to improve capacity and safety. The New Jersey Department of Transportation, a pioneer in jughandle design, classifies jughandles as either Type A (forward ramp intersecting the cross street), Type B (forward ramp curving left to intersect the mainline), or Type C (reverse loop ramp).

This research has developed a new type of jughandle design referred to as Type A+B. This design closes the minor approaches at intersections and directs traffic through a jughandle onto the mainline. It also accommodates U-turns and mainline left turns in a manner similar to traditional Type B jughandles. A unique type of signal phasing, developed to accommodate this design, allows both jughandles to move concurrently. This type of intersection is hypothesized to be most appropriate for the retrofit of suburban arterials requiring installation of a median barrier. The retrofit would install a median barrier with the jughandles, and eliminate signals at intersections with low cross-street volumes, replacing them with right turns followed by U-turns (RTUT). The objective of this research is to determine whether this is an appropriate context for the design, and under what general volume conditions the Type A+B jughandle can reduce delay.

Simulation software was used to compare performance measures for the Type A+B jughandle against a conventional intersection and a traditional Type A jughandle, for a wide range of traffic volumes whose turn movement proportions were modeled after a suburban arterial. The research also tested use of this design on an existing suburban arterial in the Pittsburgh region. Measures of performance evaluated include intersection delay, additional footprint, fuel consumption, and number of stops. It was found that the Type A+B jughandle significantly reduced delay under high-volume conditions, and resulted in a much larger intersection footprint.

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1.0 INTRODUCTION

1.1 BACKGROUND

The jughandle is a type of unconventional intersection developed to remove left turns from the high-speed lanes of an arterial and to reduce left-turn conflicts, in order to improve safety and operations. The New Jersey Department of Transportation pioneered the implementation of jughandles in the United States, using them to reduce congestion on U.S. Route 1 in urban areas. New Jersey has remained the leader in the U.S., having built over 600 of them, but many other states use the design as well [1].

The jughandle has several variations, which the NJDOT categorizes into Types A, B, and C. Here, Types A and C are introduced together for their similar functions, and Type B follows.

The **Type A** (or “forward” or “near-side”) **jughandle** consists of a forward ramp that branches to the right of a road, where it intersects a cross street to the right of the main intersection. Drivers may turn left at this smaller, usually stop controlled, intersection and then proceed straight through the main intersection, or they may make two consecutive lefts in order to execute a U-turn. This design is effective in reducing the number of phases for a traffic signal, and can be used when all left-turn volumes are low to moderate.

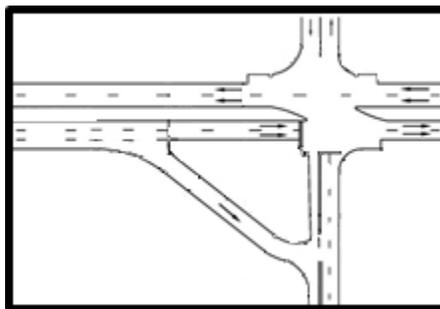


Figure 1-1. Type A jughandle [2]

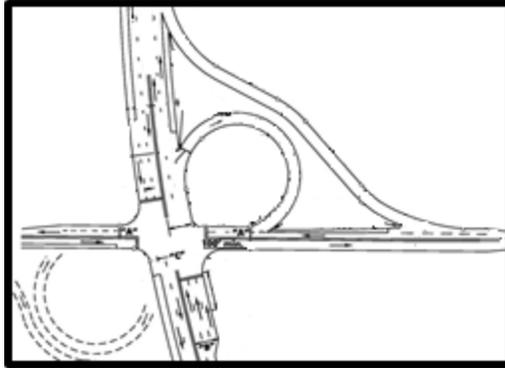


Figure 1-2. Type C jughandle [2]

A similar type of jughandle is the **Type C jughandle** (or “reverse” or “far-side” jughandle), which also places left-turning traffic onto the side street, but does so by the use of a loop ramp downstream of the intersection. This design reduces the number of conflict points and eliminates the left-turn movements onto the cross street, but has the disadvantages of forcing left turns through the intersection twice, of using more right of way, and of potentially creating a weave zone. Additionally, U-turns can become impossible, as drivers would have to merge from one side of the cross street to the other immediately upon entering.

The type of jughandle most conducive to U-turns is the **Type B jughandle**, which consists of a ramp veering forward and to the right before making a 90-degree left turn to intersect the main road at a right angle. In this scenario, left turns and U-turns will not need to pass through any intersections to reach the re-entry point. However, this type of jughandle can be used only on the far side of T intersections, or where an intersection does not currently exist. This makes its use somewhat limited, as U-turns are not generally a high priority. It is generally applicable on divided roadways that have relatively long distances between breaks in the median barrier.

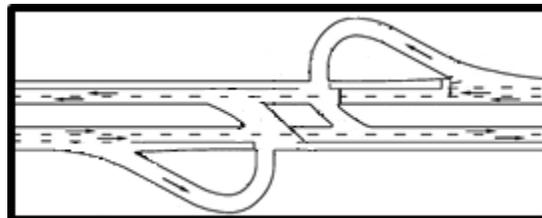


Figure 1-3. Type B jughandle [2]

Type A and Type C jughandles may be used on either major approach to an intersection or on both, and may be combined such that one approach uses Type A and the other uses Type C. Type B jughandles may be combined, as shown in Figure 1-3, to provide simultaneous U-turns in each direction; however, in this case, the intersection cannot include a cross street on either side.

In some cases, it may be advantageous to provide the opportunity for protected, high-radius U-turns on an arterial, but adding new traffic signals may increase delay and disrupt the progression. This thesis presents and investigates a new combination of jughandles developed to accommodate a Type B jughandle at an existing four-leg intersection.

Figure 1-4 depicts the proposed combination of two Type A and two Type B jughandles, referred to as a **Type A+B jughandle**. In this configuration, the main street would have a Type B jughandle on each approach. Each Type B jughandle would provide not only U-turn access for the major road but also left-turn access to a one-way cross street departing the intersection. The approaches *from* the cross street would not meet the major road at this same location, instead accessing it through Type A jughandles entering some distance upstream of the central intersection. The minor-street through movement would consist of a left turn onto the major road followed by a right turn at the central intersection.

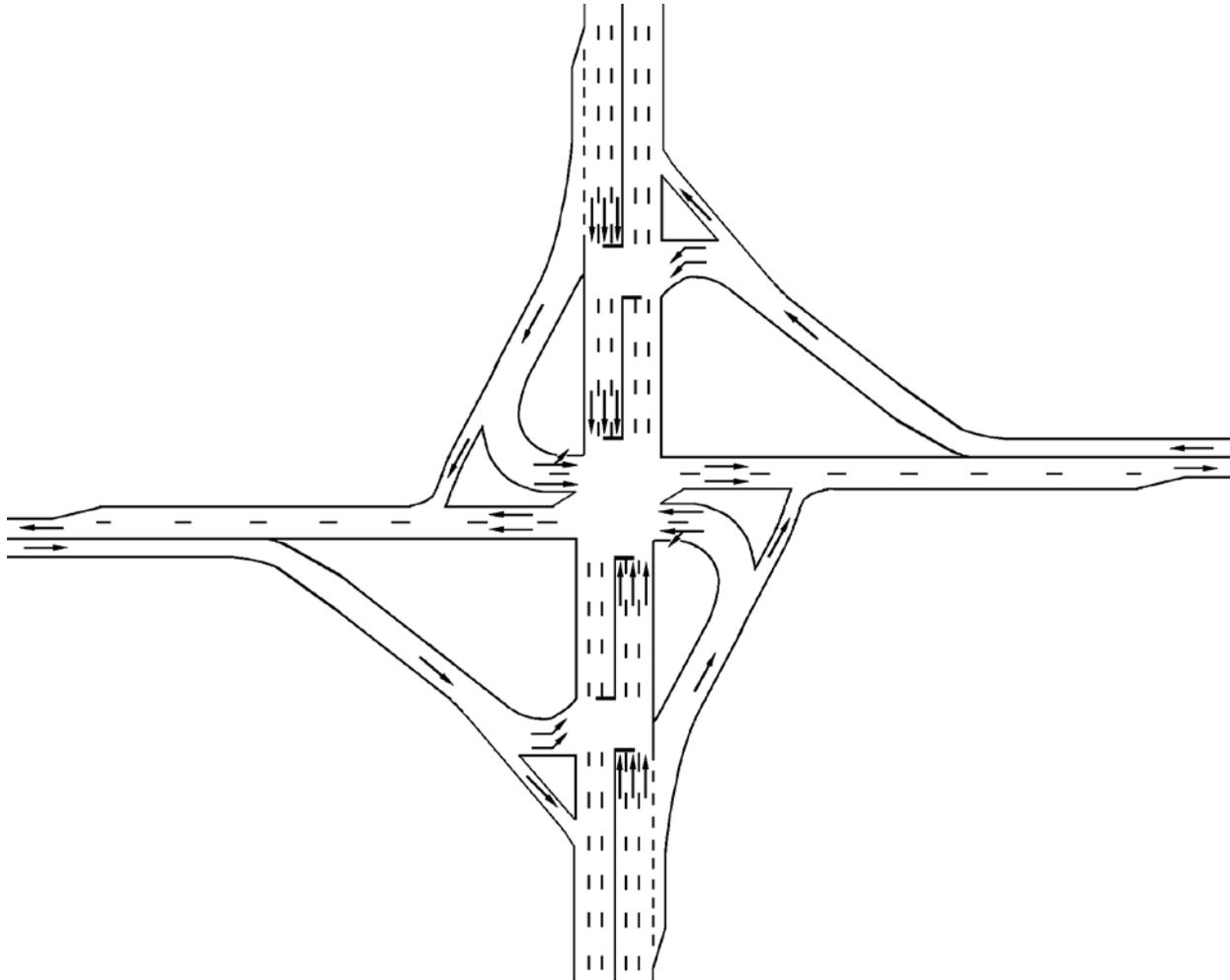


Figure 1-4. Type A+B jughandle

This design allows all left-turn, U-turn, and minor-street through movements to proceed in one signal phase, while both major-street through movements proceed in another. A standard Type A jughandle may need three phases, particularly for U-turns. By reducing this number, the Type A+B jughandle has the potential to reduce overall intersection delays, as well as improve coordination along an arterial. As a Type B jughandle, it provides a safer, easier, and higher-radius U-turn movement than any other signalized intersection. This aspect makes it a candidate for facilitating U-turns downstream of a median barrier, as in a divided highway with driveways or an unconventional intersection using an **RTUT** movement. It could permit the implementation of this configuration without forcing large vehicles to make sharp U-turns or requiring vehicles to merge across multiple lanes to cross the median. Note that, in Figure 1-4, the design is shown

with three through lanes and two left turn lanes, and the cross street is assumed to have only one through lane, requiring a merge. The number of lanes in each location may vary depending upon the specific location.

1.2 HYPOTHESIS

The author hypothesizes that, compared to a conventional intersection design, this design will have lower delay and fewer stops under certain traffic demand conditions, and that it will always have higher fuel use and a greater footprint. Specifically, it is believed that this design is best suited for high-volume arterials surrounded by low-density development, and intersections at which demand is high for left turns from all four approaches, significant for arterial U-turns approaching the intersection, and minimal for through movements on the intersecting street.

One such design scenario is that of the heavily developed suburban commercial arterial, in which nearly all non-through traffic is turning into or out of commercial driveways, and on which a median divider has become necessary for safety purposes, forcing an RTUT movement. When a conventional intersection's minor approaches must facilitate high volumes of left turns but very few straight movements, there may be inefficient phasing if double left-turn lanes require the addition of a protected phase despite minimal opposing through vehicles.

1.3 OBJECTIVES

The objectives of this thesis are to develop the concept of this intersection into a testable design; to determine whether the suggested context is indeed appropriate for the design; and to determine the approximate range of traffic volumes for which this design may provide superior performance, compared to a conventional intersection or other jughandle alternatives.

1.4 METHODOLOGY

The programs Synchro and SimTraffic will be used to compare performance measures for this intersection against both those for a large conventional intersection and those for a conventional Type A jughandle. Turn movement volumes of a fixed proportion will be raised incrementally to find the volumes for which each type of intersection provides an acceptable level of service within an acceptable footprint. A retrofit test case will then examine the impacts of converting an existing corridor segment of three signalized intersections to a central RTUT intersection and two outer Type A+B jughandles, in conjunction with the installation of a median barrier between the two Type A+B jughandle intersections. Performance measures for both tests will include total system delay, number of stops, fuel used, and intersection footprint. These tests will then be used to determine the benefits of the design and its applicability to certain contexts.

2.0 DESIGN ISSUES AND LITERATURE REVIEW

2.1 INTRODUCTION

This initial research will focus primarily on three topics important for the development of the Type A+B jughandle: the physical U-turn movement, jughandles, and unconventional intersections that involve an RTUT movement. These topics are relevant to the discussion of any jughandle design. Each section of this chapter will examine existing issues and the current literature on these issues. Following the U-turn section, there will be a brief introduction to unconventional intersections, as this topic is basic to the concept of jughandles and RTUT configurations.

2.2 U-TURNS

This section describes known design issues involving U-turns and examines current research on each issue. The issues created by U-turns, particularly when a median barrier is installed, may be a motivating factor in the use of jughandles to provide a substitute movement.

2.2.1 Right-of-Way

U-turns may often increase the right of way needed at a conventional intersection. As the turning movement with the most challenging geometry, the U-turn requires specific standards if it is to be accommodated by the road system. AASHTO's *A Policy on Geometric Design of Highways and Streets* calls for a 16-foot median for cars making U-turns about the median, and 50 feet for trucks. Figure 2-1 summarizes the requirements for various vehicles and roadway cross sections.

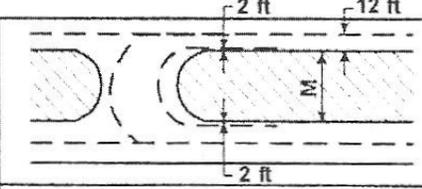
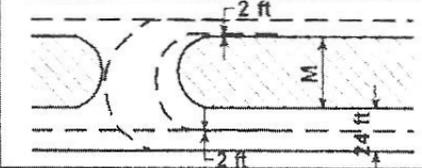
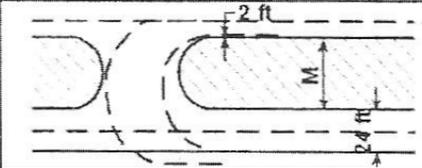
TYPE OF MANEUVER		M - MIN. WIDTH OF MEDIAN (ft) FOR DESIGN VEHICLE						
		P	WB-40	SU	BUS	WB-50	WB-60	TDT
		LENGTH OF DESIGN VEHICLE (ft)						
		19	50	30	40	55	65	118
INNER LANE TO INNER LANE		30	61	63	63	71	71	101
INNER LANE TO OUTER LANE		18	49	51	51	59	59	89
INNER LANE TO SHOULDER		8	39	41	41	49	49	79

Figure 2-1. Minimum median widths for U-turns [3]

Additionally, the U-turn opening should be no less than 50 feet before the beginning of an upcoming left-turn lane [3].

Research suggests that the U-turn movement should be disallowed when the road is not sufficiently wide for a continuous movement and vehicles are unable to use extra width, such as a shoulder [4]. The receiving pavement should also have a width greater than 24 feet [5].

2.2.2 Operations

While U-turns may have an essential role in some contexts, they can create operational problems. For capacity purposes, the Highway Capacity Manual considers U-turns to be equivalent to left turns. However, research suggests that this will not lead to an accurate analysis. Research on the operational impacts of U-turns follows.

Tsao and Chu found that U-turn headways were 1.27 times as high as those for left turns, and 2.17 times as high if executed following another U-turn. Liu et al. found capacity reductions

over 40% for U-turns in a left-turn lane. He et al. found headways of 2.91 sec, versus 2.23 for left turns and 2.15 for through movements; this is in agreement with Tsao and Chu, as the U-turn headway was 1.3 times that for the left turn. Bin Alam et al. further found that left-turn headways were also higher after U-turns than after other left turns. Headways were 1.9 seconds for left turns following left turns, 2.13 seconds for left turns following U-turns, 2.21 seconds for U-turns following left turns, and 2.37 seconds for U-turns following U-turns [6]. A simulation study by Combinido and Lim found that, despite the benefits of median U-turns, U-turn movements in general can have an exacerbating effect on traffic congestion once it exists [7].

A study by Liu found that discharge flow rate for U-turns does not reach a steady maximum, implying that a headway factor is not a sufficient adjustment. The study selected the following formulas to model potential U-turn capacity in vehicles per hour at an unsignalized intersection, for opposing through volume V in vehicles/hour:

$$\text{For shoulder encroachment not necessary: } V * \frac{e^{-.00178V}}{1 - e^{-.00064V}}$$

$$\text{For shoulder encroachment necessary: } V * \frac{e^{-.00192V}}{1 - e^{-.00086V}}$$

For a turning radius r in feet, U-turn time in seconds can be modeled by the equation:

$$3.23 * e^{22.55/r}$$

For an opposing through volume V and a U-turn volume U , both in vehicles/hour, average delay in seconds for vehicles executing U-turns was found to be:

$$1.22 * e^{.0009V + .002U + .798N}$$

where N is 1 for shoulder encroachment and 0 otherwise.

The study recommended providing U-turn opportunities before a signalized intersection for high U-turn demand, identifying the optimal placement of a median opening as mid-block before a signal, with sufficient distance from the source of vehicles needing to make U-turns [4].

Phillips et al. found a 1.8% decrease in saturation flow rate for each 10% increase in U-turn composition; the loss was 3.3% for each 10% when the minor approach's conflicting right

turn is protected. The shared left-and-U-turn lane's saturation flow rate in veh/hour/lane was found to be

$$1803 - U(4.323 - .484 \times R)$$

where U is the percentage of U-turns in the shared lane and R is the volume of conflicting right turns from the minor approach during the U-turn phase in veh/min.

However, the mere presence of protected conflicting right turns was found to be more significant to saturation flow rate than the actual volume thereof. The presence of a double left-turn lane was significant, but possibly only because it correlates with the presence of a protected conflicting right turn. Typically, left-turn lane groups did not experience a decrease in level of service until the U-turn percentage reached 70%.

A saturation flow rate adjustment factor for protected left and U-turns was found as

$$1 - U(.0018 - .0015R)$$

where U is the percentage of U-turns in the inner-most left turn lane and R is 1 for a right-turn overlap or 0 otherwise.

At unsignalized median openings, prior research suggested that capacity was given by

$$799 - .31V$$

where V is conflicting traffic flow and all units are passenger car units per hour [8].

Wang found that the speed of U-turns in miles per hour, for a radius R in feet, was

$$e^{1.78R^{.195}}$$

The left-turn flow rate adjustment factor for percentage of U-turns U was found to be

$$\frac{2.1399}{.000033U^2 + .003U + 2.1399}$$

This study did not account for start-up lost time, clearance lost time, heavy vehicles, restrictive geometry, or the impact of conflicting right turns [9].

From the above findings, it can be concluded that U-turns have a negative impact on operations, which may be more severe in some cases than in others depending on the particular conditions. This impact is not always reflected in current analysis methods for intersection operations.

2.2.3 Safety

Even with the proper roadway geometry, U-turns can increase the risk of crashes. They are perhaps the least common movement made by drivers and require a number of judgments that drivers are not accustomed to making. Some research on safety issues with U-turns follows.

Of 54 randomly selected sites and 24 sites selected solely for being U-turn “problem sites,” only 13 had any crashes involving U-turns, ranging from .33 to 3.0 crashes per year. These crashes fell into the categories of angle, for conflicting right turns from the minor approach; sideswipe, for drivers making U-turns from the outside of two left-turn lanes; and rear end, for vehicles colliding with the back of a U-turning vehicle. In one case, a vehicle turning right from the minor approach was rear-ended after yielding to a U-turn. Factors significant to the number of crashes were found to be the presence of double left-turn lanes, the volumes using the left-turn lanes, the volumes of right turns from the minor approach, and just the presence of a protected right-turn overlap (despite “U-turns must yield” warning signage in most cases). The number of crashes demonstrated that U-turns had a minimal impact on overall safety compared to the number of other crashes at the intersections studied [8].

A survey of practicing engineers determined that “No U-Turn” signs are generally used at any signalized intersection at which protected left turns from the mainline allow a protected right-turn phase from the minor approach, due to the conflict with U-turns. Additionally, it found a consensus that U-turns should be prohibited where any geometric condition prevents any vehicle approaching within 500 feet from seeing the vehicle executing the U-turn. Of course, U-turns also must be allowed only if AASHTO sight distance criteria are fulfilled [5].

The research reviewed in this section demonstrates that U-turns, even provided with specific geometric requirements, have the potential to negatively impact operations and safety.

2.3 UNCONVENTIONAL INTERSECTION CONFIGURATIONS

The following two sections discuss intersection designs that would be classified as unconventional intersections. These intersection types change the pattern of movements at a junction in order to achieve significant increases safety and efficiency. Many unconventional signalized intersections attempt to lower the number of phases at a congested junction in order to improve capacity more than would be possible through traditional means such as additional lanes. These major improvements can have a very high benefit-cost ratio when congestion reduction is taken into account, and the reduction and diffusion of conflict points increases safety as well [10].

The jughandle is one category of unconventional intersections. Another category involves the RTUT movement. Despite the increased distance and possibly increased travel time for those making these indirect movements, the overall delay for the intersection can often be reduced [10]. However, any unconventional intersection has unique implications, and requires specific conditions to be beneficial. In the following discussions of jughandles and RTUT configurations, these conditions and implications and the literature relevant to each will be examined.

2.4 JUGHANDLES

In this section, the known advantages and design issues involving current jughandle designs are described, and literature on these issues is then reviewed.

2.4.1 Right of Way

A primary concern with jughandles is the amount of right-of-way they require. Jughandles planned for new intersections are generally easier to accommodate, while those used to retrofit existing intersections must often comply with a number of geometric constraints or else incur high construction and right-of-way costs, and possible legal difficulties relating to the acquisition of property. The NJDOT specifies standards for roadway design of jughandles of each type.

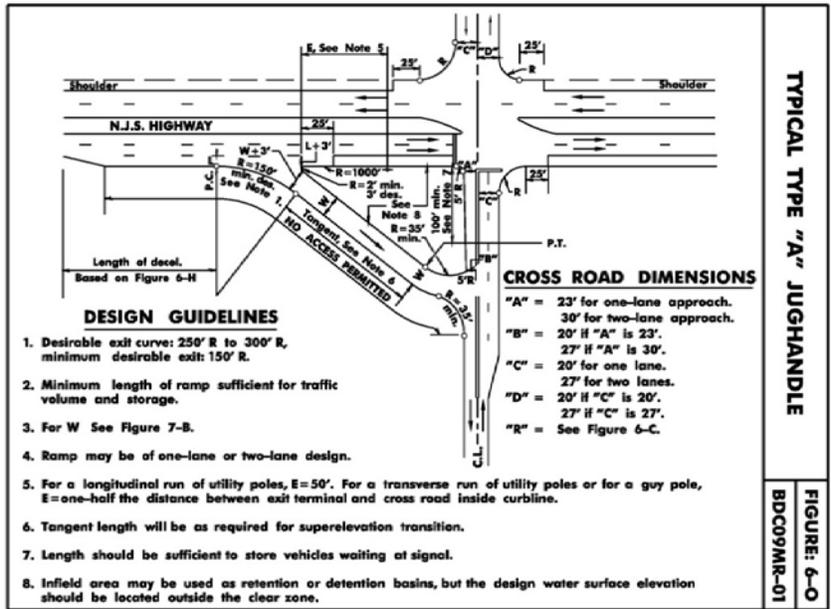


Figure 2-2. Type A jughandle design guidelines [2]

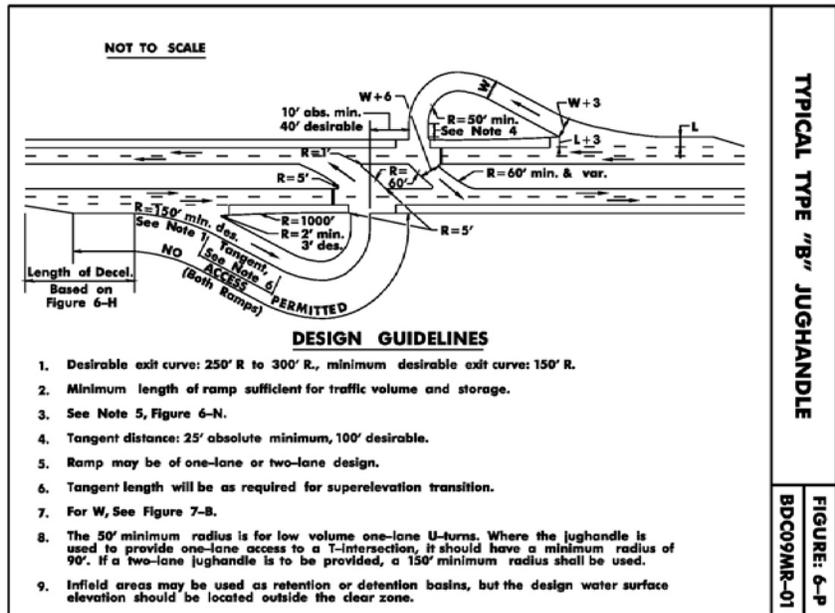


Figure 2-3. Type B jughandle design guidelines [2]

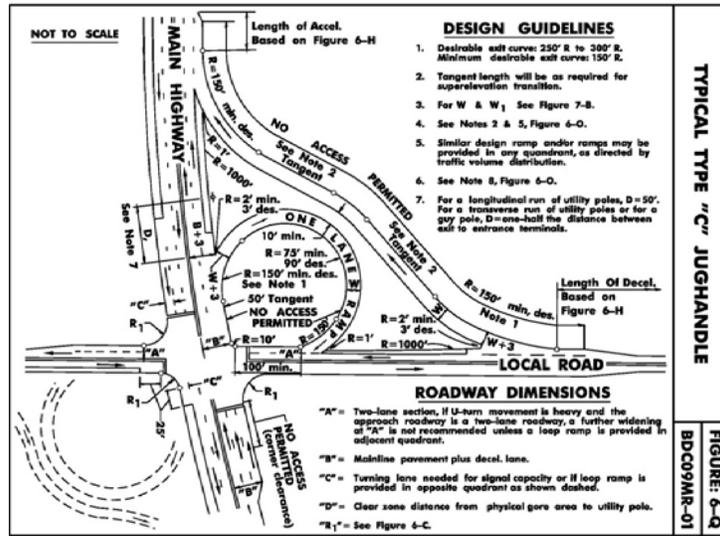


Figure 2-4. Type C jughandle design guidelines [2]

Minimum recommended design parameters for jughandles are given in Table 2-1 and Table 2-2.

Table 2-1. Minimum design speed for jughandle types [2]

Jughandle Type	Minimum Design Speed (MPH)
A	25
B (one lane)	15
B (one lane, T intersection)	20
B (two lanes)	25
C (loop)	15 (20 des.)
C (finger ramp)	25

Table 2-2. Minimum pavement width for jughandle radii [2]

Radius on Inner Edge of Pavement (ft)	Pavement Width (ft) for:			
	Entrance Width	Terminal	Ramp Proper Width One-Lane, One-Way Operation	Ramp Proper Width Two-Lane, One-Way Operation or Two-Way Operation
50		20	26	--
75		19	24	--
100		17	23	--
150		18	22	32
200		18	22	31
300		17	22	30
400		17	22	30
500		17	22	30
Tangent		17	22	29

NJDOT recommends acquiring the interior land area of jughandles, as well as prohibiting access along the exterior of ramps, except possibly along the taper length [2].

Research indicates that at least 100 feet must be provided between a Type A jughandle and the main intersection [11]. One study found that reducing this offset by 50%, from 450 feet to 230, reduced left-turn capacity by 30% [12]. Jughandles are recommended for consideration on multilane highways, divided or undivided, particularly when a median U-turn is not suitable [13]. As shown in Figure 2-4, the arterial design speed impacts the length of the deceleration lane but not the point at which the ramp diverges from the arterial [2]. It is suggested that, on an arterial with jughandles, the signals be spaced relatively far apart, so that the total cost of right-of-way does not become too large [14]. While jughandles do consume large amount of right-of-way at

intersections, they may be helpful on roads with a constrained right-of-way elsewhere, as they can remove extended or multiple left-turn lanes from the main approaches to an intersection [11].

In conclusion, it is clear that existing jughandle designs require a considerable amount of land area for construction and must merit this cost in improved operations and safety before they can be implemented.

2.4.2 Operations

Jughandle designs are often chosen specifically in order to improve operations. Under certain conditions, they can do so: for operational improvements, jughandles are generally recommended for arterials with high through volumes and low or moderate left-turn volumes [14]. A review of current literature on the operational impact of jughandles follows.

The main factors in a jughandle's operational capacity for left turns were found to be storage length, proportions of through and right-turning vehicles on the arterial, volumes on the minor approach, and for Type A jughandles, the mixing of queues for left and right turns. FHWA published a simulation study comparing Type A, Type C, and a combination of Type A and C jughandles to conventional intersections; results indicated 15-35%, 20-40%, and 25-40% lower delays for near-saturated conditions, respectively; all three designs had similar performance to conventional intersections for low-to-medium volume conditions. Maximum capacity was, respectively, 20-25%, 25-30%, and 25-40% higher than for conventional intersections. The study additionally found that protected phasing for left-turns from the minor approach improved minor road capacity by 30% but reduced overall capacity by 10% [12].

Another such study found that overall intersection travel time for jughandles, as compared to conventional intersections, was reduced by -6 to +51 percent for off-peak hours, and by +4 to +45 percent for peak hours. The overall percent of stops rose by 15 to 193 percent for off-peak hours, and by 19 to 108 percent for peak hours. It was concluded that jughandles could create more delay, stops, and travel distance than other intersections involving an indirect left-turn, but that they could still improve overall travel time and stops compared to a conventional intersection [15]. Because channelization saves distance for right turns, jughandles could sometimes reduce overall travel distance for an approach with a high ratio of right to left turns [16].

In another simulation study, Hummer compared travel time, distances traveled, and stops per vehicle for a Type A jughandle, three types of median U-turns, and conventional intersections. A statistical analysis found that left-turn volumes, through volumes, and intersection type were 99.99% significant to travel time and number of stops. The jughandle performed second-best in most categories, behind a stop-controlled median U-turn intersection, and it performed far better in distance travelled. It performed significantly better than a conventional intersection on travel time for low volumes of left turns, equally well at 150/hour, and more poorly at 200/hour. It performed significantly better than a conventional intersection for high through volumes, equally well at 500/hour, and more poorly at 250/hour. The number of stops at the jughandle as compared to a conventional intersection was equal for 50 left turns per hour, and inferior above; for average left-turn volumes, the jughandle was consistently inferior in number of stops, but it was less so for higher through volumes [16]. Other simulation studies showed that overall performance was similar to median U-turn intersections and conventional intersections, but that travel times were consistently lower for the jughandle [17].

2.4.3 Safety

While they may often be selected to reduce congestion rather than to reduce the number of crashes, jughandles can improve safety as well as operations. This would particularly be expected when jughandles are added in conjunction with the installation of a median barrier that is introduced to address a safety problem. The jughandle substitutes lateral gap selection for longitudinal gap selection; this creates a riskier turning movement, but allows drivers to make easier judgments due to a higher degree of relative motion for approaching vehicles. Jughandles may also improve overall safety on high-speed arterials by keeping exits on the right and allowing the traditional “passing lane” to operate at the higher speed [11].

The Type A jughandle reduces conflict points from 32 for a conventional intersection to 26, while Type C reduces them to 24. Left-turn crossing movements are reduced from 12 to 6 for Type A and from 12 to 4 for Type C [18]. However, Type C creates some weaving conflicts [11]. Figure 2-5 compares the conflicts in different jughandle designs.

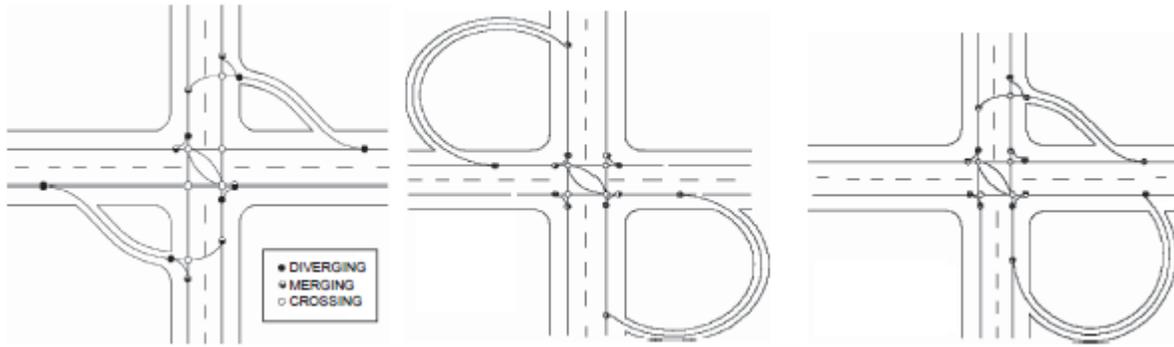


Figure 2-5. Conflict diagrams for jughandle designs [18]

Some research has been conducted on jughandle safety. A study of crashes at jughandles and at conventional intersections found that jughandles had lower rates of head-on crashes, and that more jughandle crashes were rear-end or property-damage-only than left-turn. Of Type A, Type C, and the Type A-Type C hybrid, Type A had 1.3 to 1.4 times as many crashes as either of the other types. The number of pedestrians involved in crashes at the jughandles was half that at conventional intersections, although the pedestrian volumes and vehicle movements involved in these crashes were unknown [18]. Based on this research, jughandles generally appear to be safer than conventional intersections.

2.4.4 Multimodal and Other Aspects

It is important that unconventional intersections such as jughandles not create significant problems for non-automobile users of the road. While jughandles have some disadvantages to multimodal users, they can also have some benefits. No literature specifically addressing multimodal aspects of jughandles was found, aside from the research regarding safety. However, notes on the multimodal advantages and disadvantages of jughandles follow.

Jughandles can decrease crossing distance for pedestrians by removing left-turn lanes from the arterial; pedestrians may also experience less delay if cycle lengths decrease. They may be at greater risk in crossing an unsignalized ramp terminal [15]; however, the channelization of right turns can reduce the risk of right-turn conflicts with pedestrians [11]. Bicyclists may be at greater risk where a ramp diverges, because of higher speed [15]. NJDOT recommends crosswalk

pavement, warning signs, and lighting for pedestrians, and notes that bicycle lanes can parallel vehicular lanes through the jughandle, if a significant number of bicyclists are likely to use it [2].

China has seen the addition of a supplementary left-turn lane between through lanes to service buses stopping near intersections at which they must turn left. These increase turning radius and reduce the need to merge quickly across wide streets. A study found that selection of the outside turning lane increased with the volume of traffic on the arterial, and that heavy vehicles were more likely to select the outside lane. However, more than 1% of entries to this lane were identified as inadvertent [19]. Type B or C jughandles could perform a similar function as these lanes with less conflict and driver confusion. Type B would additionally permit U-turns for buses on a loop route, although a four-leg intersection cannot accommodate a standard Type B.

The fact that jughandles cannot provide a physical barrier to left turns may reduce driver compliance, but it also retains the option for emergency response vehicles [11]. Because Type B jughandles cannot be used at a 4-way intersection, the NJDOT Design Manual recommends their implementation only in locations where adjacent land will not be developed due to environmental factors such as topography. Jughandles should be lit, as headlights may not be sufficient for the turning radii involved [2]. When a new jughandle is installed, drivers need to be educated on their use or provided with good visual cues [15]. Using jughandles as a consistent left-turn scheme along a corridor could reduce driver confusion [13].

These notes suggest that jughandles have the potential to provide some benefit to pedestrians and transit vehicles at an intersection. This research has not explored these issues, but recommends their future consideration.

The design issues and research discussed above indicate that, under certain circumstances, various types of jughandles can provide notable operational and safety benefits when compared to conventional intersections.

2.5 RIGHT-TURNS FOLLOWED BY U-TURNS

U-turns are frequently generated by restrictions on upstream or downstream movements that drivers desire to make. These restrictions may be viewed as one of several variations on the RTUT configuration. The simplest example is an unsignalized driveway with only right turns permitted, where all traffic must turn right and execute a U-turn in order to cross the major road or turn left. However, more complex examples form the basis for several unconventional intersection designs, unsignalized or signalized.

The Michigan left (or median U-turn) prohibits all left turns at an intersection, replacing the left-turn movement onto the arterial with an RTUT, and replacing the left-turn movement from the arterial with a U-turn followed by a right turn. A superstreet (or restricted-crossing U-turn, or J-turn) allows the left-turn movement from the arterial, but requires all traffic on the minor approach to turn right, which forces the minor through movement to make an RTUT and then a second right turn at the main intersection.

This review of the design issues and related literature for the RTUT strategy considers the strategy as a whole, and does not intend to recommend a particular variation.

2.5.1 Weaving Distance

One critical factor in both the safety and operations of an RTUT is the distance provided between the main intersection and the intended U-turn location. The Highway Capacity Manual states that there is no generally accepted methodology to analyze weaving distance on arterials, because the number of signals creates platoons [20]. However, for an RTUT, AASHTO recommends a distance of 400 to 600 feet between the intersection and the U-turn [3]. The Michigan Department of Transportation recommends 560-760 feet [21]. 600 feet is generally recognized as an evolved compromise [16]. A review of literature on the appropriate distance follows.

Research has found that the operational benefits of an RTUT configuration can degrade at an offset of more than 600 feet, but that safety may require it [22]. An insufficient offset distance has been shown to raise the probability of rear-end and right-turn or angle crashes between RTUT vehicles and through traffic. Models developed for a study at the University of South Florida

suggested a 3.3% decrease in crashes (4.5% for target crashes) for a 10% increase in separation distance. Additionally, more crashes in the weaving segment result from a U-turn at a signalized intersection [23]. Specifically, an offset of 400 feet is recommended for a 4-lane road using a midblock U-turn, 500 feet for a road of 6 or more lanes and a midblock U-turn, 550 feet for a 4-lane road using a U-turn at a signalized intersection, and 750 feet for a road of 6 or more lanes and a signalized U-turn. These distances do not include the necessary storage and transition lengths, which must be added separately [24].

Zhou et al. found the average weaving speed (in km/h) for a vehicle executing an RTUT was determined to be given by the equation $21.5 + .082L$ for an offset distance L (in m). This translates approximately to $13.36 + .0155L$ for distance in feet and speed in MPH. For an offset of less than 1000 feet, it was found that 85% of drivers prefer to wait for a simultaneous gap rather than attempt a weaving movement [25].

2.5.2 Right of Way

The RTUT configuration may require additional right of way where U-turns are accommodated. While U-turns do not in themselves require a turn lane, the volumes involved in an RTUT situation prompt AASHTO to recommend a median left-turn lane for storage [3]. Turn bays should be a minimum of 250 feet long, and longer for higher volumes or higher speeds. In order to accommodate trucks that must make a U-turn, required right-of-way is 139 feet for four lanes or 165 feet for eight, with medians between 47 and 71 feet. Special reductions such as loons could reduce this requirement to 84 feet for four lanes and 132 feet for eight, with a 4-foot median and 12-foot turn bay [21]. The NCHRP recommends a median of 40 to 60 feet for a U-turn at a signalized intersection; a midblock U-turn could use a median of less than 30 feet [26].

2.5.3 Operations

RTUT designs tend to be efficient when arterial volumes are at least twice as large as the volumes for the minor street; superstreets are generally considered to be appropriate for minor-street approaches with fewer than 20,000 vehicles per day [10]. One operational benefit of RTUT is

that it solves the problem of interlocking left turns. Additionally, signal progression works better with two-phase; in fact, a superstreet allows signal progression in both directions on the arterial, because the U-turn signals operate independently of each other. Median U-turn intersections are best suited for high major-road through volumes and moderate, balanced left-turn volumes from each approach. The superstreet configuration works best on high-volume suburban and rural arterials, particularly when there is low crossing volume [21].

Research suggests that RTUT travel times are comparable for heavy volumes and offsets less than .5 miles; and that delay is lower, but under only moderate or high volumes. Chowdury, Derov, and Tan found that left sending U-turns to existing intersections improves operations above 650 vehicles per lane per hour on divided highways. However, these restrictions worsened operations above 450 vehicles per hour per lane on undivided highways [13].

NCHRP Report 420 cites a 15-20% gain in capacity for a signal using two-phase control with downstream U-turns rather than multi-phase control with double left-turn lanes. For unsignalized intersections, the RTUT scheme should reduce delay, even for the left-turn movement, when arterial volumes are above 375-500 vehicles per lane per hour. For an offset of less than .5 miles, even the two-stage left-turn, in which a vehicle crosses the near lanes after a gap from the left and then waits in an enlarged median for a gap from the right, still causes more delays than an RTUT when arterial volumes are above 2000 vehicles per hour and when more than 50 vehicles per hour are making this left-turn movement [26].

A median U-turn configuration can increase throughput 30 to 45 percent over conventional intersections, and generally decreases net network travel time. In one simulation study by FHWA, superstreet travel time during peak hours was 10% lower for than TWLTL and speeds were 15% higher; they were comparable off-peak. For vehicles on the minor approach representing 10-15% of the total intersection volume, there was a 25-40% reduction in network travel time; for 18-25% composition, travel time was comparable to that at a standard intersection; and above 25% composition, travel time was 15-25% higher [21].

Median U-turns have been found superior to TWLTL in vehicle speed and in system time for peak periods; equal in system time for off-peak periods and in stops for peak; and inferior in stops in off-peak [14]. Liu found that, while RTUT does not increase overall system delay and travel time, providing the U-turn opportunity at a signalized intersection rather than a median turnaround does increase these measures for vehicles making the RTUT maneuver. Directional

median openings may be more efficient than superstreets when through traffic volumes are above 4000 vehicles per hour and left-turn volumes are simultaneously above 150 vehicles per hour [4].

For roads with driveways forced to use RTUT, AASHTO recommends a median opening every 400 - 800 feet [3]. Chowdury et al. found that using jughandles for an RTUT causes less delay than a median or intersection U-turn [13].

2.5.4 Safety

As volumes increase on an expressway, crashes become more common at intersections than between, with frequency and severity being highly dependency on the volume of traffic from the minor road [11]. RTUT configurations may often be implemented in order to remove left-turn conflicts and improve safety. Closing the median at unsignalized intersections can reduce the number of crashes, as long as left- and U-turn movements are sufficiently provided for. Figure 2-6 compares the number and distribution of conflicts for direct left turns, RTUT with directional median openings, and a superstreet configuration with no unsignalized left turns.

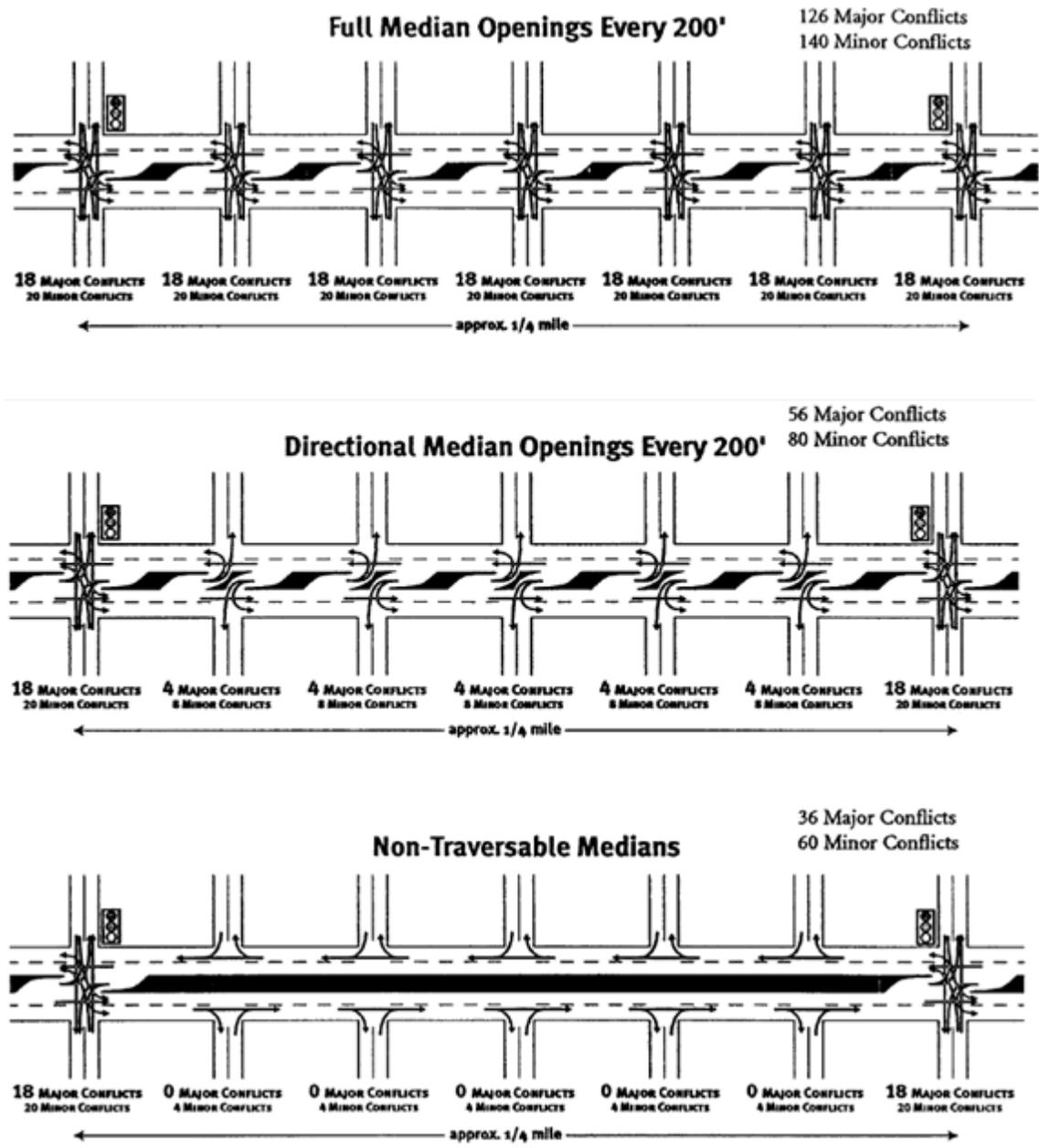


Figure 2-6. Conflicts for corridors with various levels of access [26]

Research suggests that configurations with left-turn restrictions are much safer for high-volume, multilane, major arterials than is a traditional configuration with direct left turns [13]. NCHRP Report 420 indicates that nontraversable medians had 41% lower accident rates than undivided highways, and 29% lower than roads with a TWLTL. It states that U-turns as an alternative to direct left turns can reduce accident rates by 20%, or 35% when signalized, and that roads with directional U-turn crossovers on wide medians have about half as many accidents as TWLTL roads [26].

Michigan Lefts have been found to have 20-50% lower crash rates than conventional intersections. In particular, high-injury crashes (head-on and angle) dropped significantly. The installation of these intersections reduced overall rear-end, angle, and sideswipe crashes by 17, 96, and 61 percent, respectively [21]. Other studies found that Michigan Lefts had collision rates 52% lower than unsignalized TWLTLs, 51% lower than signalized TWLTLs; 22% higher than unsignalized roads with conventional median openings, and 20% lower for signalized roads with conventional median openings [14].

A report by the Georgia Department of Transportation found that a raised median had a crash rate uniformly 45% lower, and an injury rate 43% lower, 48% mid-block. The mid-block fatality rate was 26% lower. Pedestrian fatalities were 78% lower overall. This report suggested that increasing driver distraction may make non-traversable medians increasingly important [27].

A report by the FHWA concluded that RCUT intersections had resulted in a 44% decrease in crashes, as well as reduced severity, and recommended the design for consideration for minor-road approaches with sufficient volume, stating that an increase in volume on the major road would likely increase the benefits of the RCUT [28].

A survey of engineers found a consensus that, if a location requires a left-turn lane but the right-of-way does not permit one to be installed, then the left-turn movement should be restricted, but only if an alternate location is available within one block of the location; the accident history threshold should be 5 crashes in 12 months related to left or U-turns [5].

2.5.5 Multimodal and Other Aspects

RTUT configurations have some negative impacts on non-automotive users. Medians can improve pedestrian safety by providing an area of refuge [26]; however, with refuge available, crossing may take two cycles; for Michigan lefts, the Michigan Department of Transportation determines minimum green time from distance from the curb to the median. In the case of a superstreet, pedestrians are forced to cross in an inconvenient Z shape. At Michigan-left intersections, bicyclists have the option of following the path for automobiles or dismounting and using the crosswalks; shared-use sidewalks may be appropriate here. Median U-turns that encroach upon the shoulder run the risk of endangering opposing bicyclists [21].

As with jughandles, Michigan-left intersections leave left turns physically possible, necessitating publicity and/or enforcement. However, this again permits emergency response vehicles to turn directly, and buses could also be exempted from left-turn prohibitions by means of “Except Buses” signage. Unsignalized median U-turns may require lighting for safety [21].

2.6 CONCLUSION

This review of design issues and related literature demonstrates that the RTUT scheme is often effective in improving operations and safety; that the resulting U-turns may be difficult to accommodate in many cases; and that jughandles may be beneficial under some conditions. It further suggests that jughandles may be particularly well-suited to facilitating large volumes of U-turns upstream or downstream of an intersection with left-turn restrictions. Because they feature U-turns from the right, jughandles used with an RTUT configuration may provide the safety and operational benefits without the weaving distance issues. Based on this, the hypothesis appears reasonable in its suggested application of the Type A+B jughandle for RTUT movements. The Type A+B jughandle permits U-turns in the style of the Type B jughandle but does not introduce a new signal to the corridor.

3.0 DESIGN AND OPERATIONAL GOALS FOR TESTING

3.1 INTRODUCTION

This chapter intends to provide a basis for testing the hypothesis by establishing basic parameters for the design of a Type A+B jughandles. First, terminology will be established. Next, the geometric aspects of the design, both the length of the design and the footprint of the ramps, will be determined. Finally, although phasing will ultimately be selected by Synchro, possible phasing patterns will be explained, and their implications will be quantified.

3.2 TERMINOLOGY

This and future chapters will refer to components of the Type A+B jughandle as follows: the three intersections involved will be referred to as the **central intersection** and the two **outer intersections**. The space between the central and outer intersections is called the **queue space**. The road containing these three intersections will be referred to as the **mainline**, while the other approaches, for the **minor road**, are the **minor approaches**. The ramps will be referred to as the **A jughandles** and the **B jughandles**, to distinguish them from the other intersections themselves (referred to as **Type A jughandles** or **Type B jughandles**). While the intersections are referred to separately, they share a single signal operation. The phases are called the **mainline phase** (through traffic proceeds), the **turn phase** (vehicles execute left turns from the A jughandles and left or U-turns from the B jughandles), and the **clearance phase** (vehicles clear the queue space). In this document, the “clearance phase” is not used to refer to a yellow or red interval, but an entire phase during which the queue space may clear. Figure 3-1 provides a diagram of these terms.

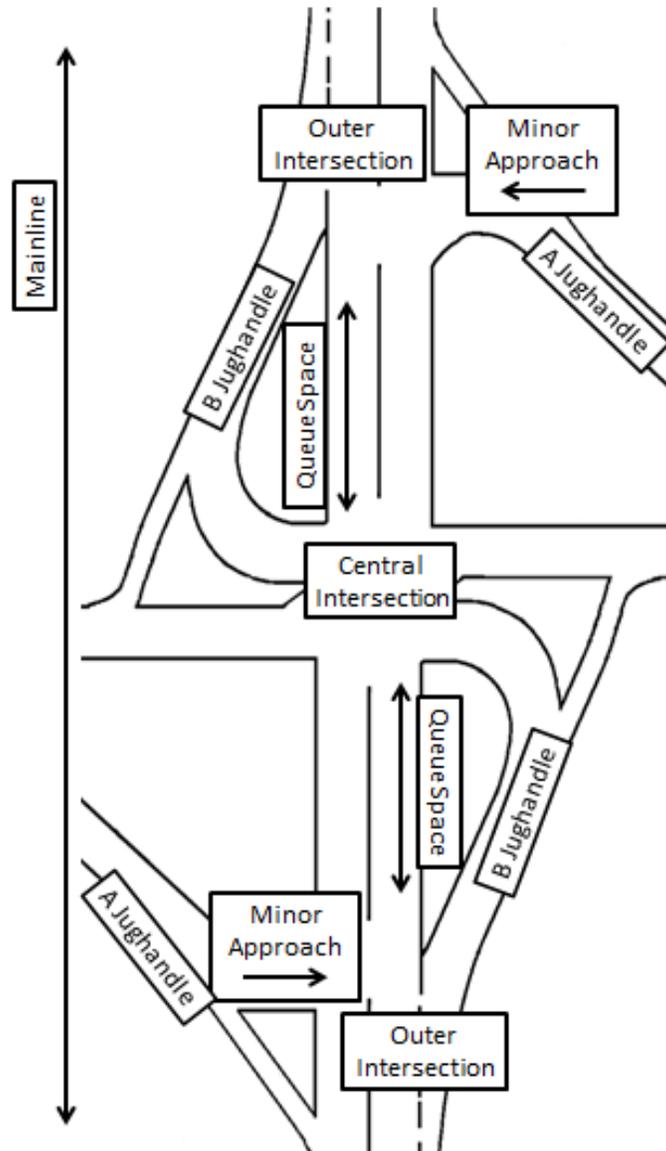


Figure 3-1. A+B jughandle terminology

3.3 QUEUE SPACE LENGTH

In this section, formulas for the required queue space length under different conditions will be developed. This is important for the initial design of the Type A+B jughandle. All formulas in this section are theoretical and not empirical, and thus are arrived at through mathematical development after some basic assumptions. Figure 3-2 labels the Type A+B jughandle with the variables that will be used. Variables such as D_A may be used without numeric subscripts when used without referring to a particular direction.

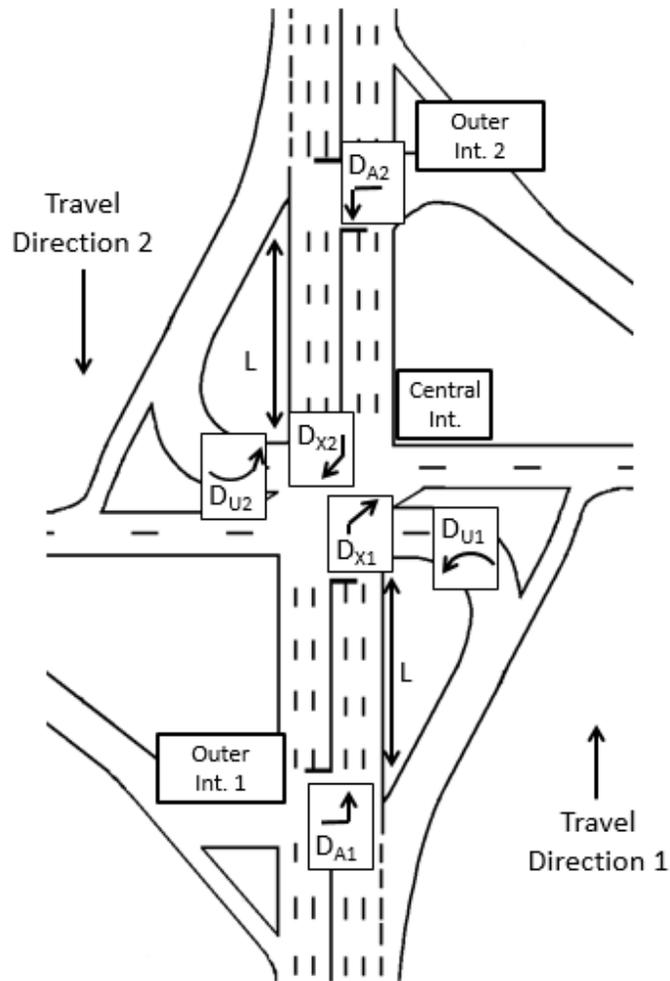


Figure 3-2. A+B jughandle labeled with quantities

The length required between the central and outer intersections for a Type A+B jughandle will depend upon the traffic demand generating queues in this queue space. As assumption of vehicle sizes composing the traffic stream must be made in order to determine queue length. Heavy vehicles will have a stored length of 45 feet and compose 5% of traffic, while passenger cars with a stored length of 25 feet will compose the other 95%. The average length of a vehicle in the queue is then 26 feet [20]. For this vehicle length, assuming that all the vehicles arriving in one cycle are allowed to join the queue, a theoretical formula for the queue length can be developed. Average vehicle length is multiplied by the number of vehicles arriving in one cycle and divided by the effective number of lanes:

$$Q = \frac{26 D C}{3600 n f_{LU}}$$

where

Q = peak queue length (feet)

D = peak demand volume entering the queuing area (vehicles/hour)

C = cycle length (seconds)

n = number of lanes

f_{LU} = lane utilization factor

Assuming that all vehicles arriving in one cycle are to be admitted to the queue space, the actual distance an outer intersection must be placed from the central intersection is the greatest of the distances needed for different movements utilizing the queue space. This space is used by minor-approach vehicles turning left onto the mainline in order to make a left turn, by minor-approach vehicles turning left onto the mainline in order to cross it by means of a subsequent right turn at the central intersection, and by mainline vehicles executing a U-turn.

Because of the unusual minor-road through movement involved in this design, there are multiple possibilities for how the minor approach's left turns will distribute in the approach lanes and within the queue space, and so appropriate values for f_{LU} are not immediately clear. These possibilities must be examined to ensure that the queue space is long enough for actual lane utilization and does not overflow. An even vehicle distribution on the approaches is not expected. Figure 3-3 depicts the two possibilities for vehicle distributions that will be considered, where t represents the number of turn lanes from the A jughandle and n continues to represent the number of through lanes.

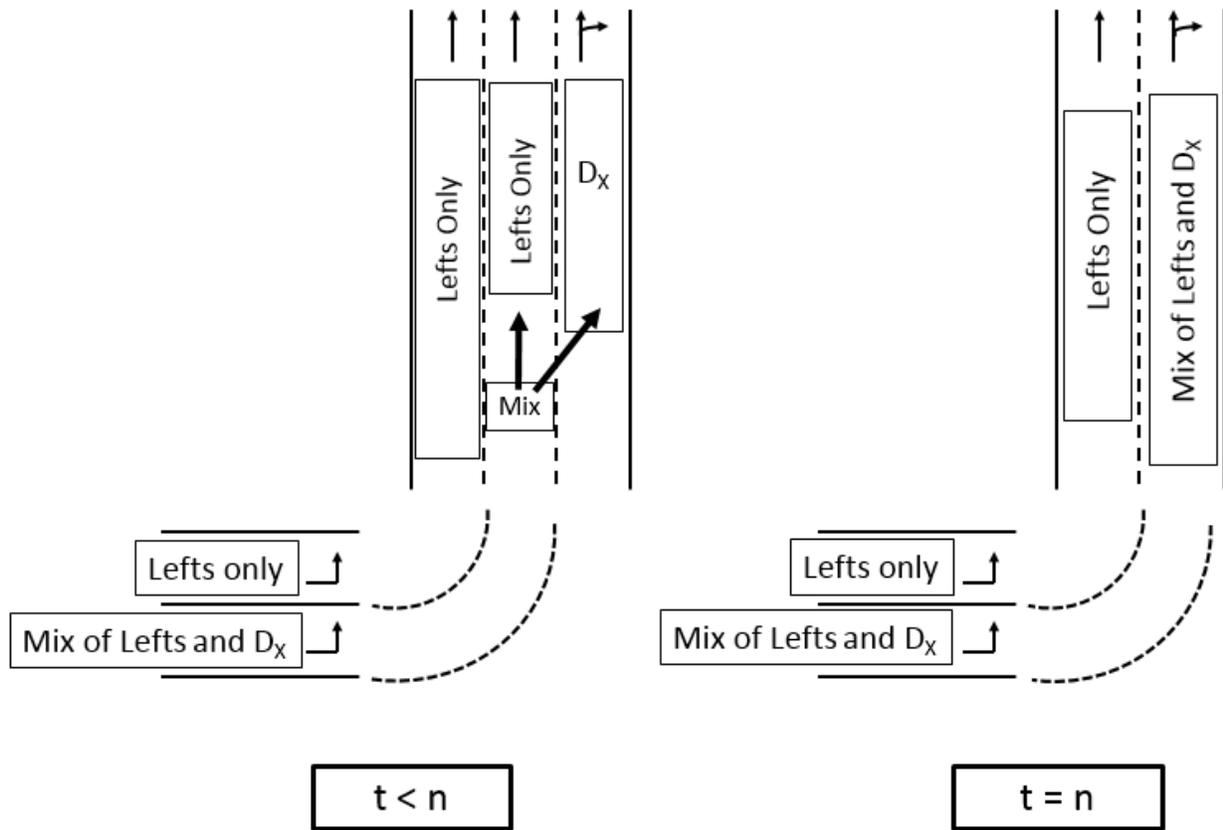


Figure 3-3. Diagram of lane utilization given the number of left-turn and through lanes

When $t < n$, left turns from the minor approach should be guided into the extreme left lanes of the mainline to reserve the right lane for the minor-approach crossing movement. Since the Type A+B jughandle is not likely to be used with a high volume of crossing traffic, it is assumed that only the right-most lane of the mainline approaching the central intersection will be permitted to turn right for this movement. It is also assumed that, given sufficient demand for this movement and more receiving lanes than left-turn lanes in the A jughandle, traffic from the minor approach will follow striping into the extreme left lanes of the queue space, and only right turns will enter the right lane.

Under these conditions, the hourly demand rate in the right lane of the mainline here is D_X . The hourly demand rate in the left lane or lanes is $D_A - D_X$, and the lane utilization factor is referred to as $t(f_{LU})_A$. Demand in the U-turn queue is D_U , with a lane utilization factor of $(f_{LU})_U$.

Therefore, choosing the maximum value of the queue length formula among all three queues, the required queue length Q_R (in feet) for one signal is given by:

$$Q_R = \frac{26 * \max \left\{ D_X, \frac{D_A - D_X}{t(f_{LU})_A}, \frac{D_U}{n(f_{LU})_U} \right\} * C}{3600}$$

for $t < n$

where

D_X = peak demand volume crossing the major road (vehicles/hour)

D_A = total peak demand volume passing through the A jughandle (vehicles/hour)

D_U = peak demand volume making a U-turn through the B jughandle (vehicles/hour)

t = number of turn lanes for the A jughandle

$(f_{LU})_A$ = lane utilization factor for vehicles queuing after a left turn through the A jughandle

$(f_{LU})_U$ = lane utilization factor for vehicles queuing after a U-turn through the B jughandle

On the other hand, in the case that the number of receiving lanes is equal to the number of turn lanes (also shown in Figure 3-3), all minor-road crossing traffic will have to turn from the right lane, but the lane will not be exclusive. If drivers queuing on the minor approach are given the choice among turn lanes, then the drivers making a simple left will be assumed in general to choose the shortest lane, while crossing traffic will always choose the right lane. The longest possible queue would result when all the left-turning traffic queued first, distributing semi-equally among all turn lanes, followed by the arrival of crossing traffic. In this case, there are only two queue groups: the vehicles turning from the A jughandle, and the U-turns. The latter volume and lane utilization remain the same as before. The former is modeled conservatively by allowing the non-crossing vehicles (volume $D_A - D_X$) to arrive first, assuming the highest volume to use the far-right lane, and then allowing the crossing vehicles (volume D_X) to arrive in the same lane. Therefore, in this case:

$$Q_R = \frac{26 * \max \left\{ \frac{D_A - D_X}{t(f_{LU})_A} + D_X, \frac{D_U}{n(f_{LU})_U} \right\} * C}{3600}$$

for $t = n$

The Highway Capacity Manual gives lane-utilization factors for exclusive left-turn lanes and for exclusive through lanes.

Table 3-1. Lane utilization factors [20]

Application	f_{LU}
Two Exclusive Through Lanes	.952
Three or More Exclusive Through Lanes	.908
Two Exclusive Left-Turn Lanes	.971

The lane-utilization factor for the queue following the U-turn movement, $(f_{LU})_U$, will be set to the value of f_{LU} for n exclusive through lanes. For $(f_{LU})_A$, if $t = n$, then the value for two exclusive left-turn lanes will be used. However, if $t < n$, then this value would misrepresent the ultimate distribution of the queue space's exclusive through lanes, because the right-most of these lanes will lose vehicles that enter the lane used for right turns, creating a greater imbalance than existed in the minor approach's queue. While information is not available on how drivers will behave between the turn onto the mainline and reaching the central intersection, it is assumed that striping, signage, and the knowledge that vehicles in the right lane will be executing a slower turn, should discourage those not turning right from merging into the right lane before crossing the central intersection. Still, if the queue space is long enough, drivers in the third lane from the right may merge into the second lane from the right as it loses crossing traffic to the far-right lane, reducing some of the new imbalance. The more conservative factor of .952 will be used for two through lanes, and .908 for three or more through lanes.

In addition to the required queue length, a short distance should be provided for transition from the movement into the queue; since this movement is from the minor approach, the deceleration length will be relatively small compared to that for a left-turn lane on a high-speed road. Assuming that drivers execute the left turn at Synchro's default left-turn speed of 15 miles per hour, and brake at a deceleration rate of 11.2 ft/s² upon completing the turn, a deceleration length of 25 feet should be sufficient [3].

When $t < n$, crossing traffic will most likely disregard striping and enter the right-most lane directly; however, some additional space should be provided to allow more compliant drivers to enter the right-most exclusive through lane and then merge right. An additional length of 70

feet will allow a 10-degree merge across a 12-foot lane, and a longer deceleration length should be provided before the back of the right-lane queue. If vehicles reach 25 MPH, they will then need 60 feet to decelerate [3]. Therefore, adding these buffer lengths to the previous formulas for queue length, total required length between intersections will be given by:

$$L \geq \max \left\{ \frac{26 D_X C}{3600} + 130, \frac{26 (D_A - D_X) C}{3600 t(f_{LU})_A} + 25, \frac{26 D_U C}{3600 n(f_{LU})_U} + 25 \right\}$$

for $t < n$

$$L \geq \frac{26 * \max \left\{ \frac{D_A - D_X}{t(f_{LU})_A} + D_X, \frac{D_U}{n(f_{LU})_U} \right\} * C}{3600} + 25$$

for $t = n$

where L = distance between the central intersection and an outer intersection (feet)

Based on the preceding development of estimations for queue length and needed transition lengths, the two minimum values for L above are recommended when the number of turn lanes is, respectively, less than, and equal to, the number of through lanes on the mainline.

In practice, a design would likely be symmetrical even given different values of L in each direction. Thus this chapter assumes the queue space length L to be equal in both directions, based on the higher of the two length requirements. It should be noted that the cycle length is a significant factor in all of these formulas. Because signal timing can vary the cycle length, particularly at actuated intersections, overflows resulting from deficient spacing can be prevented without reconstruction, but at the possible loss of optimal timing. Simulation modeling software, and an iterative process of optimizing cycle length within a range of queue lengths, may provide the most accurate estimate of length required for optimal operations.

3.4 FOOTPRINT OF RAMPS

The footprint of the ramps in the Type A+B jughandle will be considered to be the amount of land occupied or enclosed by the ramps. This consideration is essential, because excessive land area requirements may render a jughandle design infeasible, especially in a retrofit scenario. NJDOT's design guidelines are repeated here for reference, in Figure 3-4 and Figure 3-5, to provide some initial guidance as to the space requirements.

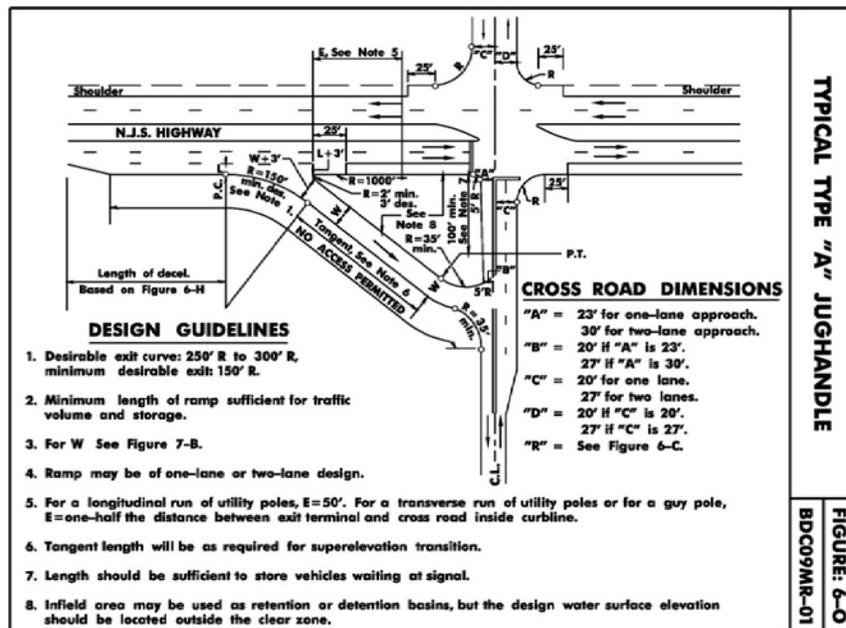


Figure 3-4. Type A jughandle design guidelines [2]

The queue space length given in the previous section will determine the approximately triangular space between the mainline and the minor street that each A jughandle consumes. As explained in Figure 3-4, in a typical A jughandle the lengths of the approaches past the divergence point are set to allow sufficient storage for traffic on each approach: neither queue of traffic from the road with the jughandle should interfere with the other. In this case, however, the length will not be constrained because there is only one approach and therefore one queue. The function of the jughandle here is simply to offset the minor-road entrance by the distance required between intersections. Therefore, the sole constraint on the geometry of this approach is that it must

facilitate a lateral movement of up to several hundred feet to the right. This allows a great deal of flexibility in the approach the ramp takes: it could curve gradually, skirt around properties, or potentially even enclose an existing property. NJDOT does not recommend permitting access along a jughandle ramp; however, the jughandle in this case is not a high-speed divergence, but is more like a quadrant roadway, and as with a quadrant connector, access could be permitted.

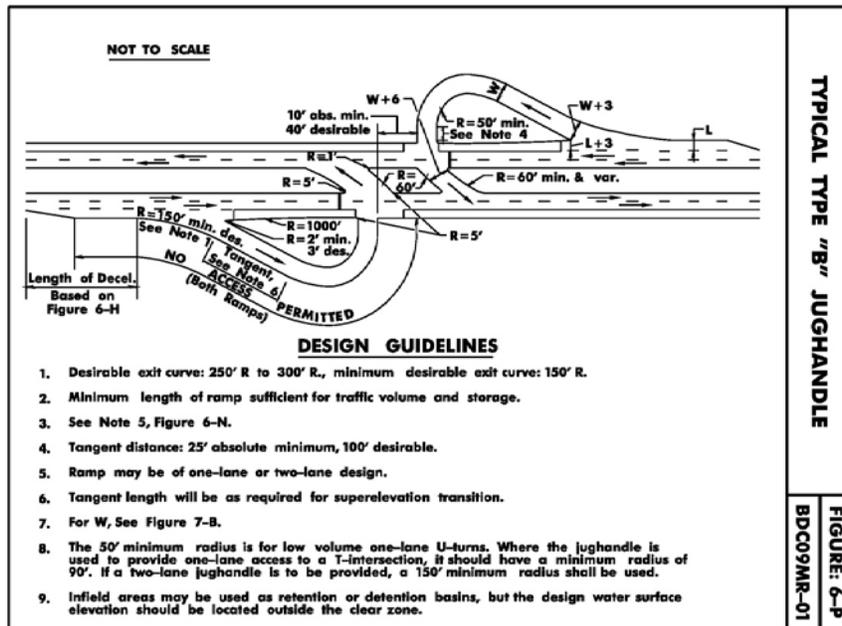


Figure 3-5. Type B jughandle design guidelines [2]

As noted in Figure 3-5, minimum ramp radius for a two-lane B jughandle is 150 feet, providing a design speed of 25 MPH for a recommended superelevation of 4-6%. Figure 3-5 also gives a desirable exit curve radius of 250 to 300 feet, or a minimum of 150 feet. The deceleration length provided will depend on the major road's speed as shown in Table 3-2.

Table 3-2. Deceleration length (interpolated) [3]

Major Road Speed (MPH)	Deceleration Length (feet)		
	Exit Radius 150 feet	Exit Radius 250 feet	Exit Radius 300 feet
30	140	0	0
35	185	145	130
40	235	180	165
45	295	245	230
50	355	310	295
55	410	375	360

As stated in Figure 3-5, superelevation transition in preparation for the ramp curve will also determine the length of the tangent section between the exit curve and the ramp curve. As opposed to the A jughandles, the B jughandles are relatively high-speed ramps and should not permit property access.

This section indicates that a large range of possible values may be used in the construction of a jughandle ramp, depending on the speed that is desired for the ramp. In order to model what is required for a high-quality ramp, the footprint for the A jughandle will be assumed to begin as far back as permitted by constraints on the right of way. The footprint for both the A and B jughandles will be calculated as triangular regions (when the B jughandle's necessary right-turn channelization is included, the entire component occupies an approximately triangular space). The B jughandle's length parallel to the mainline will be based on the assumption that it diverges just prior to the queue space. The length perpendicular to the mainline is more subject to design criteria, and thus it will be based on a measured average of this dimension at typical existing B jughandles.

This design guidance will be used to determine what footprint area is needed for the Type A+B jughandle. The simulation analysis will provide further refinement.

3.5 POTENTIAL TRAFFIC SIGNAL PHASING PATTERNS

In this research, phasing for the Type A+B jughandle will ultimately be determined through the process of optimization executed by Synchro. However, this section intends to determine a preliminary set of patterns in order to establish a basis for plans selected in simulation. Following an explanation of phasing at each individual intersection, three overall phasing patterns will be suggested as possible methods of operating the traffic signal, beginning with what is expected to be the most typical.

The operation of signals at each individual intersection consists of two phases. At the central intersection, one phase allows mainline traffic to go straight, and the other allows vehicles from the B jughandles to execute their effective left turns and U-turns (by, respectively, proceeding straight or making a protected left turn). At each outer intersection, one phase allows mainline traffic to go straight, and the other allows vehicles to make a protected left turn onto the mainline from the A jughandle. The most efficient overall phasing pattern will depend upon the specifics of volumes for each movement and the geometry of the Type A+B jughandle. Although these are referred to as three intersections, ultimately they will need to operate as one traffic signal to control queuing between them.

3.5.1 For Left Turns Determining the Queue Space Length

The first suggested phasing pattern, proposed for most cases, is depicted in Figure 3-6.

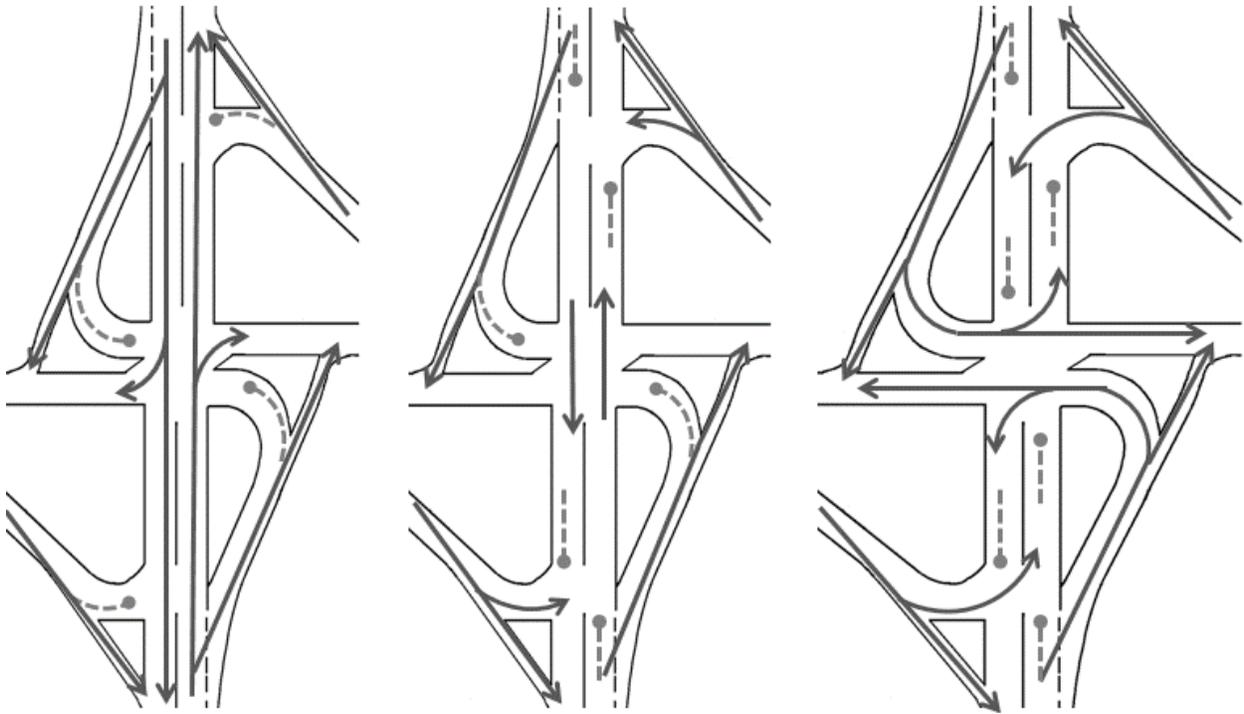


Figure 3-6. Three diagrams representing proposed typical phasing

Figure 3-6 proposes a typical three-phase operating for the Type A+B jughandle in three diagrams portraying each phase with permitted and prohibited movements at different times. Solid arrows represent permitted vehicle paths (including those that must yield to pedestrians or other vehicles) and dashed lines ending in circles represent prohibited movements. The dashed lines show where a queue may be forming at a prohibited movement. In the left-most diagram, mainline traffic proceeds and diverges, while traffic turning left onto the mainline or making a U-turn forms a queue. In the central diagram, the mainline develops a queue directed away from the central intersection when the outer intersections stop mainline traffic and mainline through traffic accumulates in the queue space. In the third diagram, U-turning vehicles proceed and add to the queue away from the central intersection, while left turns onto the mainline also proceed and form a queue toward the central intersection.

This pattern is proposed for most cases because it is expected that the distance of an outer intersection from the central intersection will most often be determined by the peak number of vehicles turning left at the A jughandle, likely a heavier peak movement than the U-turn. In this case, this movement's queue space must be cleared at the beginning of its phase, at least when the

movement is at peak (since the length requirement was based on peak conditions). This means that through traffic on the mainline should first receive a red light at the A jughandle, be allowed to clear the queue space to the central intersection, and then queue between there and the far outer intersection, in that approach's U-turn queue space. The offset between the start of these two approaches' red phases will be equal to the time required to traverse the queue area and the width of the A intersection. Allowing the A intersection to feature dual-left-turn lanes at a width of 25 feet and a 10-foot gap (including a crosswalk) between the stop bar and this intersection:

$$O = \frac{L + 35}{1.47 V}$$

where

O = offset between the central intersection and an outer intersection (seconds)

L = distance between the central intersection and an outer intersection (feet)

V = speed on the major road (miles per hour)

If the offset moves from the outer intersections toward the central intersection, then a number of vehicles will be trapped in a U-turn queue space while approaching what is, from their perspective, the second outer intersection. It is important that the number of vehicles queuing here not exceed the space unused by vehicles making the U-turn from the corresponding B jughandle. The vehicles in this space will be all those that crossed their first outer intersection's stop bar no longer before the red light than the amount of time it takes to reach their second outer intersection's stop bar. Thus the amount of time during which vehicles passing through one outer intersection will accumulate in the U-turn space by the other is the time for traversal of one outer intersection, one queue space, the central intersection, and the second queue space. Assuming the central intersection is as wide as four 12-foot lanes and a 10-foot space between B jughandles, and that the outer intersection has the width used previously, a formula can be calculated for the total time during which vehicles entering the Type A+B jughandle will be trapped in the U-turn queue:

$$\theta = \frac{2L + 93}{1.47 V}$$

where θ = time during which vehicles accumulate in the U-turn queue (seconds)

Multiplying this amount of time by the rate of vehicle arrivals at the end of the mainline phase allows a prediction of how much queue space will remain for vehicles in each direction to make U-turns:

$$U_1 = L - 26 \theta \lambda_2$$

$$U_2 = L - 26 \theta \lambda_1$$

where

U_1 = queue space remaining for vehicles from direction 1 making U-turns (feet)

U_2 = queue space remaining for vehicles from direction 2 making U-turns (feet)

λ_1 = average mainline arrival rate from direction 1 at end of phase (veh/second)

λ_2 = average mainline arrival rate from direction 2 at end of phase (veh/second)

3.5.2 For U-Turns Determining the Queue Space Length

In some implementations, particularly when the Type A+B jughandle is used as part of an RTUT scheme, L may be determined by the volume of U-turns rather than left turns at the A jughandle. For this case, the offsets required to prevent overflow of the U-turn queue spaces will essentially be the same as those required to prevent overflow of the A jughandle queue in the previous case. The only differences are the greater intersection width for vehicles crossing the central intersection, and that the progression moves to the outside rather than to the inside. Assuming, as before, a central intersection width of 58 feet:

$$O = \frac{L + 58}{1.47 V}$$

The time θ for accumulation in the A jughandle queue is equal to the offset plus the time for traversal from the outer intersection stop bar to the central intersection stop bar. Thus θ takes the same value as before:

$$\theta = \frac{2L + 93}{1.47 V}$$

3.5.3 For Mixed Controlling Volumes

The third and final case for the sequence of mainline red is that the volume of A jughandle movements will control one queue space and that the U-turn movement will control the other. Here, sequencing must allow all vehicles to clear the queue spaces in one direction, requiring mainline red to advance simply in one direction rather than inward or outward. This takes a significant amount of time and would seem to erode the benefits of the design.

This chapter has recommended options for the traffic signal phasing and for the sequencing of the mainline red through the signal when the controlling movement is at peak. The primary case considered is that the outer intersections will stop the mainline before the central intersection does, in order to reserve the queue space for left turns onto the mainline. In the second case considered, the central intersection stops the mainline before the outer intersections do, to reserve the queue space for U-turns. Practical details will likely vary with actual volumes, and offsets between the intersections may be shorter than the ideals given here, particularly when the entire length of the queue space is not needed. In the simulation used for this research, Synchro will determine the phase sequencing that minimizes overall delay.

4.0 APPROACH FOR TESTING THE HYPOTHESIS

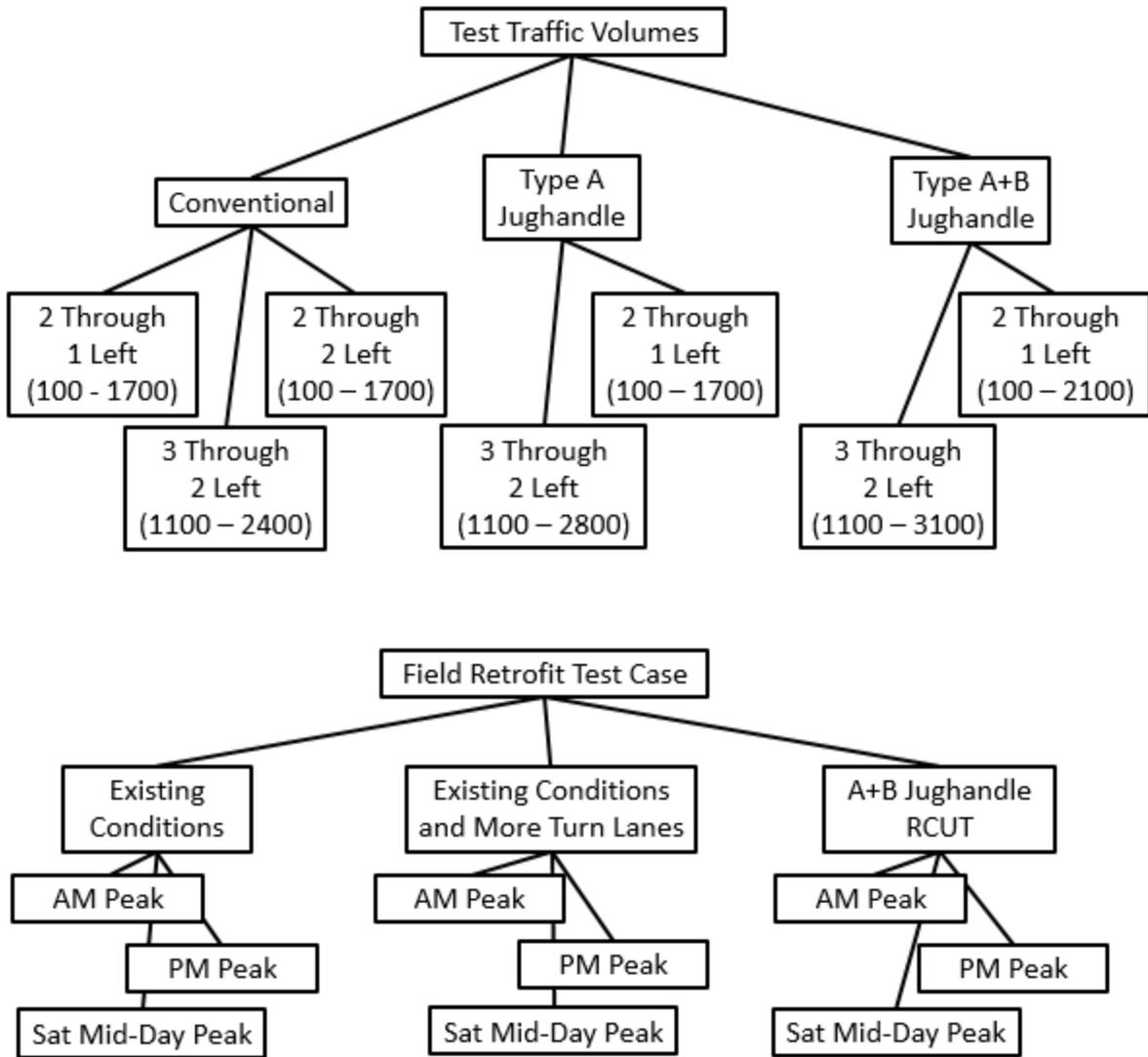
4.1 OVERVIEW OF SIMULATIONS

In order to test the hypothesis, two sequences of simulations were performed in the Trafficware programs Synchro and SimTraffic. Simulations were chosen as a methodology for their ability to measure the operational aspects of new unconventional intersections without the expense and time requirements of field testing [29]. This is an accepted method in the development of alternative intersection design and traffic signal operations. To balance the software's random generation, ten 60-minute runs, with a 10-minute seeding period, were performed, monitored, and averaged for each simulation. Figure 4-1 summarizes the test sequences performed, listing the independent variables for each. Table 4-1 summarizes all variables and constants involved in the experiments, which are discussed in more detail in the descriptions of each test. In both tests, the performance measures considered relevant for testing the hypothesis are average per-vehicle delay, fuel used, number of stops, and intersection footprint. The importance of these measures is discussed at the end of this chapter.

The first set of simulations is referred to as the **test traffic volumes**, and used a pattern of turning movements representative of the type of corridor described in the hypothesis. A student version of Synchro and SimTraffic 7 was used for this test. The purpose of this test was to approximate the magnitude of traffic volumes and mixture of turn movements for which a Type A+B jughandle working in isolation could achieve superior performance as compared to a conventional intersection or a Type A jughandle.

The second set of simulations, referred to as the **field retrofit test case**, used data from a three adjacent intersections in western Pennsylvania to model the impact of replacing the central intersection with an RCUT and the outer two with Type A+B jughandles. Synchro and SimTraffic 8 were used for this test case. The purpose of this test was to demonstrate the potential

advantage of retrofitting an existing corridor with Type A+B jughandles and a median barrier, and eliminating a traffic signal. The corridor was selected for its similarity to the conditions described in the hypothesis, and for the availability of recent traffic volume data following a widening project to install a two-way left-turn lane.



In the Test Traffic Volumes chart, numbers in parentheses represent the range of volumes of x, as described in the next section

Figure 4-1. Independent variables for all tests

Table 4-1. Constants and variables in the experiments

Test Traffic Volumes	
Independent Variables	Magnitude of demand, number of through/left-turn lanes, type of intersection (conventional, Type A, Type A+B)
Constants	Turning movement proportions, speed, other default Synchro factors
Dependent Variables	Left-turn phasing, cycle length, required spacing, footprint, per-vehicle delay, fuel used, number of stops
Field Retrofit Test Case	
Independent Variables	Time of day and week, configuration of the network (conventional, more turn lanes, or RCUT)
Constants	Speed, other corridor-specific factors
Dependent Variables	Left-turn phasing, cycle length, required spacing, footprint, per-vehicle delay, fuel used, number of stops

4.1.1 Determination of Cycle Length

The cycle length must be known in order to determine the length of the queue space in the Type A+B intersection and run the simulation, but it is ultimately selected during a computer optimization process that assumes a certain input of queue space length. Thus an iterative process was used to determine the queue length required. Webster’s formula was used for the initial calculation of cycle length, after which queue space length was determined for the initial optimization using this cycle length. Queue space length was then increased and decreased until the network delay calculation reached its smallest observed value. The final chosen cycle length roughly minimized delay among all intersection lengths checked, and the final chosen intersection length was the shortest length that allowed this near-minimum delay, without causing the intersection to exceed its footprint constraints. The cycle length range was set to a minimum of 40 seconds and a maximum of 150 seconds. Cycles for the Type A+B jughandle were generally around 60 to 80 seconds in the less congested simulations, with a rapid rise to 150 seconds as the intersection approached failure.

4.2 DEVELOPMENT OF TEST TRAFFIC VOLUMES

4.2.1 Proportion Model

Based on available turning movement count data from the corridor in western Pennsylvania and professional guidance on the expected ratios of turn volumes to mainline through traffic, a simple model of turn proportions based on the conditions described in the hypothesis was developed. Turning movement counts were performed by the Pennsylvania Department of Transportation in 2002 along the U.S. Route 19 corridor, a suburban arterial in Wexford Flats, north of Pittsburgh. For the model, four signalized four-approach right-angle (“plus”) intersections were selected and their turn volumes were averaged to represent a typical candidate intersection. Counts were taken in the weekday AM peak hour, mid-day peak hour, and PM peak hour; in the Saturday mid-day peak hour, PM peak hour, and evening peak hour; and in the Sunday mid-day peak hour. Volumes were generally highest during the weekday PM peak, followed by the Saturday mid-day peak. PM peak hour volumes were selected for the model.

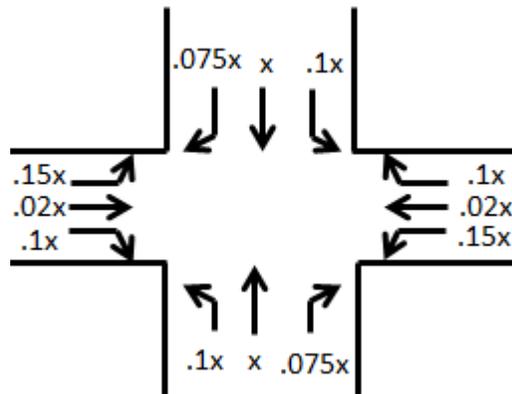


Figure 4-2. Model of turn volume proportions

Figure 4-2 depicts the model, in which x represents the volume of through traffic on the mainline passing through the intersection. The model assumes symmetry to ensure that the magnitude of volume is the only independent variable being changed. The major-approach left turns are given volume $.1x$, and the major-approach right turns $.075x$. The minor-road through movement

volume is $.02x$, its left-turn volume is $.15x$, and its right-turn volume is $.1x$. This method of testing allows the determination of an optimal magnitude of volumes, rather than an optimal set of individual volumes with too many characteristics and interactions to measure.

While the ability to execute safe U-turns is a benefit of both the Type A and the Type A+B jughandle, these were not considered in the traffic volumes test, because it is difficult to approximate a typical demand for U-turns without origin and destination information, which was not available. This capacity can be considered an unquantified benefit of the jughandles, particularly the Type A+B jughandle.

4.2.2 Number of Lanes

The number of through and turn lanes in the simulation ranged across a set of values typical on a high-volume arterial. This served two purposes: first, it would allow the results to represent a larger number of actual conditions, and second, it would allow a comparison between the degrees of improvement the Type A+B jughandle created given different numbers of lanes.

Each type of intersection was first tested with 2 mainline through lanes in each direction and a single left-turn lane wherever left-turn lanes were present; each was then tested with 3 through lanes and 2 left-turn lanes where left-turn lanes were present. Additional testing on the conventional intersection tried 2 through lanes with 2 left-turn lanes on the mainline only, as well as 2 through lanes with 2 left-turn lanes from all approaches. In all tests, the conventional intersection's minor approaches featured a shared lane for through and right-turn traffic. A right-turn channelization onto the mainline was provided in the test with 3 through lanes for the conventional intersection. (Because the channelization tended to cause stuck vehicles on the jughandles, they used it only for the very highest volumes.) In its first test, the Type A jughandle's minor approaches featured a shared lane for through and right-turn traffic, but in its second test there was additionally an exclusive through lane. Since the ramps were unsignalized for the Type A jughandle, only one left-turn lane from the mainline was provided, even in the second test. In order to reduce the number of variables, all lanes on a link were allowed to occupy the entire link, rather than beginning at a set distance from the intersection; actual footprint was later calculated based on the 95th percentile queue for an approach.

4.2.3 Magnitude of Volumes

The proportions of turning volumes in the model remained constant, while the magnitude x was raised by intervals of 100 vehicles per hour, proceeding until level of service D could no longer be provided. LOS is defined using network delay per vehicle and the standard HCM limits of 10, 20, 35, and 55 seconds for A, B, C, and D, respectively. The cut-off of LOS D as a minimum was selected based on typical design standards in urban areas, and because of the instability of operations at LOS E; the simulations indeed showed that LOS E was generally the point at which congestion became too heavy for the performance measures to be observed accurately.

The 2-through-lane tests set the initial value of x to 100 vehicles per hour; the 3-through-lane test began with $x = 1100$ vehicles per hour. The performance measures for each of the three types of intersections were compared across all values of x that yielded an acceptable level of service.

Table 4-2 shows the values of x used in the tests, where **T** stands for “through lanes” and **L** stands for “left-turn lanes.”

Table 4-2. Values of x used in the traffic volume tests

Mainline Through Volume (x)	Conventional			Type A		Type A+B	
	2T 1L	2T 2L	3T 2L	2T 1L	3T 2L	2T 1L	3T 2L
100							
200							
300							
400							
500							
600							
700							
800							
900							
1000							
1100							
1200							
1300							
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2100							
2200							
2300							
2400							
2500							
2600							
2700							
2800							
2900							
3000							
3100							

Delay for the Type A and Type A+B jughandles depends to a large extent upon how much space is allocated for queuing. Thus, to control this variable, the footprint for the entire intersection was limited to 1000 feet by 1000 feet

4.2.4 Other Inputs

The distance between the ramps and intersection for the Type A jughandle was determined in a similar way as that for the Type A+B jughandle: the maximum expected queue length on the minor road (including major-road left turns, minor-road through traffic, and minor-road right turns) and a deceleration length. However, it was found that Type A jughandles were more sensitive to variations in demand, so the distance was increased when the simulation showed that demand exceeded capacity.

The simulation included 300 feet of additional mainline in either direction in order to capture more effects of the intersection. All performance measures were taken for the entire network, to ensure that the same start and end points were measured for each type of intersection. Mainline speeds were set to 45 MPH and minor approach speeds to 30 MPH.

4.3 FIELD RETROFIT TEST CASE

4.3.1 Context of Data Source

The same U.S. 19 corridor used for the test traffic volumes provided data for the test case. Specifically, three intersections were selected based on their land use context.

The northernmost intersection, Brooktree Road/Brooker Drive, provides access on the west to a large group of businesses on a loop that does not connect to any through roads and has no other left-turn access with the arterial. On the east, it primarily provides access to a very large plaza including a grocery store and around 600 parking spaces. The central intersection, North Meadows Drive, provides access on the west to a couple businesses, but mostly apartments on a cul de sac; on the east it serves as the main entrance to the aforementioned plaza. The southernmost intersection, Richard/Reichold Road, provides access to some businesses and mostly apartments on the west, but also continues into a suburban residential area and does not terminate at a dead end. On the east, it similarly enters a suburban residential area and provides access to other similar roads.

The corridor was recently widened to provide a two-way left-turn lane, and does not feature dual left-turn lanes on the mainline. Its speed limit is 35 miles per hour.

4.3.2 Conditions Compared to the Baseline

In this simulation, the signal at the intersection with North Meadows Drive was removed and a median barrier was presumed to have been erected. The signals at Brooktree/Brooker and Richard/Reichold were each replaced by the three-part Type A+B jughandle, with spacing determined by peak demand volume. Left-turn and crossing movements from the North Meadows Drive approaches, as well as left-turn movements to North Meadows Drive, were replaced with RTUT movements at the corresponding jughandles, with the majority of this traffic being redistributed through the plaza connections to choose different entrances and exits. Left turns at intersections accommodated by the existing two-way left-turn lane were counted, but were not found to have a significant impact on the performance measures.

Performance measures for the RCUT configuration were compared to the baseline condition of conventional intersections as they are today. An alternate upgrade was also tested, in which all conditions were kept as is but all shared turn lanes were separated, each with a storage length of 200 feet. In this test, a second left-turn lane was added on all approaches with high numbers of left turns. All three configurations were tested using three sets of data: the AM weekday peak, PM weekday peak, and Saturday mid-day peak.

4.4 LIMITATIONS OF THE MODEL

As with any traffic simulation tool, Synchro and SimTraffic present a number of limitations. Because the central intersection of the Type A+B jughandle essentially features traffic driving on the wrong side of the road, it must be modeled as two very closely spaced intersections, which leaves some mainline traffic trapped and blocking the B jughandles; thus, in the simulation, the spacing between the two B jughandles was increased and modeled as another link to prevent blockage. Following this change, blockages were rare and had minimal impact on the results.

Origin-destination weighting could be used for the Type A jughandle, but not for the minor-road crossing traffic on the A+B jughandle. However, a very small proportion of vehicles used this movement. A brief sensitivity analysis suggested that weighting would not make a significant impact on any performance measures.

While the ramps in the simulation intersect the mainline at the appropriate locations, their curvature was not modeled precisely; this was found to have no significant impact on performance measures output by the program. In order to reduce the number of intersections, the mainline right turn channelizations were laid out to merge into the minor road at the same intersection where the A jughandles diverge: this too should have minimal impacts on outputs. All diverges were treated as unsignalized intersections with all movements free; the merges are treated as unsignalized intersections with yield control for only the merging movement.

Once a queue exceeds the Synchro network, outside vehicles are not considered. Thus, results for intersections very close to failure may slightly underestimate the severity of delay, stops, and fuel used. For the test traffic volumes, this issue was found to occur only rarely.

4.5 PERFORMANCE MEASURES CHOSEN

The performance measures were selected based on the outputs provided by SimTraffic and based on general goals for intersection operations and design.

Delay is the primary numerical measure of how efficiently an intersection is serving vehicle arrivals. Reducing delay is a major goal of introducing an unconventional intersection. Per-vehicle delay was selected as the output to be measured for several reasons. First, it helps to control for variation in the number of vehicles randomly generated by SimTraffic. Secondly, it allows for a more accurate measurement of the increase in congestion as the number of vehicles increases, since pure delay naturally increases with the number of vehicles. Finally, per-vehicle delay determines intersection level of service, which is also the upper limit on x in the experiment.

Fuel used allows the measurement of how efficiently vehicles operate at an intersection, given time spent idling, the amount of acceleration and deceleration, and the distance traveled. Reducing fuel usage has benefits in the environmental, economic, and political spheres. Reducing delay may reduce fuel used during idling, but this must be balanced against longer paths traveled in unconventional intersections. Because jughandles involve longer paths for left turns, it is important to quantify their net impact at different volumes.

The number of stops counted during the simulation provides a measurement of quality for users of the intersection. Stops consume more fuel and put more wear on the vehicles than reduced speeds. Splitting an intersection into several intersections can increase the number of stops, but if it provides a large enough operational benefit, the net number of stops may decrease.

Footprint is a very significant disadvantage of jughandles, particularly the Type A+B jughandle. Selecting additional footprint, defined as the amount of land consumed by non-through lanes, as a performance measure allows a comparison of the land required for jughandle ramps to the land required for left-turn lanes. Additional footprint was selected rather than overall footprint, both for this reason and because it is difficult to define the exact ends of an intersection. Because the length of a left-turn lane is based partly on queue length, a conventional intersection may see a large increase in footprint when it reaches failing volumes; still, even a jughandle that performs very well will have a greater footprint.

5.0 TEST RESULTS

5.1 TEST TRAFFIC VOLUMES

5.1.1 Introduction

The results presented for the test traffic volumes are divided into two subsections: the tests in which two through lanes and one left-turn lane were provided, and the tests in which three through lanes and two left-turn lanes were provided.

Results for the test on the conventional intersection with two through lanes and two left-turn lanes from the mainline were omitted because they all increased delay compared to a single left-turn lane, and are not as representative of a typical arterial intersection design. Another test provided two left-turn lanes from the mainline and from the minor road, and saw a further increase in delay. Since these tests presented only disadvantages compared to two through lanes and one left-turn lane, the latter makes a fairer comparison to unconventional intersections. Reasons for the unexpected increase in delay are discussed in greater depth in the next chapter.

5.1.2 Two Through Lanes and One Left-Turn Lane

Results for all three intersection types in the test traffic volumes tests involving two through lanes and one left-turn lane are presented in Table 5-1 and Figure 5-1 through Figure 5-4. In these tests, additional footprint is defined as all land occupied by roadways and/or enclosed by ramps that is not occupied by the through lanes on the major or minor approach. Thus, additional footprint for the conventional intersection includes turn lanes, whose length was determined by the 95th percentile queue and the required deceleration length and taper.

Table 5-1. Performance measures for two through lanes and one left-turn lane, at given volumes of mainline through traffic (x)

x	Delay (seconds)			Fuel (gallons)			Stops			Footprint (1000 ft ²)		
	Conv	A	A+B	Conv	A	A+B	Conv	A	A+B	Conv	A	A+B
100	6.4	7	11.4	2.5	2.7	3.8	121	155	163	19.3	38.9	95.3
200	7	7.1	11.6	5.2	5.4	7.9	247	296	351	19.6	38.9	95.6
300	7.9	7.8	12.4	8.7	8.2	11.9	404	465	514	19.7	38.9	96.2
400	8.7	8.2	12.6	11.8	11.1	16.2	557	619	685	19.9	38.9	96.7
500	9.4	8.7	13.4	15	14.1	20.3	696	781	874	20.0	38.9	97.1
600	10.5	9.3	13.8	18.6	17.1	24.5	885	969	1010	20.2	38.9	98.1
700	11.5	10.2	14.5	22	20.4	29	1070	1166	1203	20.4	38.9	98.6
800	11.3	11.1	15.2	24.8	23.8	33.4	1129	1390	1390	20.5	38.9	99.1
900	12.6	12	15.7	28.2	27	37.1	1325	1606	1494	21.0	38.9	100.5
1000	17.2	13.4	16.5	32.4	30.7	41.4	1798	1890	1674	21.3	41.7	101.1
1100	17.5	15.5	16.7	35.5	34.3	45.5	1914	2286	1832	22.2	43.9	100.7
1200	19.7	14.6	17.8	39.4	36.4	50	2236	2155	2041	22.4	53.2	102.3
1300	22.8	16.1	18.9	43.7	40.7	54.5	2668	2497	2246	22.6	55.9	102.9
1400	24.1	18.8	21.4	47.1	44.9	59	2596	2933	2486	24.0	58.9	104.9
1500	29.1	24.8	21.9	52.1	51.4	63.9	3102	3766	2788	24.0	66.0	104.2
1600	44.1	58.5	24.6	61.6	64.8	68.1	4570	4525	3015	24.0	88.0	115.5
1700	69.8		26.4	73.8		72.8	5086		3283	24.0		120.8
1800			30.2	122		78.3	5024		3650	24.0		138.0
1900			38			84.8			4451			149.0
2000			43.4			89.2			4770			149.9
2100			73.3			104.4			6671			148.9

Table 5-1 shows steady increases in delay for all intersections as x increases, with the conventional intersection crossing into LOS E between 1600 and 1700 vehicles per hour, the Type A jughandle between 1500 and 1600 vehicles per hour, and the Type A+B jughandle between 2000 and 2100 vehicles per hour. This demonstrates that the Type A+B design has an advantage of higher capacity. For each intersection, fuel used and stops each increase at accelerating rates. The footprint remains fairly steady in the conventional intersection, reaching the right-of-way limits at 24,000 square feet, while it increases quickly for the Type A jughandle and the Type A+B jughandle. The Type A+B reaches its limit around 150,000 square feet. Figure 5-1 through Figure 5-4 depict each performance measure graphically as a function of x . Footprint is always much higher for the jughandles, particularly for the Type A+B jughandle. This demonstrates that the A+B design requires a larger area than a conventional or Type A jughandle to provide the same level of effectiveness.

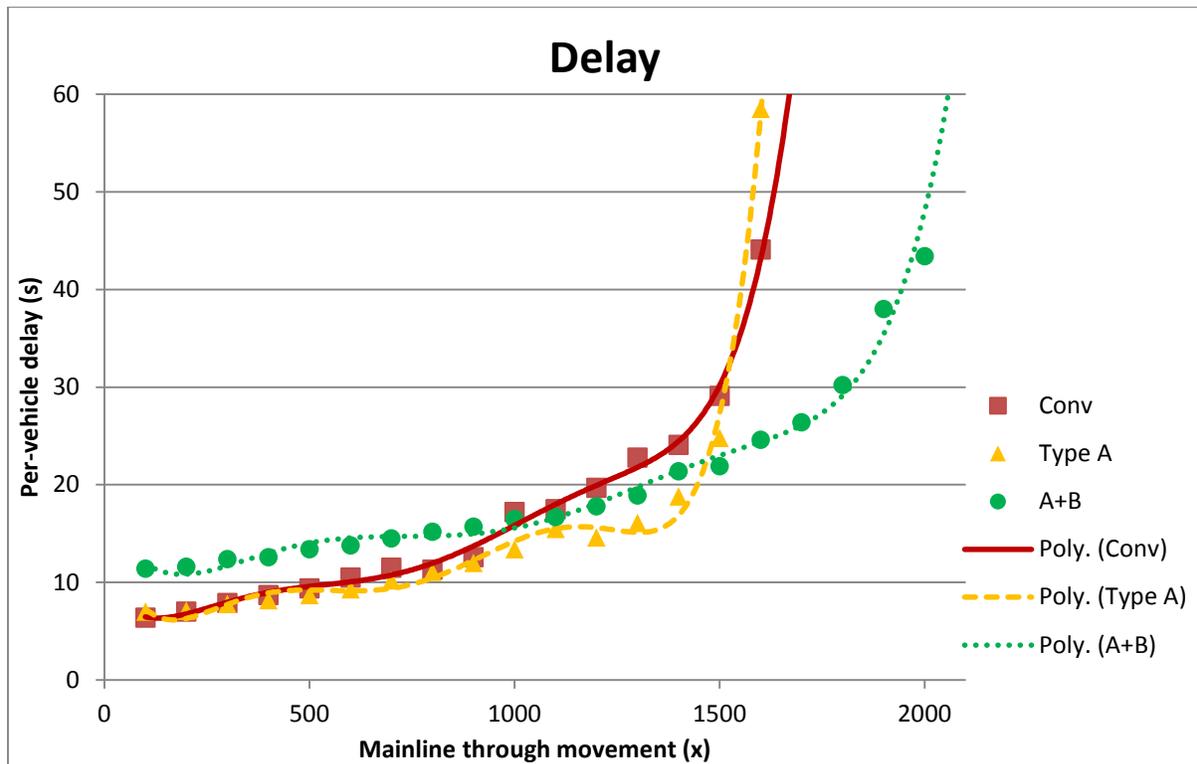


Figure 5-1. Delay for two through lanes and one left-turn lane

Figure 5-1 again shows the point at which each intersection reaches LOS E, here represented by vertically crossing 55 seconds. Delay at each intersection accelerated very quickly at this point.

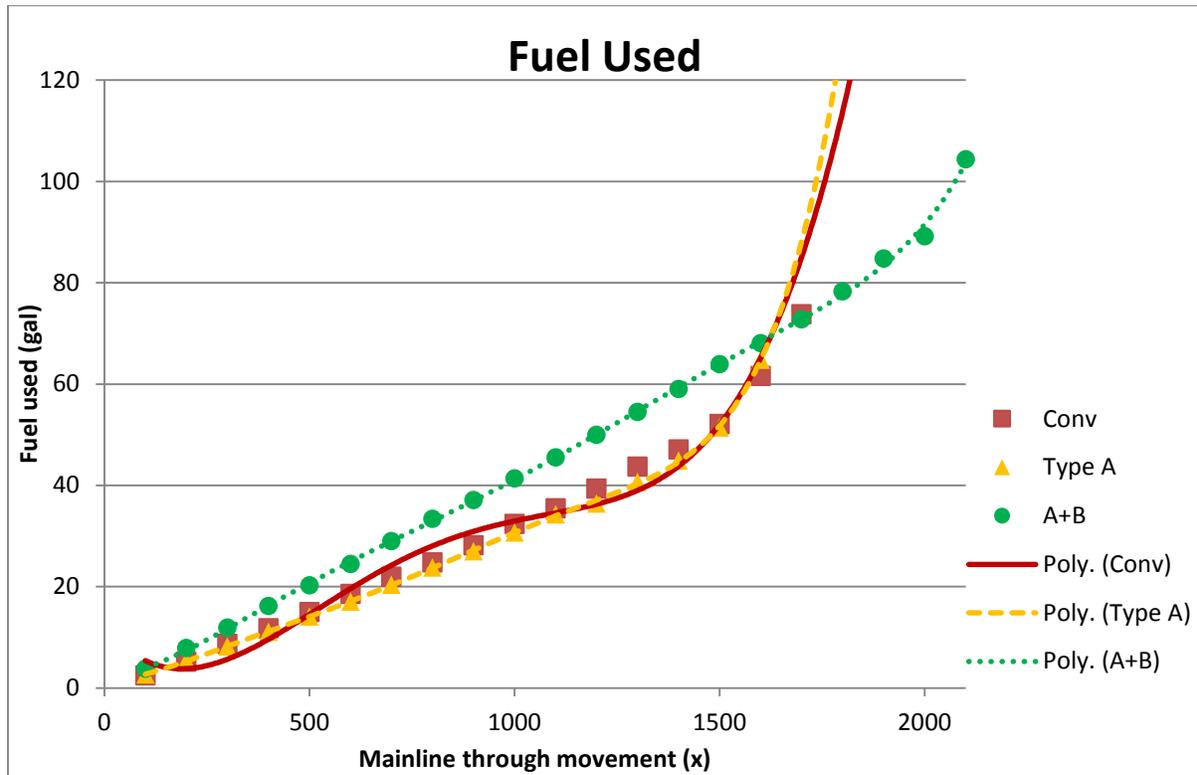


Figure 5-2. Fuel used for two through lanes and one left-turn lane

Fuel use in both the conventional and Type A intersections was linear and essentially the same; it spiked at the same x -value of approximately 1500 vehicles per hour. For the A+B jughandle, fuel use was linear but slightly higher than the other intersections, until their spike at 1500. It continued linearly until 2000 vehicles per hour.

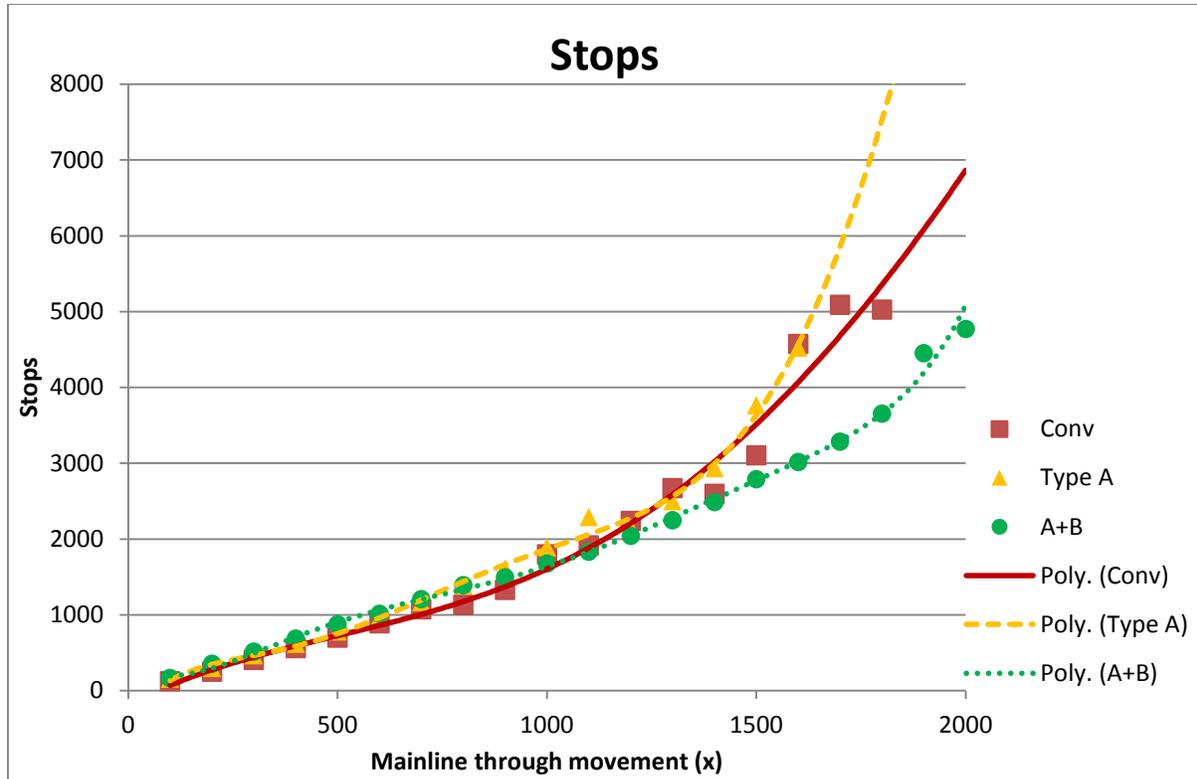


Figure 5-3. Stops for two through lanes and one left-turn lane

The number of stops followed a largely linear path, with all three intersection types sharing a very similar value until the A+B value became the lowest of the three, beginning around 1200 vehicles per hour.

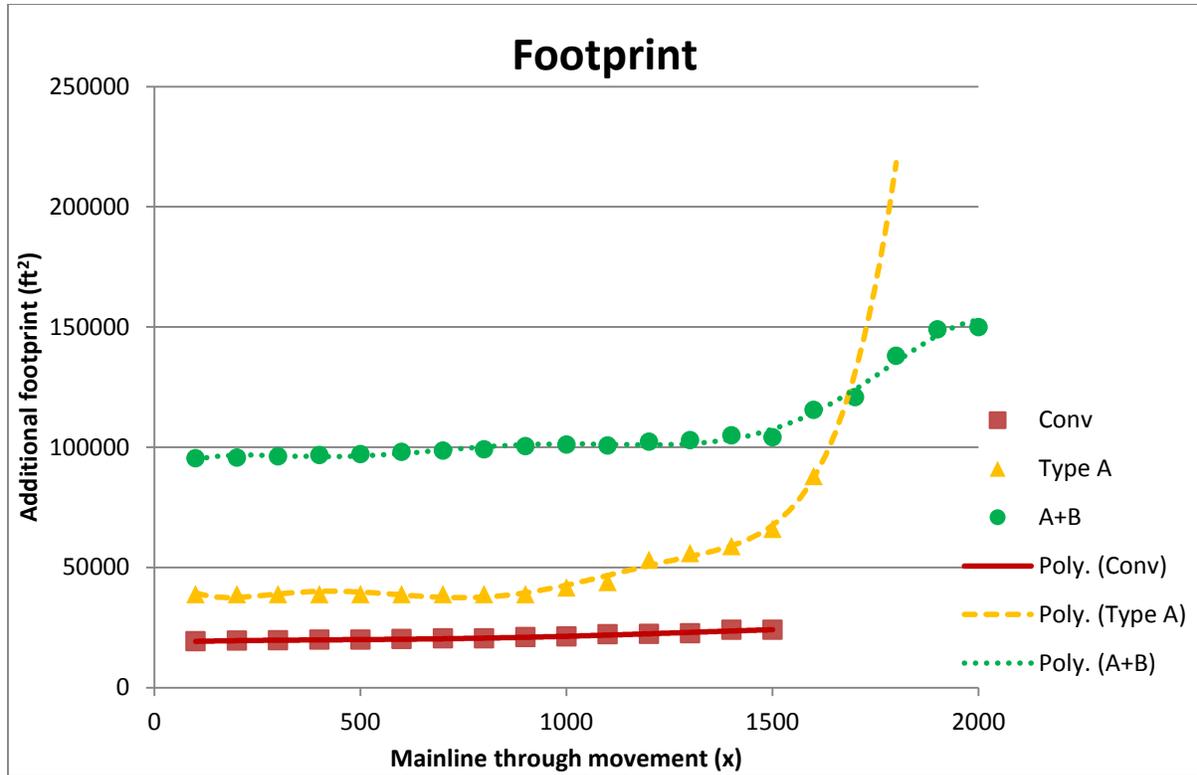


Figure 5-4. Additional footprint for two through lanes and one left-turn lane

Additional footprint was largely constant for the conventional intersection, at 19,000-24,000 square feet. The Type A jughandle required around 39,000 square feet at minimum and the requirement accelerated rapidly beginning around 1500 vehicles per hour. The Type A+B jughandle required approximately 100,000 square feet until 1500 vehicles per hour, when it spiked before reaching the boundary limits at 150,000 square feet.

5.1.3 Three Through Lanes and Two Left-Turn Lanes

Results for the test traffic volumes tests involving three through lanes and two left-turn lanes on all approaches with left-turn lanes are presented in Table 5-2 and Figure 5-5 through Figure 5-8.

Table 5-2. Performances measures for three through lanes and two left-turn lanes, at given volumes of mainline through traffic (x)

x	Delay (seconds)			Fuel (gallons)			Stops			Footprint (1000 ft ²)		
	Conv	A	A+B	Conv	A	A+B	Conv	A	A+B	Conv	A	A+B
1100	22.1	15.8	14.4	38.1	34.2	45.2	2176	2221	1869	42.7	41.3	105.8
1200	24.2	16.8	14.6	42.2	37.7	49.4	2516	2484	2039	43.0	41.3	106.3
1300	26.4	18	14.9	46.3	41.9	53.8	2756	2818	2227	44.5	41.3	106.9
1400	33.7	17.5	15.6	52.5	44	57.9	3735	2800	2327	43.8	41.3	108.7
1500	28.5	19.4	15.7	53.8	48.8	62.3	3270	3232	2579	45.4	41.3	107.9
1600	35.3	19.7	15.8	60	51.6	66.3	4081	3319	2742	46.0	41.3	108.5
1700	31.9	21.4	16.9	62	56.6	70.4	3849	3737	2852	45.8	41.3	110.6
1800	33.9	23	17.3	66.3	61.4	74.9	3990	4136	3019	48.3	45.8	111.2
1900	43.3	24.8	18.2	74.5	66.7	79.7	5305	4653	3265	47.7	47.3	111.8
2000	38.6	30.1	18.7	75.3	71.5	82.3	4649	5379	3280	48.5	55.3	114.2
2100	47.7	29.1	19.4	83.5	75.4	88.3	5684	5345	3695	49.3	57.1	113.1
2200	47.5	28.5	20.4	87.6	77.4	92	5859	5301	3908	49.4	62.8	113.7
2300	53.6	30.6	22.2	94.3	83.1	96.8	6147	5713	4127	49.4	60.7	116.4
2400	61.4	30.5	23.8	102	85	101.7	6699	5308	4406		66.7	117.1
2500		34.4	25.5		92.1	106.2		6030	4797		93.5	117.9
2600		41.7	27.9		100.2	111.3		7036	5119		96.5	120.9
2700		48.4	29.6		110.7	115.5		7072	5273		126.3	130.4
2800		56.9	32.6		122.7	125.7		7474	5743		145.0	143.5
2900			38.5			135.2			6094			147.3
3000			42.9			140.7			6609			156.6
3100			56.8			144.5			8211			158.0

As seen in Table 5-2, LOS D can be achieved for up to 2300 vehicles per hour with the conventional intersection, between 2700 and 2800 vehicles per hour for the Type A jugghandle, and between 3000 and 3100 vehicles per hour for the Type A+B jugghandle. This again demonstrates the capacity advantage of the Type A+B jugghandle in high-volume corridors. As in the previous test, fuel and stops rise throughout, and rise more quickly as the intersections approach failure. Footprint is again much higher for the jugghandles, with the Type A jugghandle reaching 145,000 square feet before failure and the Type A+B reaching a limit around 158,000 square feet. Figure 5-5 through Figure 5-8 provide graphs of each performance measure as a function of x .

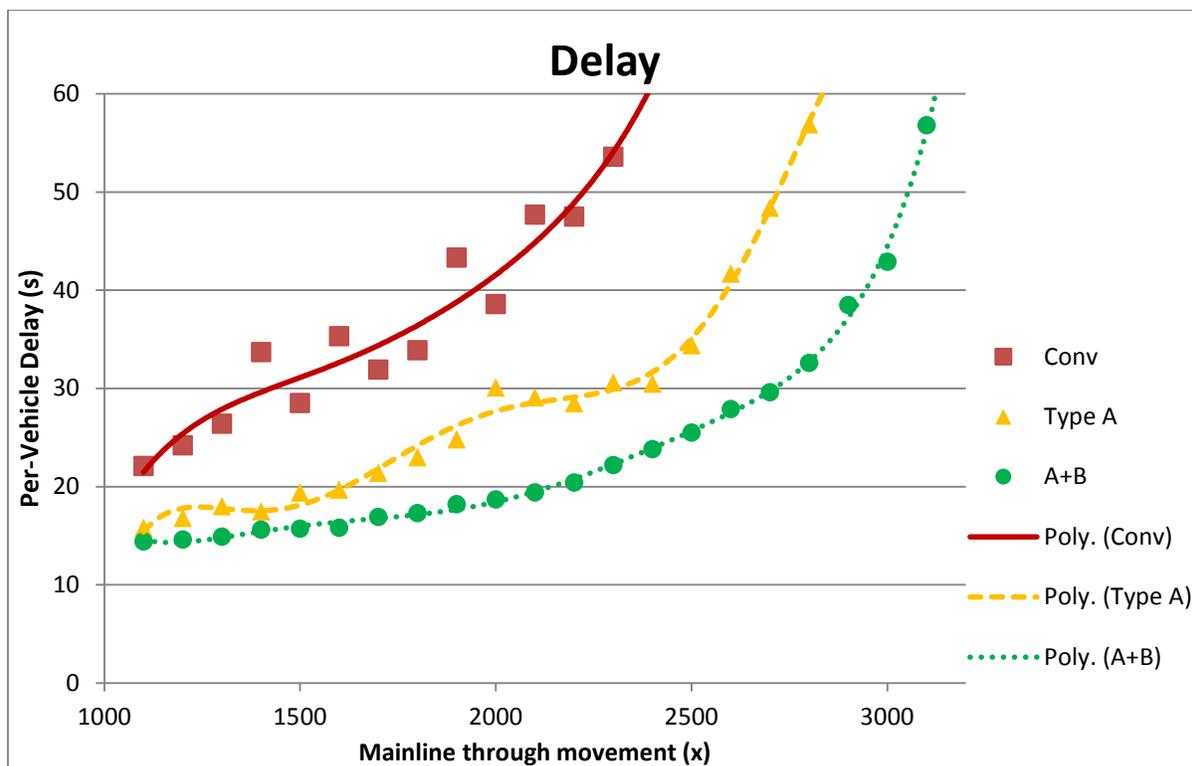


Figure 5-5. Delay for three through lanes and two left-turn lanes

Figure 5-5 again shows the delay at each intersection increasing as x increases, with LOS E at the vertical crossing of 55 seconds. As opposed to the sharp jumps in delay seen in Figure 5-1, here each intersection featured a more gradual acceleration of delay even near failure.

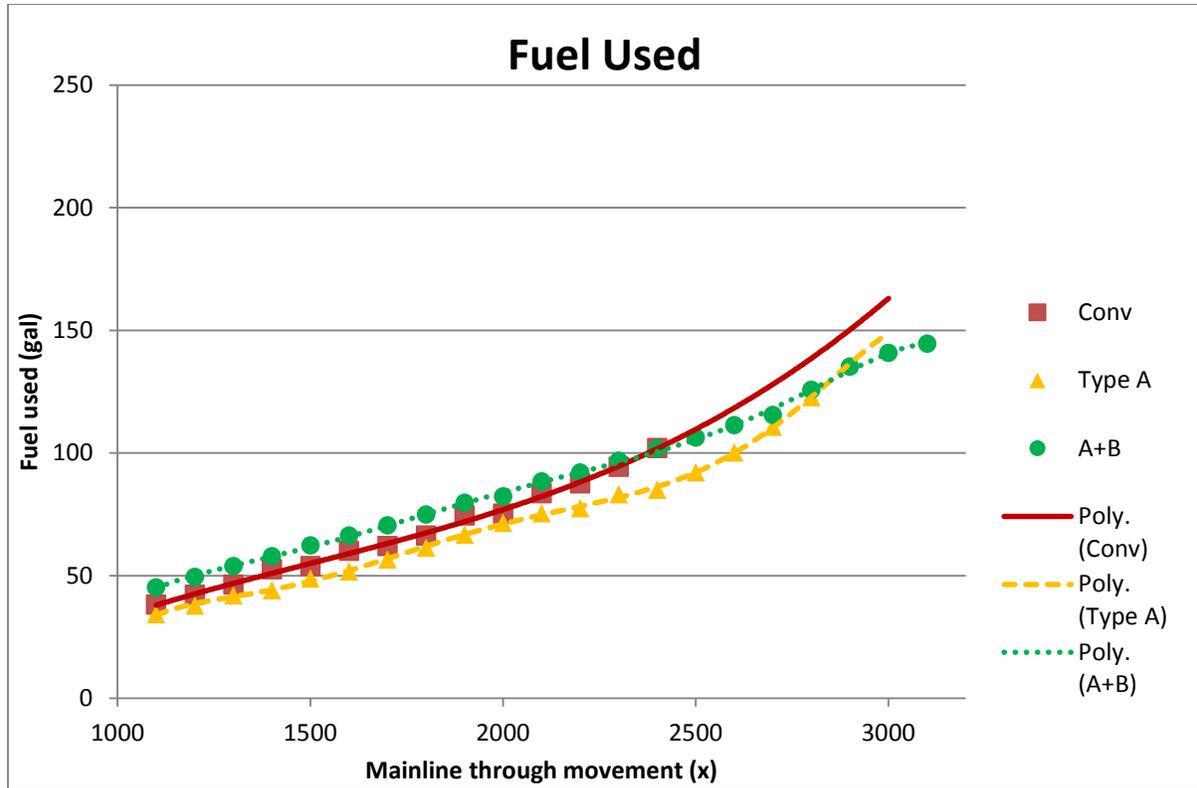


Figure 5-6. Fuel used for three through lanes and two left-turn lanes

Fuel use followed a mostly linear path, with Type A slightly lower than conventional, and Type A+B slightly higher than conventional until being surpassed by it around 2400 vehicles per hour, and by Type A around 2800.

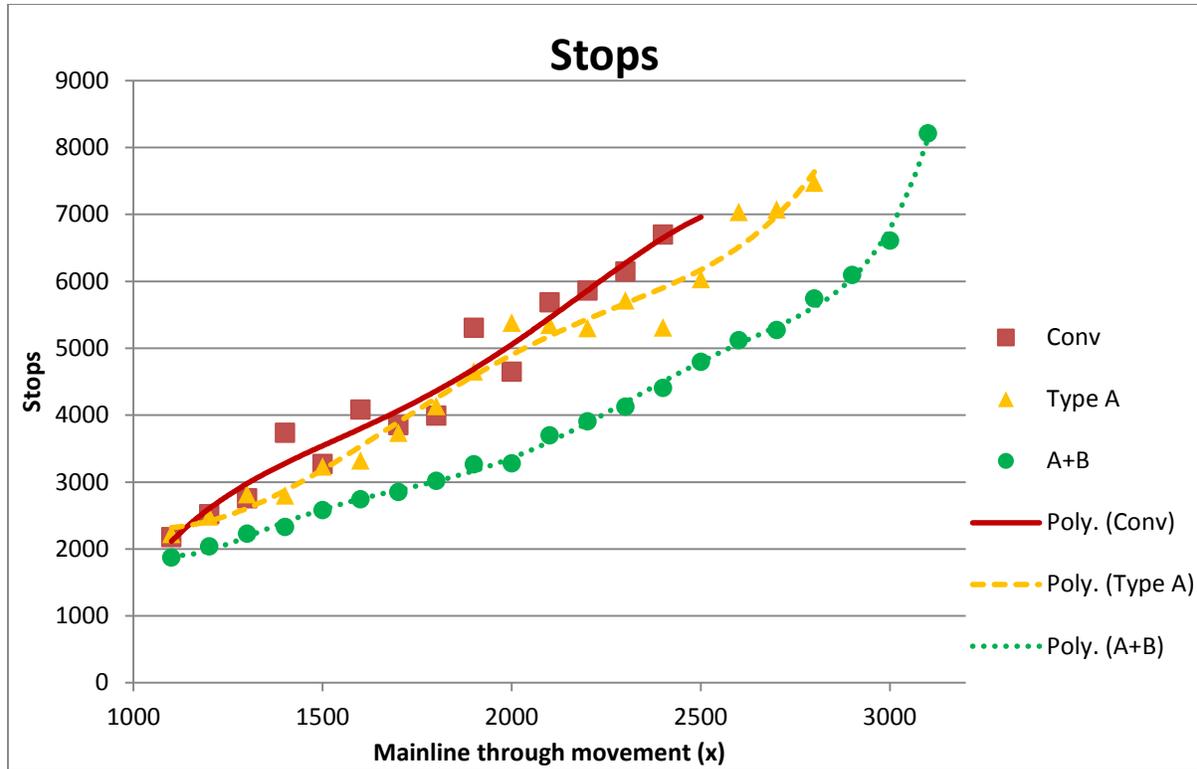


Figure 5-7. Stops for three through lanes and two left-turn lanes

The number of stops was mostly linear until peaking when the intersections reached LOS E, with Type A always slightly lower than conventional and Type A+B always moderately lower.

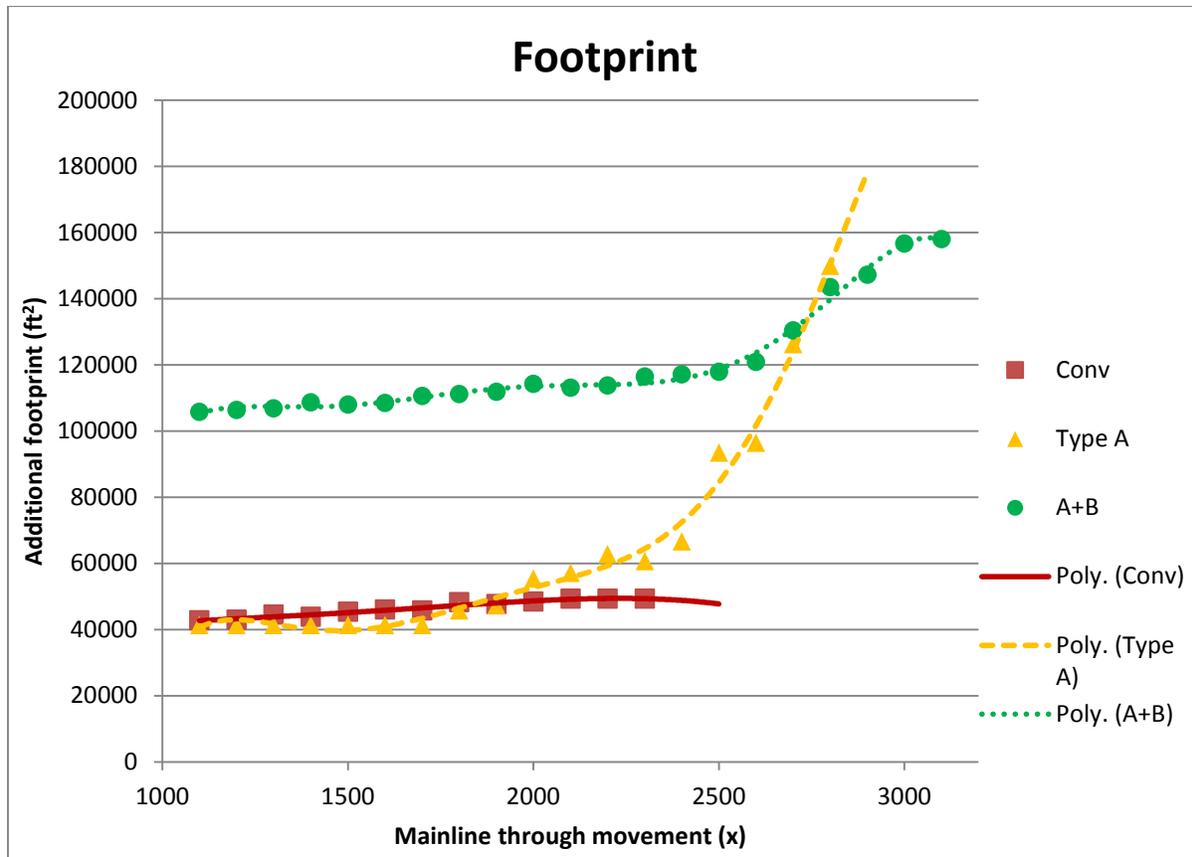


Figure 5-8. Additional footprint for three through lanes and two left-turn lanes

Additional footprint for the conventional intersection was again nearly constant at 43,000-49,000 square feet. Type A started at 41,000 square feet and spiked sharply around 2400 vehicles per hour. The polynomial trend line is inaccurate past a point, because of the limits on the footprint that had not yet been reached. Type A+B began at 106,000 square feet, began to accelerate around 2600 vehicles per hour, and reached its boundary limit at 158,000 square feet.

5.2 FIELD RETROFIT TEST CASE

Results for the field retrofit test case are presented in Table 5-3 and Figure 5-9 through Figure 5-11. The additional footprint needed for the field retrofit test case was found to be 85,600 square feet.

Table 5-3. Performance measures for the field retrofit test case

		Network Delay (seconds/veh)		Fuel (gallons)	Stops	Change in Delay
Weekday AM Peak	As-Is	37.8		63.8	2310	0
	More Turn Lanes	37.5		63.1	2745	-0.8%
	A+B RCUT	20.1		73.7	2342	-46.8%
Weekday PM Peak	As-Is	57		94.3	4331	0
	More Turn Lanes	60.2		92.3	4038	+5.6%
	A+B RCUT	28.5		105	2736	-50%
Saturday Mid-Day Peak	As-Is	43.5		80.9	3091	0
	More Turn Lanes	61.9		84.4	3255	+42.3%
	A+B RCUT	24.2		94.2	3233	-44.4%

While the per-vehicle network delay was not unacceptable for existing conditions, the RCUT configuration reduced it by 46.8% in the weekday AM peak, 50% in the weekday PM peak, and 44.4% in the Saturday mid-day peak, for a total reduction of 47.4% among the three peaks. Visual representations of the data follow in Figure 5-9 through Figure 5-11.

The additional footprint required for the RCUT configuration, including the land savings of removing intersection left-turn lanes and the two-way left-turn lane, was estimated at 85,600 square feet, though this would vary with design standards. Some properties might be enclosed by the A jughandle ramps rather than removed, which would significantly reduce the land required.

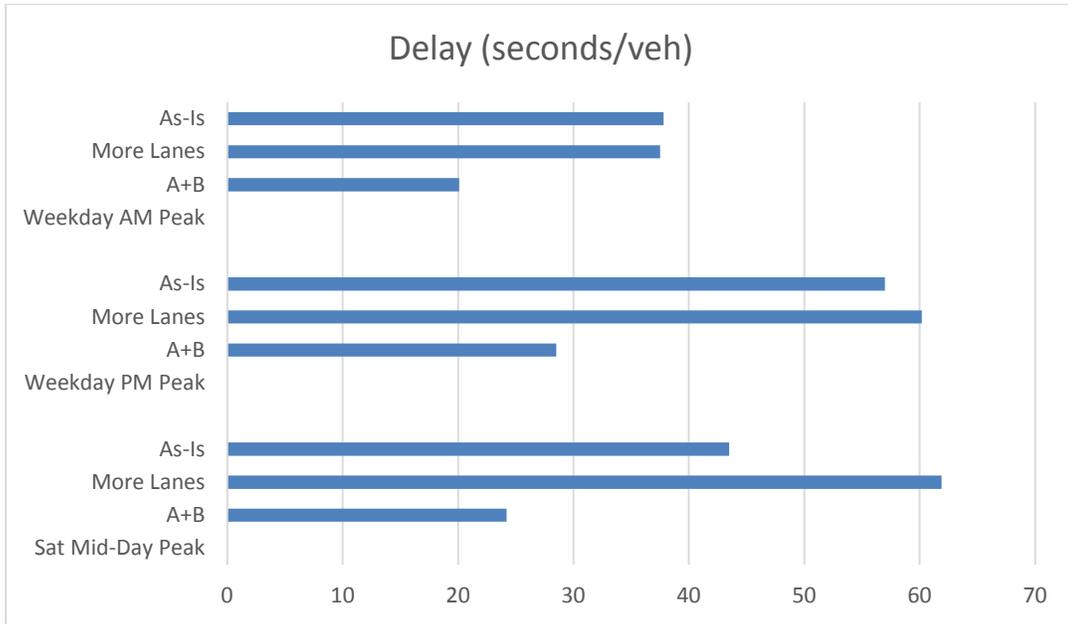


Figure 5-9. Delay for the field retrofit test case

In the field retrofit test case, additional turn lanes were not found to improve delay significantly, and for the Saturday mid-day peak the additional lanes caused significantly more delay. As noted in the results for the previous test, this is mostly likely due to the new requirement of protected-only phasing for dual left-turn lanes, where it may cause major inefficiencies. The Type A+B scenario reduced delay by about half in each case.

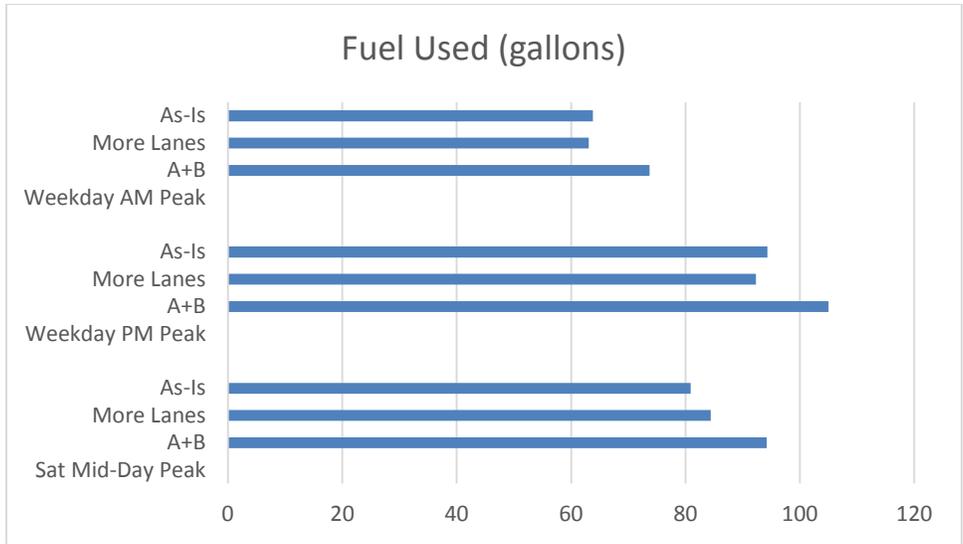


Figure 5-10. Fuel used for the field retrofit test case

Fuel use was slightly higher, presumably due to the U-turns required. Additionally, because the origin of the 75% of vehicles choosing to use the signalized exit to turn left was unknown, the additional length these vehicles had to travel is underestimated.

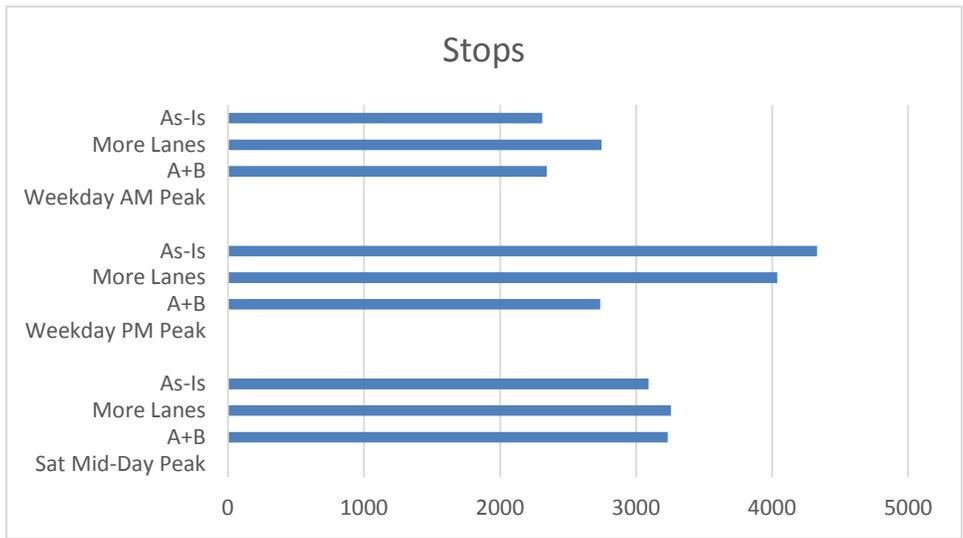


Figure 5-11. Stops for the field retrofit test case

The number of stops was not significantly changed except in the weekday PM peak, where the Type A+B case reduced it by around one third.

6.0 SUMMARY AND CONCLUSIONS

This chapter synthesizes the results, revisits the hypothesis to determine to what extent it was confirmed or contradicted, and presents topics for further research.

6.1 SUMMARY OF RESULTS

6.1.1 Review of Tests Performed

Two experiments were performed: the test traffic volumes, and the field retrofit test case. The first varied the magnitude of traffic volumes (with a fixed proportion) at an isolated intersection, in order to compare performance measures between a conventional intersection, a Type A jughandle, and the Type A+B jughandle. The test traffic volumes considered both a two-lane arterial with a single left-turn lane and a three-lane arterial with a dual left-turn lane.

The field retrofit test case replaced a corridor of three actual intersections with two Type A+B jughandles on the outside and a right-turn-only intersection in the center, to compare performance measures for this configuration against those for the current conditions and those for the addition of turn lanes.

The tests performed in each experiment are listed in Figure 4-1 and compared in Table 4-1.

6.1.2 Maximum Volumes at an Isolated Intersection

The test traffic volume results offer maximum volumes at which the modeled intersection types can maintain level of service D given the turning movement proportions used. The Type A+B

jughandle featured the highest maximum permitted volumes. With two through lanes, one left-turn lane on the A jughandle, and a single-lane B jughandle, it could provide LOS D at an x value of 2000 vehicles per hour. If the volumes in the proportion model were accurate at a real intersection, this would correspond to a bidirectional volume of approximately 4850 vehicles per hour on a section of mainline between intersections. With three through lanes, two left-turn lanes on the A jughandle, and two lanes in the B jughandle, the Type A+B jughandle could provide LOS D at 3000 vehicles per hour, which corresponds to a bidirectional mainline volume of around 7275 vehicles per hour.

Table 6-1 lists the highest x values for which each intersection could provide LOS D. It also shows, for each intersection, the ranges of x values for which that intersection had the lowest delays of all the intersections with the same number of lanes.

Table 6-1. Values of x at which intersections are acceptable or optimal

Number of Lanes	Intersection Type	Highest Passing x Value	Range of Lowest Delay
2 Through, 1 Left	Conventional	1600	100-200
	Type A	1500	300-1400
	Type A+B	2000	1500-2000
3 Through, 2 Left	Conventional	2300	N/A
	Type A	2700	N/A
	Type A+B	3000	1100-3100

It should be noted that, in the “3 Through, 2 Left” set, ranges below $x = 1100$ vehicles per hour were not tested. If they had been, there would likely be a range of lowest delay for the conventional intersection and the Type A jughandle.

6.1.3 Delay

As summarized in Table 6-1, LOS D (per-vehicle delay under 55 seconds) can be maintained up to an x value of 1600 vehicles per hour for the conventional intersection with two through lanes and one left-turn lane on each approach. The Type A jughandle with two through lanes on the

mainline and one left-turn lane and one through lane on the minor approach could maintain LOS D until $x = 1500$ vehicles per hour. The Type A+B jughandle with two through lanes, one left-turn lane on the A jughandle, and a single-lane B jughandle could provide LOS D at 2000 vehicles per hour. As previously seen in Table 5-1 and Figure 5-1, delay at each intersection began to rise very quickly when approaching the point of failure.

For the test with 3 through lanes and 2 left-turn lanes, LOS D was achieved for up to 2300 vehicles per hour with the conventional intersection, up to 2700 for the Type A jughandle, and up to 3000 for the Type A+B jughandle. Table 5-2 and Figure 5-5 show that each of these intersections featured a more gradual acceleration of delay here than in the previous set of tests.

It was found that the addition of a second left-turn lane to the mainline in the conventional intersection with two through lanes resulted in a universal increase in delay, and these results were thus omitted. This was most likely due to the requirement of protected-only left-turn phasing. With permissive phasing, the small number of left-turning cars were in some sense absorbing the delay, whereas protection added a phase that caused delay for the large number of through vehicles. The addition of a left-turn lane on both the mainline and the minor approach resulted in even greater delay. In this case, protected left-turn phasing had to be used on the minor road despite a minimal number of opposing through vehicles, wasting a significant amount of the cycle.

In the field retrofit test case, delay was reduced by approximately 50% at each peak hour when using the Type A+B RTUT configuration.

6.1.4 Fuel Used

Fuel use was increased for a Type A+B jughandle in the test traffic volumes experiment, except at the point that other intersections were failing. In the retrofit test case, fuel use was always increased by a small amount, and may still have been underestimated due to the redistribution of volumes within the plaza.

6.1.5 Stops

The number of stops for the Type A+B jughandle was sometimes reduced and sometimes slightly increased, compared to the other intersections. For the test traffic volumes with two through lanes, the number of stops was slightly higher until delay began to spike for the other intersections. With three through lanes, the number was always somewhat lower. In the field retrofit test case, the number of stops was not significantly different for the weekday AM or Saturday mid-day peak, but was reduced by over a third in the PM peak.

6.1.6 Footprint

The footprint for the Type A+B jughandle was always much higher than that for the conventional intersection, and almost always much higher than that for the Type A jughandle. For the test traffic volumes with two through lanes, the additional footprint was up to six times as high as that for a conventional intersection; it was up to three times as high with three through lanes. In the retrofit case, more compact ramps were able to be used, but the additional footprint, even including savings from removing left-turn lanes along the corridor, was still estimated at 85,600 square feet.

6.2 COMPARISON TO THE HYPOTHESIS

The hypothesis proposed that, compared to a conventional intersection design, the Type A+B jughandle would have lower delay and fewer stops under certain traffic demand conditions, and that it would always have higher fuel use and a greater footprint. It was specifically believed that the design was best suited for high-volume arterials surrounded by low-density development, and for intersections at which demand is high for left turns from all four approaches, significant for arterial U-turns approaching the intersection, and minimal for through movements on the intersecting street. The RTUT movement was highlighted as one potential application.

Data used in the preceding tests were taken from a corridor with land use patterns typical of the conditions hypothesized appropriate for the Type A+B jughandle. The turning movement

proportion model used for the test traffic volumes, as well as the potential for an RCUT configuration in the test case, largely reflected the conditions described in the hypothesis, the only notable exception being the lack of U-turns in the proportion model. The results indicate that per-vehicle delay and the number of stops can be significantly reduced for very high demand volumes when such conditions are present at an isolated intersection *without* U-turns. They additionally indicate that footprint is always significantly higher than a conventional intersection or conventional jughandle, and that fuel use is generally slightly higher until volumes that cause other intersections to fall into LOS E.

Based on observations of queuing during the simulations and the U-turn equivalency factors established in the literature review, it is reasoned that adding U-turns in the conventional intersection would moderately increase delay and number of stops. The Type A jughandle would, for each U-turn, incur a new left turn at both the outer and central intersection, and would probably also see a moderately negative impact. However, in the Type A+B jughandle, there is very little difference between a left turn and a U-turn, and some U-turns could likely be accommodated without major impacts. The average queue reported on the queue space segment was found generally to be much shorter in the section directed away from the central intersection than in the section directed toward it, indicating that a large number of U-turning vehicles could be stored without a change in phasing. Therefore, the hypothesis, which is here largely confirmed under the exclusion of U-turns, should hold true to no lesser degree with the addition of U-turns. The field retrofit test case did utilize the U-turn capacity, and nearly reduced delay by half during the highest peaks. In these cases, adding more turn lanes to the conventional intersections could not increase capacity.

The hypothesis was in part motivated by the inefficiency found at intersections with very little through traffic on the minor road, a condition that simulation testing demonstrated to be inefficient for volumes of minor-road left turns that required a double left-turn lane. In order to merit minor-road left-turn protected phasing with a single left turn lane, the value of x would need to be well over 5000 vehicles per hour, an unrealistic volume for interrupted flow. But with a double left-turn lane, protected-only phasing is required regardless of the volume, lengthening the cycle. As expected, both jughandles act to reduce this inefficiency.

The portion of the hypothesis most clearly contradicted by the results is the amount of fuel used. While the hypothesis assumed that the Type A+B jughandle would always increase this

quantity, its fuel use was lower than that for the other intersections when the other intersections were beginning to fail. However, fuel use was not decreased in the field retrofit test case, where many vehicles were going far out of their way.

6.2.1 Appropriate Context

The results demonstrate that the hypothesized ideal context of high-volume arterials with a high proportion of left turns and a minimal volume of minor-road through traffic may experience significantly improved operations with a Type A+B jughandle design, as opposed to a conventional intersection or an established jughandle design. The very large footprint makes a retrofit difficult in areas with existing structures surrounding the road. In some cases, the A jughandles could consist of paths through parking lots to secondary exits, or the ramps could enclose and provide access to small properties; however, the B jughandles should not allow access and cannot enclose any vehicular destinations, only sidewalks, utilities, poles, and signs.

The field retrofit test case suggests that a retrofit from multiple conventional intersections to some Type A+B jughandles and some right-turn-only intersections between them could reduce congestion where additional turn lanes cannot do so and where additional travel lanes are not feasible. Additionally, this reconfiguration allows the installation of a median barrier without forcing drivers who wish to turn left to make difficult or dangerous U-turns. For this reason, the A+B RCUT system is an option for tight sections of roadway where widening even for a directional or two-way left-turn lane between signals would be prohibitive.

The construction of a new corridor with Type A+B jughandles seems like a difficult case to make compared to a retrofit, given that such a complex system would not seem necessary unless the corridor had already become overdeveloped and highly congested. However, if development patterns in a planned corridor were seriously expected to create such a situation, and right-turn-only intersections and driveways were being considered, then the A+B RCUT system could be implemented, or at least the needed land could be reserved for future use. Because this design removes left-turn lanes from the main roadway cross section, it may be an appealing option where a minimal width is desired for the roadway itself, particularly when accommodating U-turns requires a wider median.

7.0 RECOMMENDATIONS FOR FUTURE RESEARCH

Several crucial aspects of the Type A+B jughandle were not investigated in this paper but require examination. The following sections provide some background for further study in two of these areas: multimodal impacts and safety. A final section then concludes with general recommendations for future research.

7.1 CROSSWALK PHASING AND MULTIMODAL IMPACTS

An intersection requiring this much land will likely be built in a low-density, automobile-dependent area without numerous pedestrians; still, it must accommodate pedestrians. Figure 7-1 shows the location of crosswalks and walking paths between them. This paragraph refers to crosswalks using their numeric labels in the figure. Signalized crosswalks across the mainline may be placed on the outside of each A jughandle (1) and in between the two B jughandles (2). For a case in which minimal U-turns are expected for a B jughandle, a signalized crosswalk may also be placed on its outside (3). Signalized crosswalks should also cross the A jughandle termini (4). Unsignalized crosswalks can connect the islands to the exterior of the intersection, crossing channelized right turns, as long as proper sight distance is maintained (5). Since the distance between intersections and the length of the B jughandle ramp will determine the alignment of the islands, the island inside the B jughandle ramp could need to be extended outward to meet the crosswalk (1) from the A jughandle island. Note that pedestrians might utilize a diagonal shortcut to this crosswalk when they observe that this will not conflict with the left-turn movements from the A jughandle.

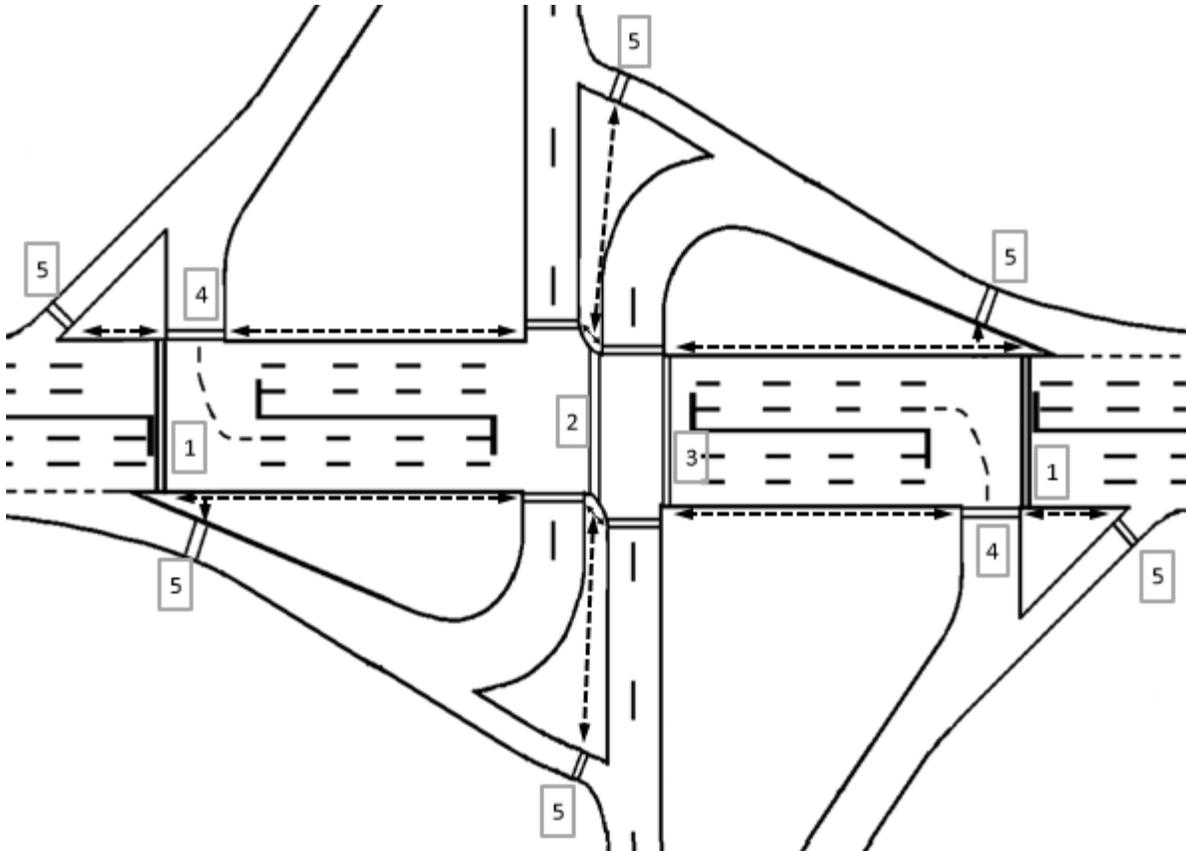


Figure 7-1. Location of crosswalks and pedestrian paths

As shown in Figure 7-2, crosswalk phasing can follow the standard protocol, with the exception of the crossing of the lanes exiting the central intersection. A pedestrian lag is recommended for this signal, due to the fact that the only vehicles needing to turn at the central intersection are those that just entered from the A jughandle and are turning right to make the crossing movement. These movements will be served at the beginning of the mainline green phase, so a crosswalk lag equal to the time for clearance of the queue space is sufficient. The right-turn movement can receive a turning arrow during this lag period. This lag should not prevent pedestrians from making a full crossing in one phase; after the queue space clears, the mainline will still receive green for at least long enough to clear the external queue waiting to enter the queue space.

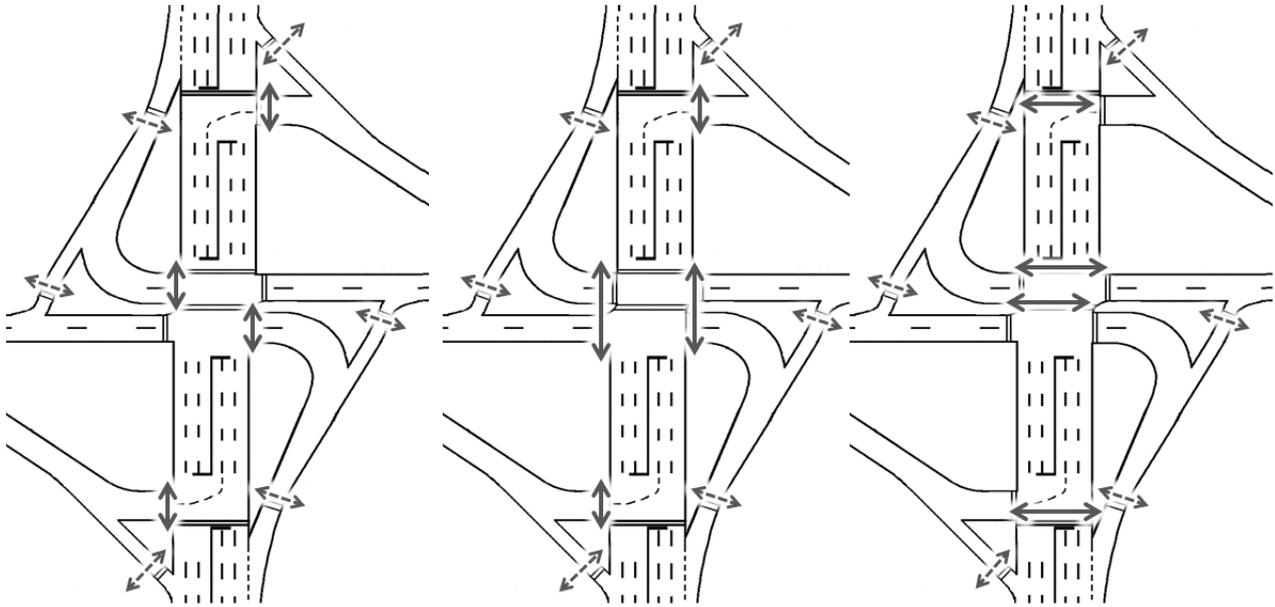


Figure 7-2. Proposed pedestrian phasing

In the first diagram, at the *beginning* of the mainline phase, pedestrians receive protection across the A and B jughandle termini. In the second diagram, the lag has ended and pedestrians additionally receive protection across the central intersection's exits. This lasts for the remainder of the mainline phase (unless mainline vehicle actuation requires the pedestrian protection to end when mainline green begins to be extended). The third diagram shows the left phase, during which pedestrians may cross the mainline at any of the three intersections. It is important to note that this figure displays an example in which low U-turns from direction 2 allow a crosswalk on the exterior of the central intersection; this crosswalk receives protection simultaneously with the others. In general, this will not be the case. In all phases, drivers must yield to pedestrians in the unsignalized crosswalks, represented by a dashed double-headed arrow across the ramps. Queue space clearance phases could include some of the flashing don't-walk indication and the buffer if absolutely necessary, but are not pictured.

Aside from the conflict minimized by the lag, and the optional crosswalk on the outside of a B jughandle with few U-turns, no signalized crosswalk presents a conflict between pedestrian movements and permitted vehicular movements. Walking across the ramps presents some increased danger, particularly with higher speeds, and ramp diverges may endanger bicyclists going straight. Transit vehicles are not likely to be affected, but could potentially benefit from

the opportunity to make a wide U-turn movement, especially following a nearby bus stop on the right side of the road. Emergency response vehicles on the mainline can bypass the B jughandle by making a direct left turn, but they will still have to use the A jughandle when approaching from the minor road.

7.2 SAFETY

Based on the previously discussed safety benefits of jughandles, it is expected that the Type A+B jughandle will also improve safety over a conventional intersection. There are no permitted or stop-controlled movements, only protected signalized movements and yield conditions on the right-turn channelizations. For this reason, the Type A+B jughandle may be an improvement over the Type A jughandle, where drivers must select a gap at which to turn left.

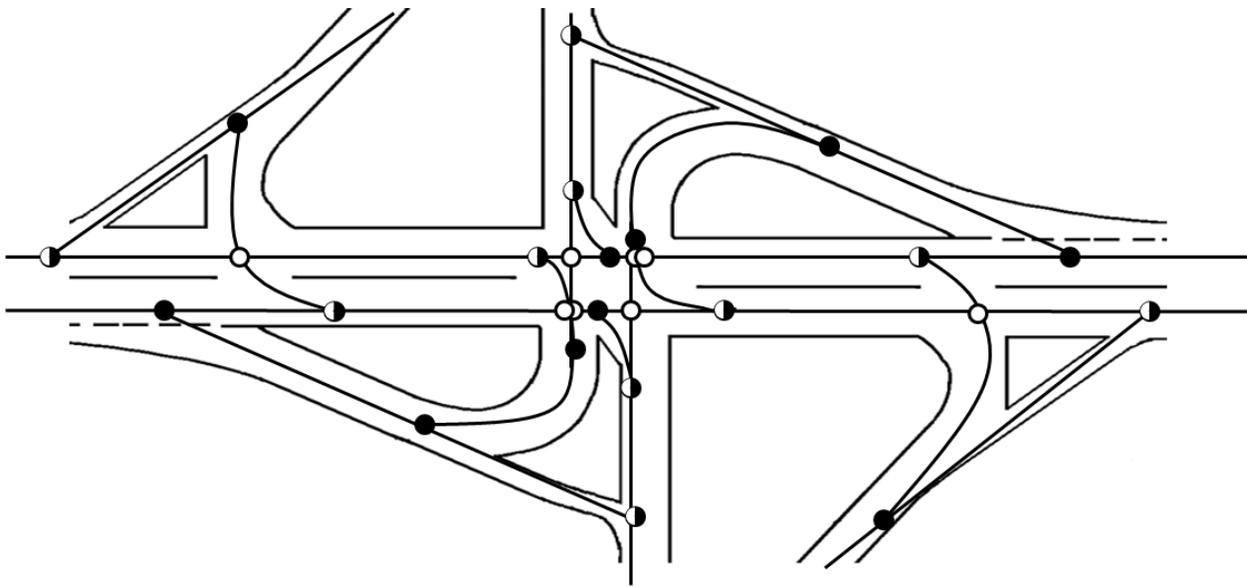


Figure 7-3. Conflict diagram for the Type A+B jughandle

Table 7-1 compares the conflicts involved in the Type A+B jughandle to those at a conventional intersection, and at unconventional intersections with similar characteristics. Since U-turns are

not included in the tally for conventional intersections, closely-spaced Ts, and Type C jughandles, a row is provided for comparison with the number of conflicts in a Type A+B jughandle excluding the U-turn movement.

Table 7-1. Conflict totals for different types of four-leg intersections

Intersection Type	Diverge/Merge	Crossing (Left)	Crossing (Angle)	Total Conflicts
Conventional 4-Leg	16	12	4	32
2 Closely Spaced Ts	12	6	0	18
Type A Jughandles	16	6	4	26
Type C Jughandles	16	4	4	24
Type A+B Jughandle	20	4	4	28
A+B without U-turns	16	2	4	22

7.3 GENERAL RECOMMENDATIONS FOR FUTURE RESEARCH

This thesis is the first literature published on the Type A+B jughandle; thus there are a large number of directions in which the research could be extended. First and foremost, the design should be examined to seek variations that may be superior to the version presented herein. If a version is found that is clearly preferable in all situations, then any further research should be conducted on that concept. Because there is no straightforward process for originating unconventional intersections, the possibility of a variation incorporating this design's motivating benefits with fewer drawbacks cannot be dismissed.

At this publication, implementation of the design is far from advisable. Determining, for intersections in isolation, the significant characteristics of turning movement proportions most relatively beneficial to the operations of the Type A+B jughandle would greatly improve the ability to evaluate which types of intersections would make ideal candidates for eventual trials of the design. The design should also be investigated specifically in the context of the RCUT configuration, as well as any other situations that require the facilitation of U-turns on an arterial.

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