DEVELOPMENT OF PERFORMANCE MATRICES FOR EVALUATING INNOVATIVE INTERSECTIONS AND INTERCHANGES

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16.   Abstract

Innovative intersections and interchanges, primarily Continuous Flow Intersection (CFI) and Diverging Diamond Interchange (DDI), have seen an increase in numbers in the State of Utah over the past several years, making Utah a leader in the country in implementation of these designs. Although on the surface these designs seem to improve traffic performance, their complete impacts and benefits are hard to assess. There are still no clearly defined guidelines and methodologies for monitoring and measuring performance of these designs from state DOTs manuals, AASHTO, HCM, NEMA and HSM. Innovative designs have impacts on operations, safety, accessibility, transit, pedestrian and non-motorized traffic, land use, economic development, and environment. Due to this variety of impacts, there is a need to develop methodologies to further evaluate innovative intersection designs with regards to different performance measures. The primary objective of this research project is to develop a matrix of performance measures which stakeholders could apply in practice to effectively evaluate innovative intersection designs in terms of operations, safety, access, and multimodal accommodations.

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INNOVATIVE INTERSECTIONS: OVERVIEW

Introduction

Innovative intersections (also known as unconventional intersections or alternative intersections) are generally defined as any at-grade design concepts that are able to reduce the number of phases at the main intersection, thereby improving the overall intersections operational and safety performance (*1*). In most cases this is accomplished by rerouting left turns at a point well ahead of the main intersection, or accomplishing left turns through a combination of through, right and U-turn movements. These designs are regarded to be "unconventional" because they incorporate geometric features or movement restrictions that would be permissible at standard at-grade intersections (*2*). Such elements include the elimination and/or relocation of various through and turning maneuvers, the use of indirect turning movements, and the inclusion of roundabout designs.

The general goal of innovative intersections is to improve the overall operation of the intersection by favoring heavy volume through movements on the arterial street, to decrease crash frequency and severity through geometric designs whish encourage conflict avoidance, and to provide adequate alternative which accommodates travel demand without the need to fund major construction work usually required by grade-separated solutions. The ways that innovative intersections improve traffic conditions can be summarized as follows:

- Reducing and/or separating the intersection conflict points, in order to improve safety
- Restricting and/or rerouting movements, to disperse traffic and reduce congestion
- Reducing the complexity of traffic signal phasing, in order to reduce the overall intersection delay
- Providing enough capacity for the existing and predicted travel demand, in order to avoid grade-separated solutions which require major economic investments

One of the recognized problems with new implementations of innovative intersections is the issue of driver expectancy. As drivers are used to conventional intersection designs, maneuvering through innovative intersections might be challenging due to rerouted movements. While some states lead the way in innovative designs implementation (e.g., Michigan, Utah), drivers in other states did not have the chance to experience driving through these "unusual" intersections. It is imperative that the DOTs provide adequate drivers education and guidance to cope with potential confusion which may occur due to unexpected intersection designs, particularly during the initial period following the installation.

Different intersections designs have appeared during the last few decades that are considered "unconventional". These new designs for urban intersections are context sensitive, efficient, and often affordable, especially if such a design is envisioned when adjacent land uses are first established (*1*). In most cases, they can accommodate more traffic than grade-separated designs, with much lower construction and maintenance costs.

Innovative intersections and interchanges, primarily Continuous Flow Intersection (CFI) and Diverging Diamond Interchange (DDI), have seen an increase in numbers in the State of Utah over the past several years, making Utah a leader in the country in implementation of these designs. Although on the surface these designs seem to improve traffic performance, their complete impacts and benefits are hard to assess. There are still no clearly defined guidelines and methodologies for monitoring and measuring performance of these designs from state DOTs manuals, AASHTO, HCM, NEMA and HSM. Innovative designs have impacts on safety, accessibility, transit, pedestrian and non-motorized traffic, land use, economic development, and environment, making them an excellent candidate for an in-depth analysis of different benefit-impact combinations. The primary objective of this research project is to develop a set of performance matrices for evaluation of innovative intersection designs from different standpoints.

This report provides an overview of various innovative intersection designs in the first section. The summary of the most important geometric design considerations is presented in the second section, based on the FHWA's Alternative Intersections Informational report, published in 2012. Then the report focuses on the review of the existing performance evaluation methods, and provides a detailed insight into innovative intersections performance measures related to safety, access, and multimodal transportation. The final product of this research are the "performance modules," five of them developed for interchange evaluation and four of them developed for intersection evaluation. Both sets of modules, for interchanges and intersections, are combined into two final performance matrices, in the form of a spreadsheet output to be used by stakeholders for the evaluation purposes.

Diverging Diamond Interchange (DDI)

The Diverging Diamond Interchange (DDI) eliminates left turn phasing by moving left and through movements to the opposite side while crossing the freeway overpass, as shown in Figure 1. The DDI also allows vehicles taking a right turn to do so before entering the intersection.
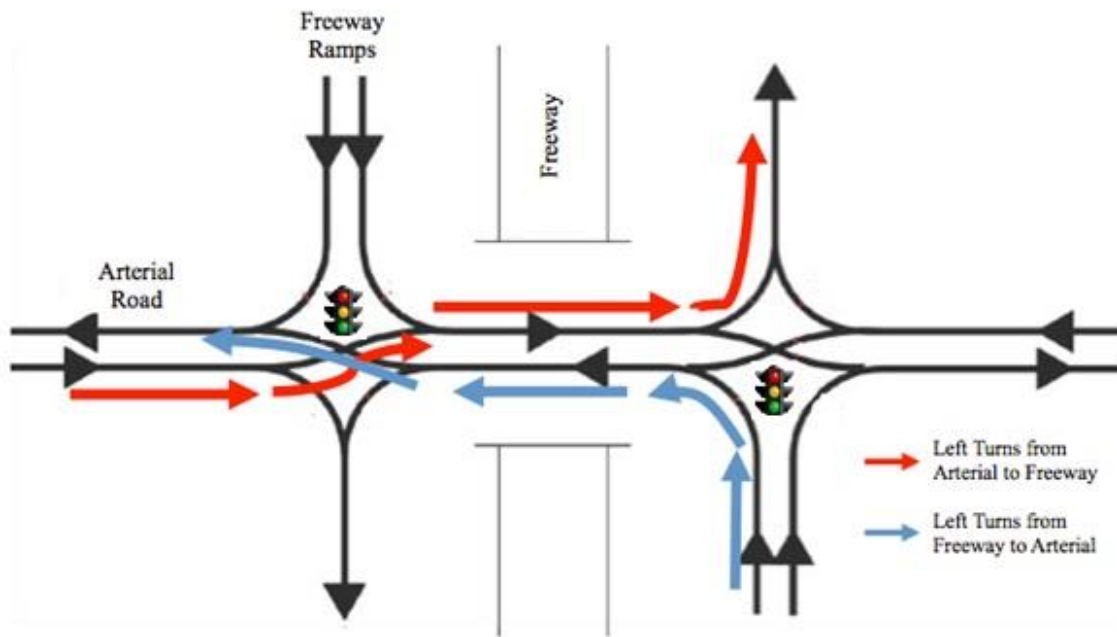
Figure 1: Diverging Diamond Interchange (DDI)

There are traffic lights at each side of the freeway overpass, any vehicles making right turns at either of these lights can exit before entering these intersection. Vehicles going through the intersection onto the freeway go through the traffic light in the left lane, merge onto the left side of the road and merge onto the highway after crossing the overpass. Through movements are completed by crossing the overpass, driving on the opposite side of the road, and entering the right lane at the second traffic light after the overpass.

A major advantage of the DDI is that it removes the turning movements from the intersections, as seen in the diagram above, all left and right turning movements are completed before approaching the traffic lights (*3*). A DDI reduces the total number of conflict points from the 26 points associated with a traditional diamond to 14. The signal operation of a DDI is improved because the two-phase signal can decrease cycle lengths in order to reduce delays and increase the intersection's capacity (*4*).

There are also a few disadvantages that have been observed with DDIs. Interchanges with higher through traffic volume are not ideal for implementing a DDI because through traffic is most inconvenienced in a DDI. Pedestrian access has also been identified as an issue with traffic flow in a DDI, pedestrian movements are signalized to help alleviate some of the confusion; however, it can still be difficult for pedestrians to adjust given that the traffic is moving in a different direction than they expect (*4*).

Continuous Flow Intersection

The Continuous Flow Intersection (CFI) is another complex unconventional intersection design in terms of the amount and proximity of channelizing and control features. The basic concept of the CFI is to move left turn traffic from all approaches of the main intersection across the opposing traffic lanes prior to the main intersection (*1, 2, 7*). Left turn maneuvers are then completed simultaneously and unopposed with their accompanying and opposing through movements, allowing the intersection to operate on a two-phase signal. For comparison, a standard signal with protected left turn arrows must serve eight major movements, four left turns and four through movements, but only two movements can occur at a time, which demands a four-phase signal. The left turns prior to the intersection are also signalized, but they are coordinated with the main signal allowing the left turning vehicles to cross the main intersection without stopping. The diagram of a CFI intersection is given in Figure 2. It shows only the CFI design on the major roadway, although it can be implemented on all approaches.
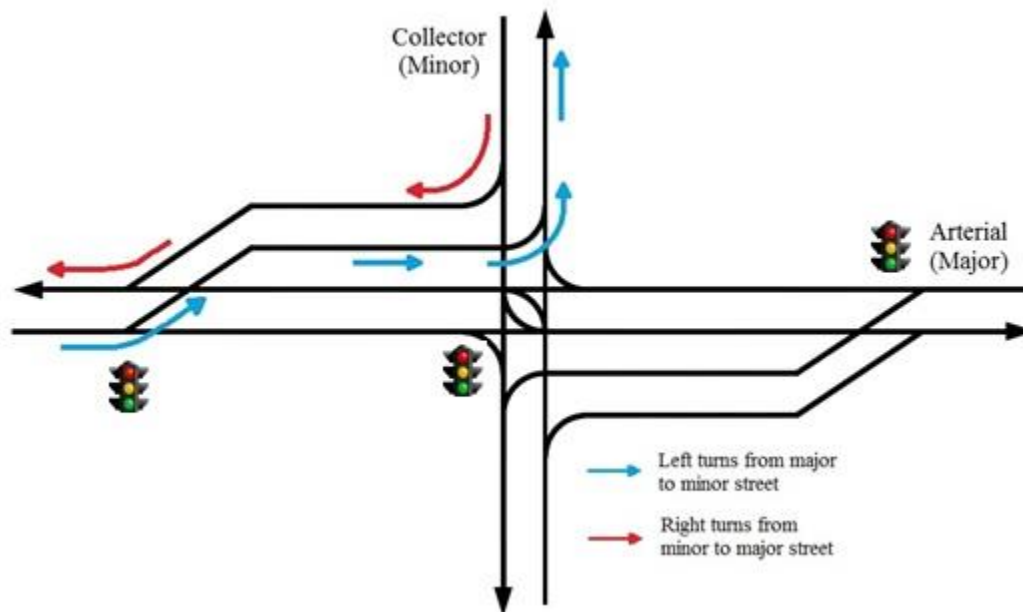
Figure 2: Continuous Flow Intersection (CFI)

It has proven to be simple for drivers to get used to, and in some cases can fit within existing rights-of-way (*1*). A full 4-approach CFI with 2-3 lanes per approach can handle about 10,000-14,000 vehicles per hour at LOS E. A standard intersection with the same number of through lanes and with dual left-turn lanes on all approaches can handle about 6,000- 8,000 vehicles per hour at the same level of service. The CFI design can greatly increase capacity and reduce delays.

The CFI also has some disadvantages. Drivers need to be aware of the need to make left turns prior to the intersection, so clear guidance must be given to warn them of the impending roadway and guide them into the appropriate lanes. Because of the multiple lane crossings within the

intersection, pedestrians would also need to be guided and informed of the vehicle approach direction. Other disadvantages include the need for U-turn opportunities because access to and egress from intersections quadrant developments would be difficult for most approach movements. The CFI would be most appropriate for high volume arterials with few needs for U-turns. Another important consideration is the level of development near the intersection. Because of the locations of the left and right turn lanes, the CFI does not provide easy access to and from adjacent properties.

Median U-Turn Intersection

The main objective of the median U-turn intersection (a.k.a. Michigan U-turn, Through-turn) is to remove all left turn traffic from the main intersection. It redirects left turns through a combination of through, right, and U-turn movements (*1, 2, 5, 6*). A schematic diagram of this intersection type is given in Figure 3.
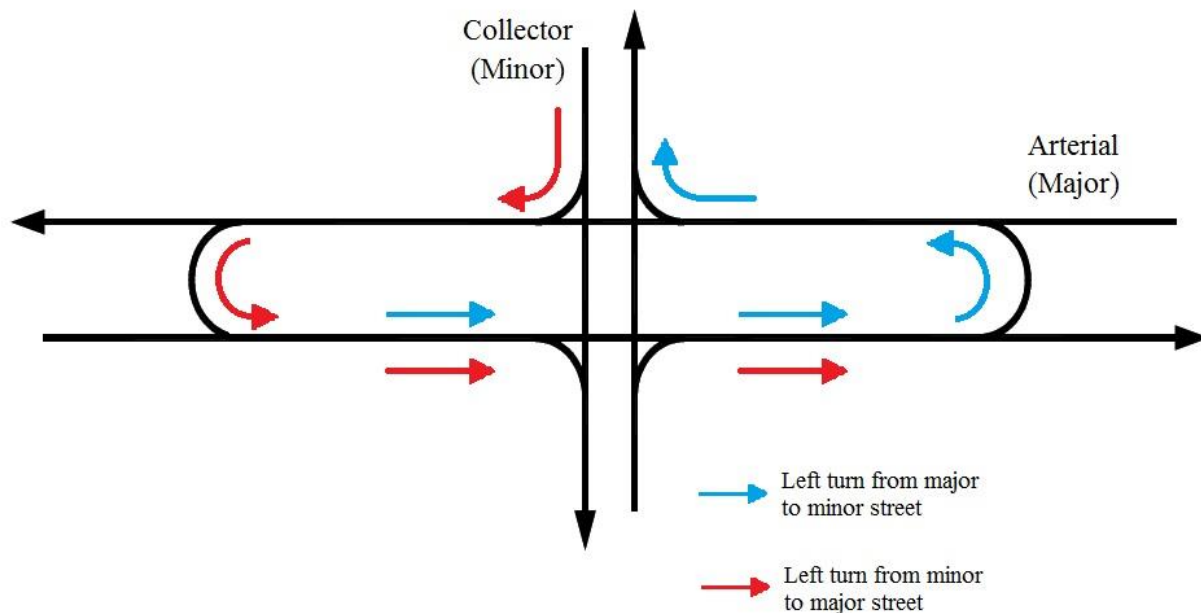
Figure 3: Median U-turn

Vehicles turning left from the major to minor street continue through the intersection, make a U-turn at the designated place on the Major Street, and then turn right at the intersection. Vehicles turning left from the minor to Major Street first turn right at the intersection, make a U-turn at the designated place on the major street and then continue straight through the intersection. The relocation of left turns at the intersection simplifies its signal phasing. The intersection can operate on a simple two-phase timing plan, increasing capacity, reducing delays and improving intersection coordination. Safety at this intersection is also improved, since it eliminates conflicts between left turning and through vehicles. For the same reason it is more pedestrian-friendly, since there are no conflicts between pedestrians and left-turning vehicles. Studies on median U-

turn intersections show an increase in capacity of about 50% when compared to double left turns, and a crash rate that is 20% lower (*1*).

The main disadvantage of median U-turn is increased delay and travel distance for left turning vehicles. In some cases, the U-turn may require a separate signal if the traffic volumes on the major street are too high. Also, sometimes it may be needed to expand the roadway at the U-turn section, which takes up more space.

This type has been in use in Michigan since the 1960s (hence its name). The drivers in Michigan are used to this design type, so it does not conflict their expectancy. They are not so common in other states, which can cause unusual driver expectancy in the early stages of implementation.

Superstreet Intersection

The Superstreet intersection (also known as the Restricted Crossing U-Turn (RCUT) Intersection) has a lot of similarities with the Median U-turn intersection. In this case, the main intersection is closed for both through and left movements from the minor street. They are achieved through a combination of a right and U-turn movement. The effect of this configuration is that it allows a four-approach intersection to operate as two separate three approach intersections, and allows each direction of the major street to operate on an independent timing pattern (*1, 2*). In this case, left turns from the major roadway on to the minor street are allowed at the main intersection. This configuration is shown in Figure 4.
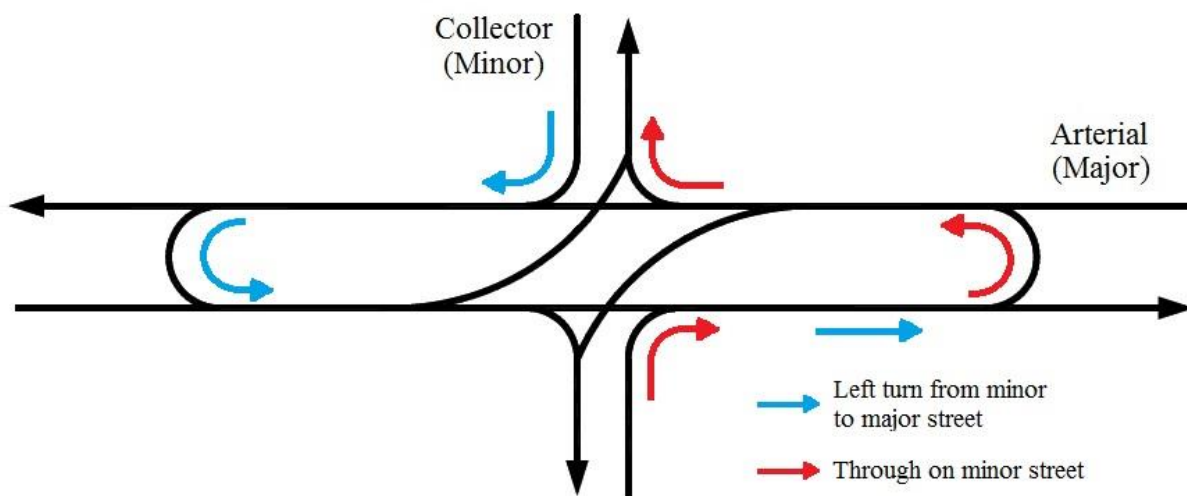
Figure 4: Superstreet Intersection

Because of the ability to independently control the major street directions, the superstreet design permits coordinated progression for the major street regardless of its spacing relative to upstream and downstream intersections. This significantly reduces delays on the major roadway. The most significant disadvantage is that it does not permit through or direct left turn movements from the minor roadway. This increases delays and travel distances for those movements. The driver

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expectancy can also be a problem. Pedestrian are required to cross the main intersection at an angle, parallel to the left turn crossovers, requiring a longer pedestrian phase.

Bowtie Intersection

The turning movements at Bowtie intersections are similar to median U-turn intersections. The difference is that the Bowtie uses roundabouts located on the minor road, as shown in Figure 5 (*1, 2, 5, 6*). The advantages are similar to those seen at median U-turns, with elimination of left-turn phases, increased capacity and improved safety. Also, Bowties eliminate the need of having signalized U-turns, since roundabouts are used in this case. Having a roundabout on the minor street is also an advantage, because the turning movements face lower traffic volumes. The roundabouts in the Bowtie variation also provide unique opportunities for side-street tie-ins, improved aesthetics, and traffic calming, which are qualities attractive for livable corridors.
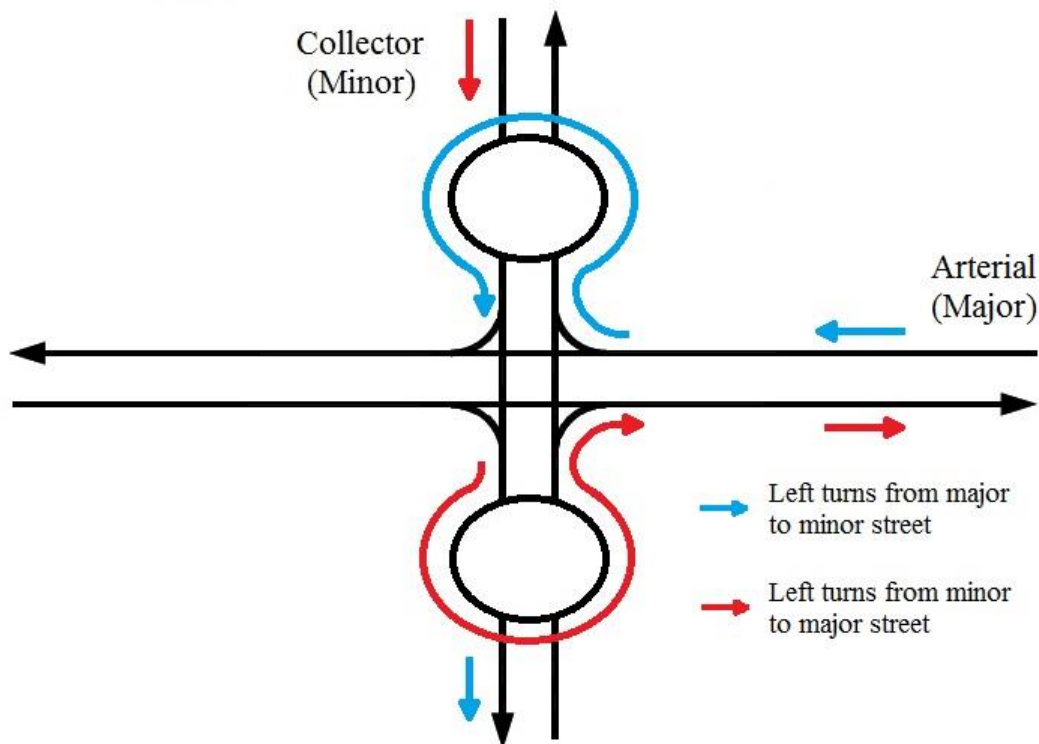
Figure 5: Bowtie Intersection

The distance between the main intersection and the roundabouts depends on the amount of storage space required for minor street approach queuing. The size of the roundabouts would depend on the design speed and design vehicles in a particular location.

Bowties increase delays and travel distances for left turning vehicles, which is the major disadvantage. Also, the roundabouts in the Bowtie require additional space for construction. Unusual driver expectancy should also be considered with this intersection type.

Quadrant Intersections

At a Quadrant Intersection, left-turns are redirected onto an adjacent roadway that connects two legs of the intersection at locations that could allow traffic to bypass the main intersection. This decomposes the main large intersection into three smaller signalized intersections. All left turn movements from both roads are completed prior to or after the main intersection on a by-pass road (*1, 2, 5*). The diagram of a single Quadrant intersection is given in Figure 6. It is possible to achieve all left turns with a single quadrant, although it is not recommended.
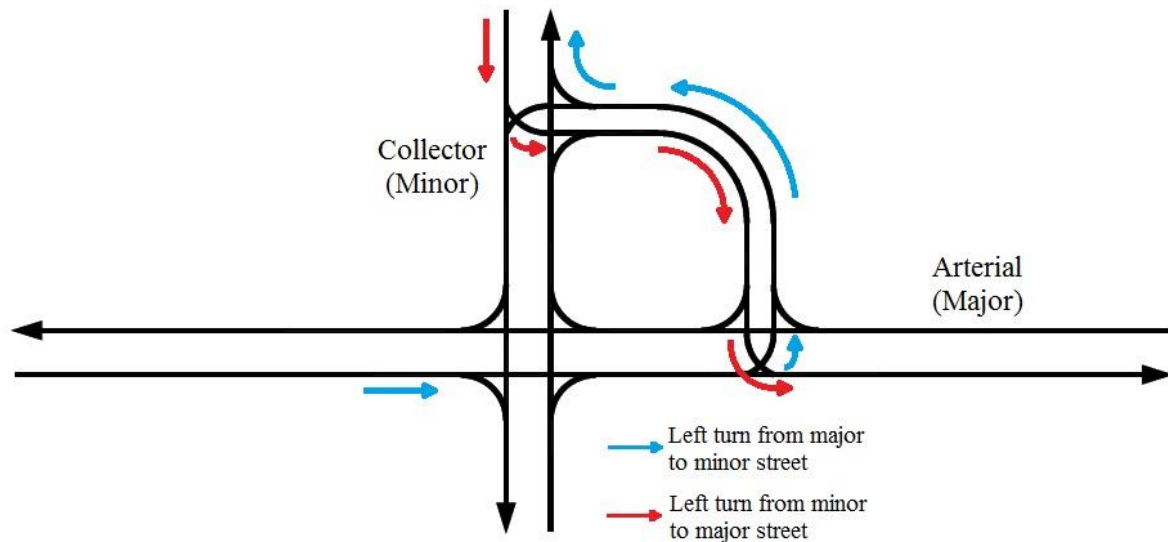
Figure 6: Single Quadrant Intersection

Eliminating left-turn movements at the main intersection increases the intersection capacity and efficiency by eliminating left turn signal phases, which in turn provides more green time to through traffic. Without left-turn movements, a simple two-phase signal can be used, which may increase corridor capacity by as much as 50%. Eliminating the left-turn movements also improves intersection safety by decreasing the number of vehicular and pedestrian conflict points, therefore reducing the opportunity for collisions. In the case of a single Quadrant intersection, a key component is the coordination of the three signals. The left turning movements into and out of the quadrant roadway occur during the phase that overlaps the coinciding movement at the main intersection, which minimizes (or even eliminates) the number of stops required to complete the left turn. The length of the quadrant roadway and the locations of its accompanying intersections are dictated by a trade-off between the amount of storage required for left turn queuing, and distance and time required to travel to the intended direction. Although building a quadrant intersection is more costly, it provides access to and from developments within the selected quadrant. A Quadrant can also provide opportunity for additional store front opportunities. A higher number of vehicles on the connector roadway will provide a unique and potentially profitable location for businesses. Aesthetic improvements can also be made to the Quadrant to help improve its appeal. Some other advantages of this design

include a reduction in conflict points at the main intersection, and reduced intersection widths that benefit pedestrians.

The main disadvantage of this intersection type is increased delay and travel distance for left turning vehicles. This configuration could also be more confusing for the drivers, because the left turn movements are not the same for different directions. Left turns for two of the approach directions would be made prior to the main intersection and the other two approaches would initiate their left turn maneuvers after the main intersection. Some of these problems can be solved by introducing two or four quadrant intersections.

Jughandle Intersection

The Jughandle intersection introduces a design similar to quadrant intersections. The principle of the jughandle design is to remove all turning traffic (including right turns) from the main intersection by shifting them from the major street approaches and onto an adjacent ramp (*1, 2*). The diagram of the Jughandle intersection is given in Figure 7.
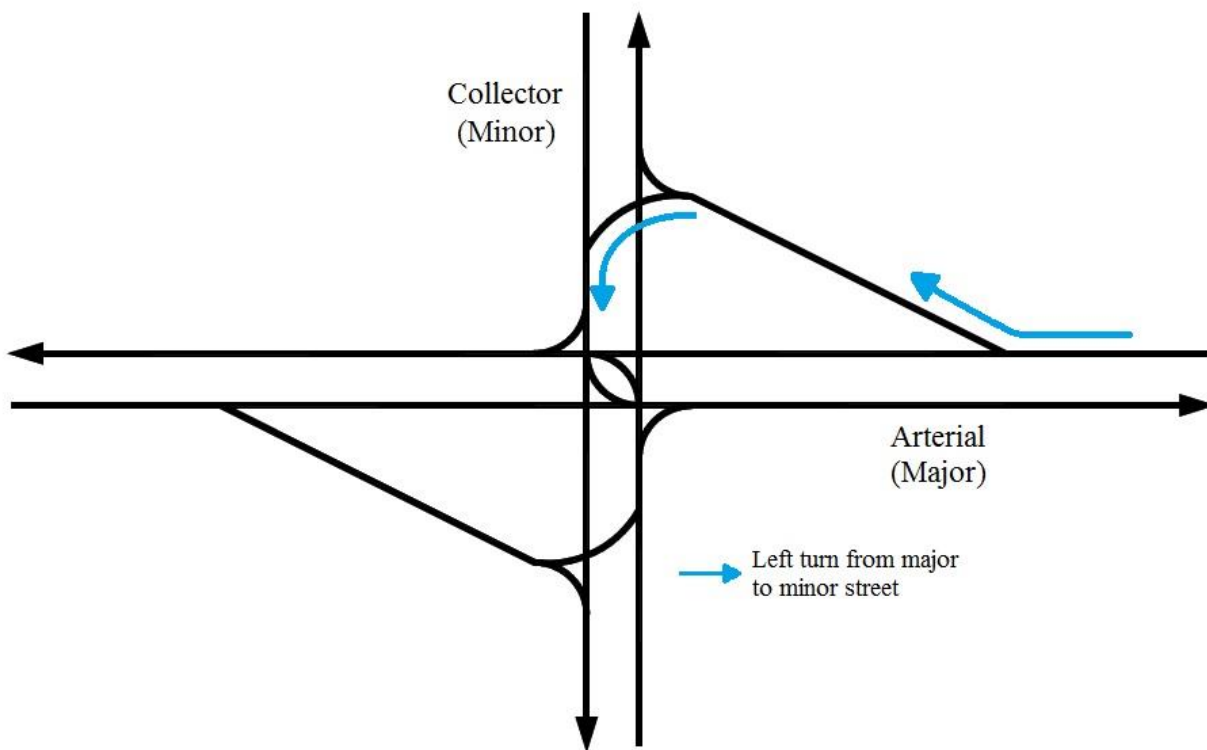
Figure 7: Jughandle Intersection

The turning maneuvers are completed at an intersection created between the ramp and the minor highway, and then proceed through the main intersection, similar as for the Quadrant intersection. However, a difference is that left turns from the minor street are permitted on to the major roadway. This design type is best suited for high volume arterial roadways with moderate to low left turn volumes. It eliminates the need for left turn phase on the major roadway

(although it may be needed for the minor road, depending on the volumes). Other advantages and disadvantages are the same as for the Quadrant intersection.

Split Intersection

The split intersection separates directional traffic flows into two offset one way roads. This configuration is similar to an at-grade diamond interchange without a separate bypass for through traffic (*2*). A diagram of this intersection is given in Figure 8.
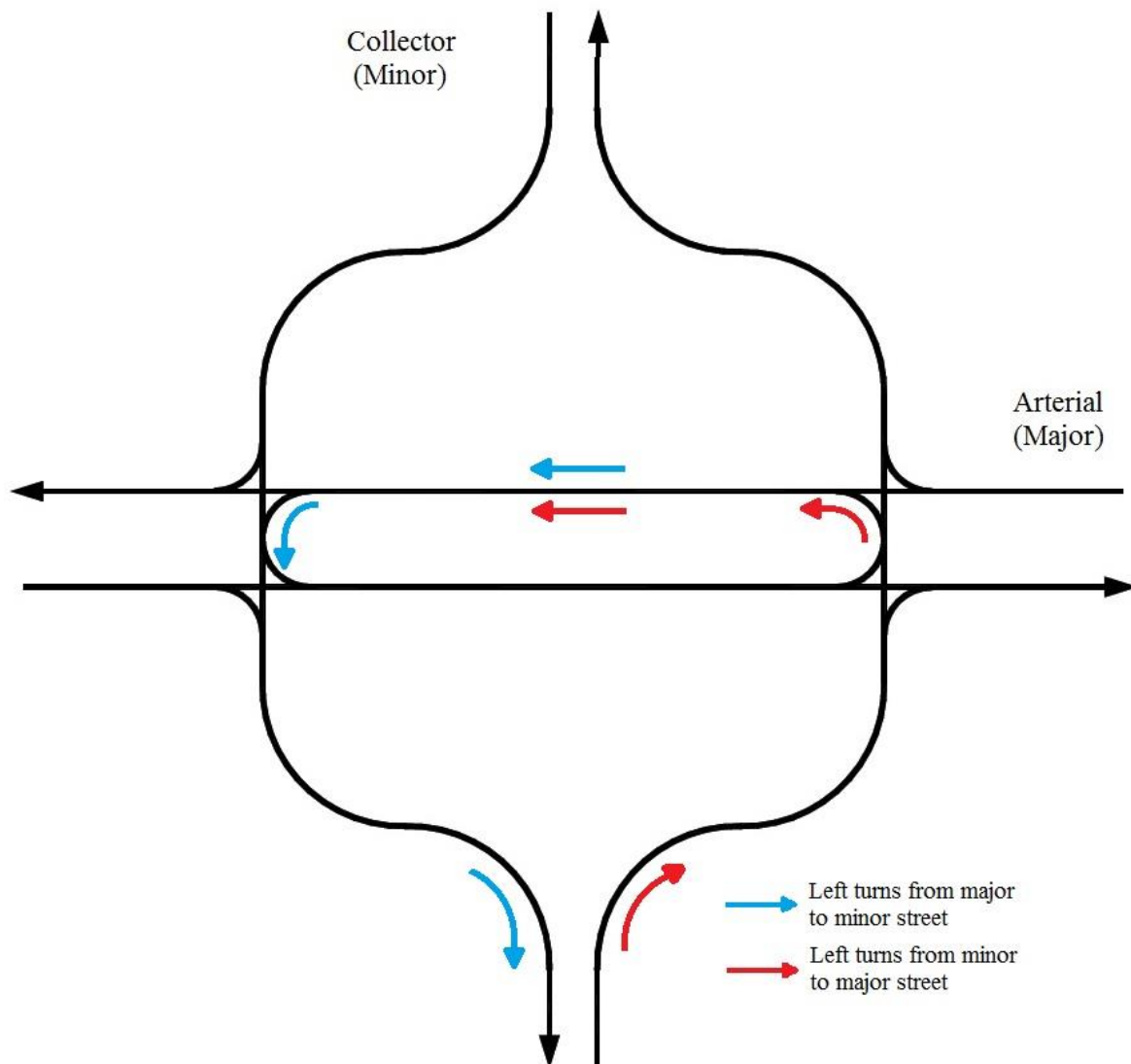
Figure 8: Split Intersection

The separation of flows reduces delay and eliminates turning conflicts compared to a conventional four-legged intersection. The majority of the delay reduction results from the elimination of one of the four traffic-signal phases of the intersections. This adds more green time to the cycle for left-turning vehicles. Reducing the number of conflicts between left turning and through vehicles has been shown to increase safety. The main disadvantages of the Split

intersection are the high initial cost, right-of-way acquisition, and possible wrong-way movements by unfamiliar drivers. Split intersection can also be achieved by separating flows for both the major and minor roadway (or two roadways of the same class). In that case, it is known as the Town Center Intersection, Couplet or the Square-about. The Split intersection is a common design in New Jersey.

GEOMETRIC DESIGN CONSIDERATIONS REVIEW: STANDARDS, PRACTICES AND RECOMMENDATIONS

This section presents the most important geometric design considerations for innovative intersection designs, based primarily on the summary of FHWA's Alternative Intersections Informational report, published in 2010, and some other relevant sources. This FHWA report includes geometric design, access management, traffic signalization, multimodal users' accommodation, operational performance, safety performance, construction costs and other relevant considerations for four types of innovative designs: displaced left-turn intersection, median U-turn intersection, superstreet intersection, and quadrant roadway intersection. Other innovative intersections and interchanges are briefly discussed in the FHWA report.

Diverging Diamond Interchange (DDI)

Geometry of a DDI

The primary design element of a DDI interchange, shown in Figure 9, is the relocation of the left-turn and through movements to the opposite side of the road within the bridge structure. There are two on-ramps and two off-ramps that connect the crossroad and the freeway. The off-ramps have two left-turn lanes and one right-turn lane. One left-turn lane and one right-turn lane lead to the on-ramp. The arterial has one through lane, one through+left-turn lane, and one dedicated right-turn lane. The movements can be better understood by following the arrow markings in Figure 9. Two signalized intersections (A and B) are situated at the two crossover locations. The radii of the curves are usually in the range of 150 to 200 ft. In rural high-speed environments, the nature of this directional crossing of through flows may be hazardous. A suggested forgiving design could provide curved approaches to motivate speed reduction by heightening drivers' awareness. In addition, the directional crossings are made more perpendicular and occupy shorter crossing distances. The conventional diamond interchange that is compared with the DDI has the following design. On-ramps and off-ramps are exactly the same as DDI, but there is a change in the number of lanes on the arterial. It has two through lanes, one dedicated left-turn lane, and one dedicated right-turn lane. Clearly, the section between the ramps needs more right-of-way as compared with the DDI (two extra left-turn lanes) (*8*).
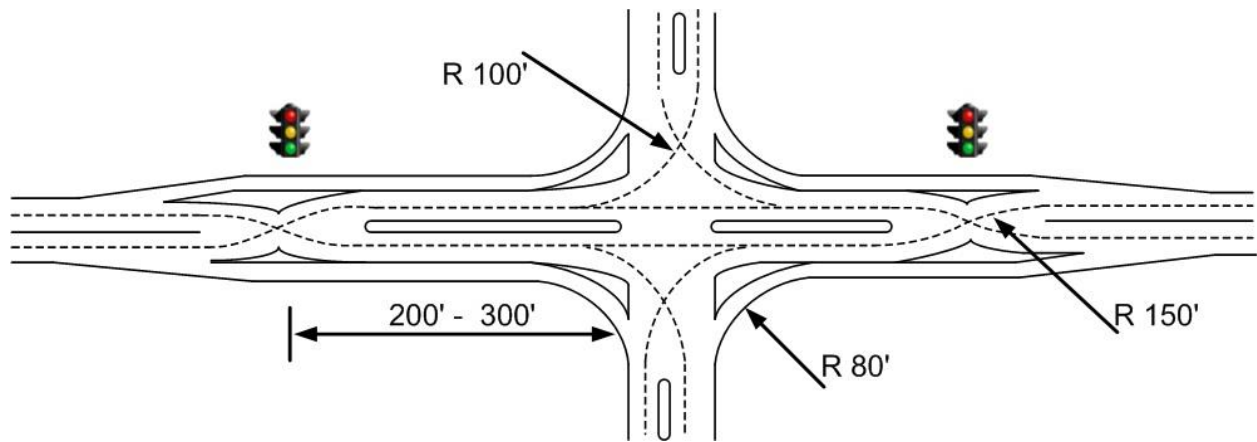
Figure 9: Diverging Diamond Interchange Layout

The turning radii used at the crossover junction to displace the left turn and through movements are around 300 ft (*9*). Consideration should be given to designing radii at crossovers with heavy vehicles in mind. On rural locations where the minor street has high-speed limits, the use of reverse curvature has been suggested. This may result in loon-like flare-outs at the ends of the bridge structure, as shown in Figure 10. Additional right-of-way may be required to widen the bridge or the underpass structure.
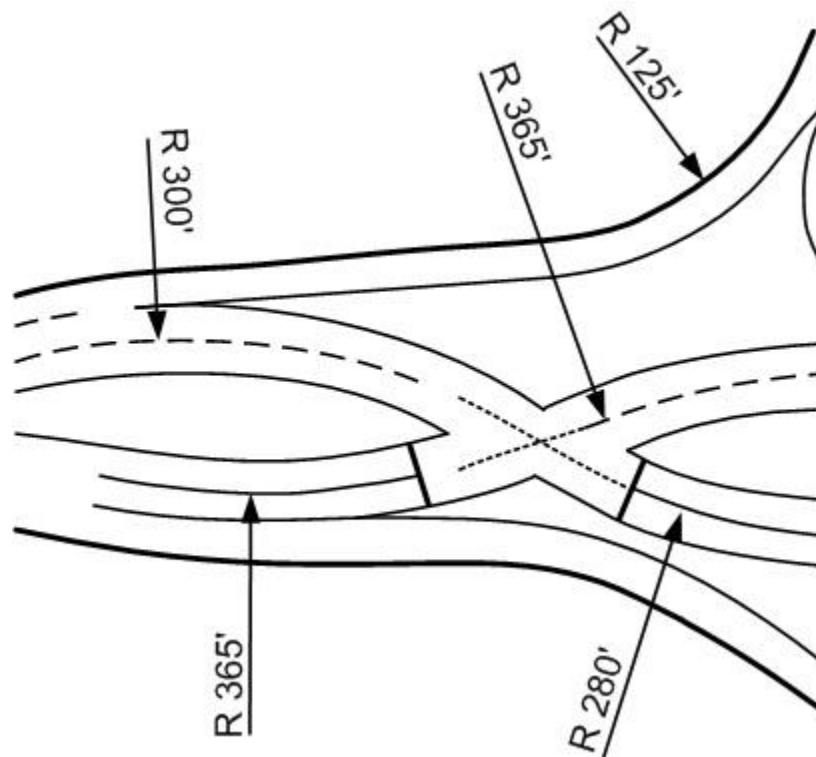
Figure 10: Crossover Movement in a DDI

Median width is also an important design element for a DDI. Greater median width is required for the flaring needed for reverse curves. Designers can obtain minimum median widths from the

AASHTO Green Book. Designers should also take into account the installation of post-mounted signs on medians on the bridge deck for safe and effective channelization of traffic. Appropriate offsets for signs should be in accordance with the MUTCD. Driver simulator experiments on the DDI, which included the use of glare screens, showed no erroneous maneuvers by tested subject drivers.

The Missouri Department of Transportation (MoDOT) performed extensive analysis on the benefits of the DDI alternative compared to a tight urban diamond interchange (TUDI). Some of the conclusions from comparing the two alternatives for this location are as follows:

- The DDI design reduces the number of lanes required under bridges from five to four, eliminating the need to build retaining walls for the specific interchange.
- The DDI design reduces the number of lanes needed on cross streets beyond the interchange.
- The DDI design has more storage capacity between the ramp terminals – 550 ft for a DDI versus 350 ft in a compressed diamond.
- The DDI design provides better sight distance. With this mainline over situation, bridge columns do not block the views of left-turning drivers to oncoming traffic as they wait to turn left onto the on-ramp.
- The DDI incorporates geometry, which has traffic-calming features, by reducing speeds while increasing throughput. This should result in fewer and less severe crashes.

Some suggested design practices, based on MoDOT input, include the following:
- The minimum crossing angle of intersection should be 40 degrees.
- The radius design should accommodate between 25 and 30 mph.
- Superelevation may not be needed because it could detract from any desired traffic calming effect.
- Lane width should be around 15 ft.
- Design should accommodate WB-67 trucks.
- Adequate lighting should be provided.
- Nearside signals should be considered.
- DCD interchange designs may only be appropriate where there are high-turning volumes.
- Nearby intersections with high cycle lengths should be avoided.
- Pedestrians at free-turning movements should be evaluated, and pedestrian signals may be needed.
- The noses of the median island should extend beyond the off-ramp terminals to improve channelization and prevent erroneous maneuvers.
- Left- and right-turn bays should be designed to allow for separate signal phases.

Cross-Over Intersection Design Considerations

Since one of the keys to success of the DDI is the ability to coordinate the signal timing of the two cross-over signals, the ability to coordinate those signals with one another is directly related to the distance those signals are apart. A good rule of thumb for the spacing between the two cross-over intersections is about 800 to 1,000 feet (*10*). This provides sufficient space for queue storage and the ability to move traffic through the system.

The approach angle for cross-over intersections of a DDI should be 30 degrees or greater. There should be a balance between providing a smooth transition through the cross-over and meeting driver expectancy of a square intersection. If the angle is too flat, drivers may be confused and run down the wrong side of the road.

Barrier should be placed with sharp angles at the corners of the cross-over intersections where right turns are to be prevented. These sharp edges, as opposed to the normally rounded corners, help emphasize the need to go straight. Once traffic is on the opposite side of the road, barrier should separate traffic and if possible block the view of oncoming headlights and traffic.

Lane configuration

Situations may arise in which turning volumes are very heavy or are metered and there is concern that the left turn traffic along the crossroad may interfere with through traffic. If additional storage is needed, the left turns can be pulled out of the through traffic stream even farther in advance of the interchange area (*3*).

When designing a DDI, for operational purposes, it may be necessary to carry an asymmetric number of lanes over the freeway. The example that follows is a DDI proposed at Ashland, Oregon and has two eastbound lanes and one westbound lane over I-5. This configuration was chosen due to the relatively higher volume of left turn traffic to through traffic. Note that the eastbound left lane becomes a trap lane that leads to the northbound entrance ramp. Had this design been constructed, care would have been necessary to properly sign the appropriate lane usage and guide drivers to their desired destination.

Profiles and Grade Separation

Where possible, DDIs should be built as flat as possible. Crest vertical curves must be checked to verify that adequate sight distance is provided. Sight distance is critical at any interchange, but more so at an interchange type that drivers are not familiar with (*3*).

An interesting potential modification to a DDI in the event of significant through volumes is to grade separate the crossing intersection. This would require widely spaced ramp terminals as well as significant modifications to the ramp alignments. Another potential issue that arises is that all signalization can be removed from the intersection making it far less friendly to pedestrians and also potentially introducing merging and weaving issues between the ramp terminals. This modification to the DDI concept has never been constructed and additional study would be necessary prior to undertaking such a design though it would appear to have its benefits and drawbacks and may apply to specific situations.

Crossing Intersection Angle

FHWA's initial guidance to the MoDOT during the Kansas City design was to achieve a 45 degree angle of intersection. Due to space constraints requiring a narrow median and cross section to fit beneath the existing underpass and to minimize impacts to existing properties, the MoDOT design determined that an angle of intersection closer to 25 degrees was adequate. The Oregon DOT initially attempted to obtain a 40 degree angle for the Fern Valley interchange (I-5 Exit 24). The final design at that site also ended with a 25 degree angle (*3*).

As with other intersections, ideally, the closer the intersection angle is to 90 degrees, the better. For DDIs, a 40 degree angle of intersection is desirable. There is no minimum angle of intersection that has been established, though the conventional thinking is that larger angles of intersection should reduce the likelihood of wrong way travel at the crossing intersections and would also minimize the actual crossing distance which would decrease the clearance time required for signal phase changes.

Note that there are existing intersections, not necessarily at interchanges, that have angles that are much smaller than 20 degrees that have been operating without causing the safety concerns that conventional thinking would imply should exist at such an acute angle. These intersections rely on the existing guidance provided by state and federal standards for signing and striping, such as the MUTCD, to provide adequate guidance to the traveling public.

Crossroad to Freeway Intersection Angle

Unlike more traditional diamond interchanges and single point urban interchanges (SPUIs), DDIs do not require the crossroad and freeway to intersect at a 90 degree angle. If the angle is not perpendicular, as in Figure 10, the turns to and from the ramps take up more space than they would if the two roadways intersected at 90 degrees (*3*).

Ramp Terminal Separation Distance

The minimum distance between ramp terminals will be governed by design, traffic operations, and site conditions. The minimum distance between ramp terminals should be sufficient to allow for proper horizontal alignment design (*3*). Rural horizontal design should include a tangent section for the crossroad in the middle of the intersection that is long enough to resolve superelevation and allow for at least a two second normal crown section at the design speed or treatment as in reverse curves where the tangent length is not possible. The urban convention of curvature and normal crown accepting less comfortable conditions could prevail in urban areas and be consistent with the system practice. The tighter the ramp terminals are together, the more difficult it will be to provide offset signal timing that will allow for continuous flow in each direction. While this is not a requirement, an analysis using traffic microsimulation will indentify if this will cause queuing problems into adjacent intersections.

Right-of-Way

Right-of-way requirements will vary greatly from site to site and are heavily dependent on the design speed of the reversing curves on the crossroad (*3*). In general, the DDI results in a narrower crossing of the freeway and fewer lanes on the crossroad approaches. The lack of left turn bays plays a significant part in the narrow cross section. However, the reversing curves may require right-of-way widths that may exceed other alternatives, depending on the design speed selected.

Access Control

While every agency has access control guidelines dictating the desired minimum distance from a ramp to the nearest access point, in developed areas, it is difficult, if not politically impossible, to achieve the standards set forth. In the Kansas City DDI, existing right-in-right-out access points will be maintained as close as 100 feet from the ramp terminals. Existing signals are located 400-500 feet away from the ramp terminals (*3*).

During the conceptual layout of the Ashland DDI, while the impacts and shifting of access seemed extensive, when compared to a diamond interchange or SPUI, the access impacts were comparable. Major factors that influence the impact on access control that a DDI would have are largely governed by the individual state's design standards regarding placement and length of barrier medians. For example, for a standard diamond interchange in Oregon with dual left turn lanes, no physical barrier is required on the approaches. However, a similar interchange in Illinois would require non-mountable barrier curb on the approaches to the interchange as opposed to a painted median.

Design Vehicle/Lane widths

Lane width should be governed by the applicable roadway standard (typically 12' minimum) but may be wider in certain portions of the interchanges to ensure that a design vehicle (typically a WB-67) in each lane of the design can make the movements required without encroaching into the adjacent lane (if there is one) (*3*). Turning templates should be applied on the turns on to and off of the ramps as well as the reverse curvature on the cross road to ensure safe operation of the design vehicle.

Design Speed and Reverse Curvature

The design speed of the reverse curve for the through movement should be governed by the posted speed on the approaching roadway. The design speed of the curves should be less than the design speed of the approaching roadway. This speed reduction should be no greater than 15 mph and preferably only 10 mph (*3*). Speed differential greater than this may pose a safety risk for off-peak drivers and may increase the likelihood vehicles leaving the roadway as they enter the reverse curves at speeds that are too fast for the curvature. The Missouri DOT (MoDOT) DDI (I-270 and Front Street) had a through movement design speed of 25 mph with posted

approach speeds of 40 mph. A slower design speed was also examined, but a traffic simulation showed that the high percentage of trucks in the vehicle stream with a 20 mph speed began to impair the operations of the interchange.

DDIs can be used with approach speeds greater than 40 mph; however, this increases the design speed of the reverse curvature and significantly increases the footprint as the median width increases to accommodate the higher speed reverse curvature.

The key elements of a DDI are interrelated, further enforcing the fact that DDIs are very site-specific in their design characteristics. While the overall concept remains the same from one to another, specific measures, such as the design speed, radii of the reverse curves, median width, etc., may vary significantly from one location to another.

The design speed of the curves coming from the ramps need not be high. At a standard diamond interchange, these ramp movements are essentially stop condition type movements. Turning speeds of 10 mph would be acceptable, but more likely than not, the design speed will be governed by traffic operations requirements as well as the design vehicle. Turning speeds for DDIs that have progressed into the design phase are in the 10-20 mph range. The Kansas City DDI has dual left turn lanes from the ramps that are designed to accommodate side-by-side WB-67 trucks turning at 10-15 mph.

Continuous Flow Intersection (CFI)

Basic Geometric Design Considerations

Figure 11 illustrates typical designs for CFI intersections. The design in Figure 11 is for a full version, which has displaced left turn (DLT) movements on all four approaches (for this reason, CFI is also known as CFIs – Displaced Left Turns). This design reflects a shift of the through traffic lanes into the median in an attempt to minimize the need for additional right-of- way. At several locations where CFIs have been implemented as a retrofit to an existing conventional at-grade intersection, the existing median has been preserved, and there is no shift in the through lanes (*9*). CFIs are also being implemented as partial or three-legged CFI designs.
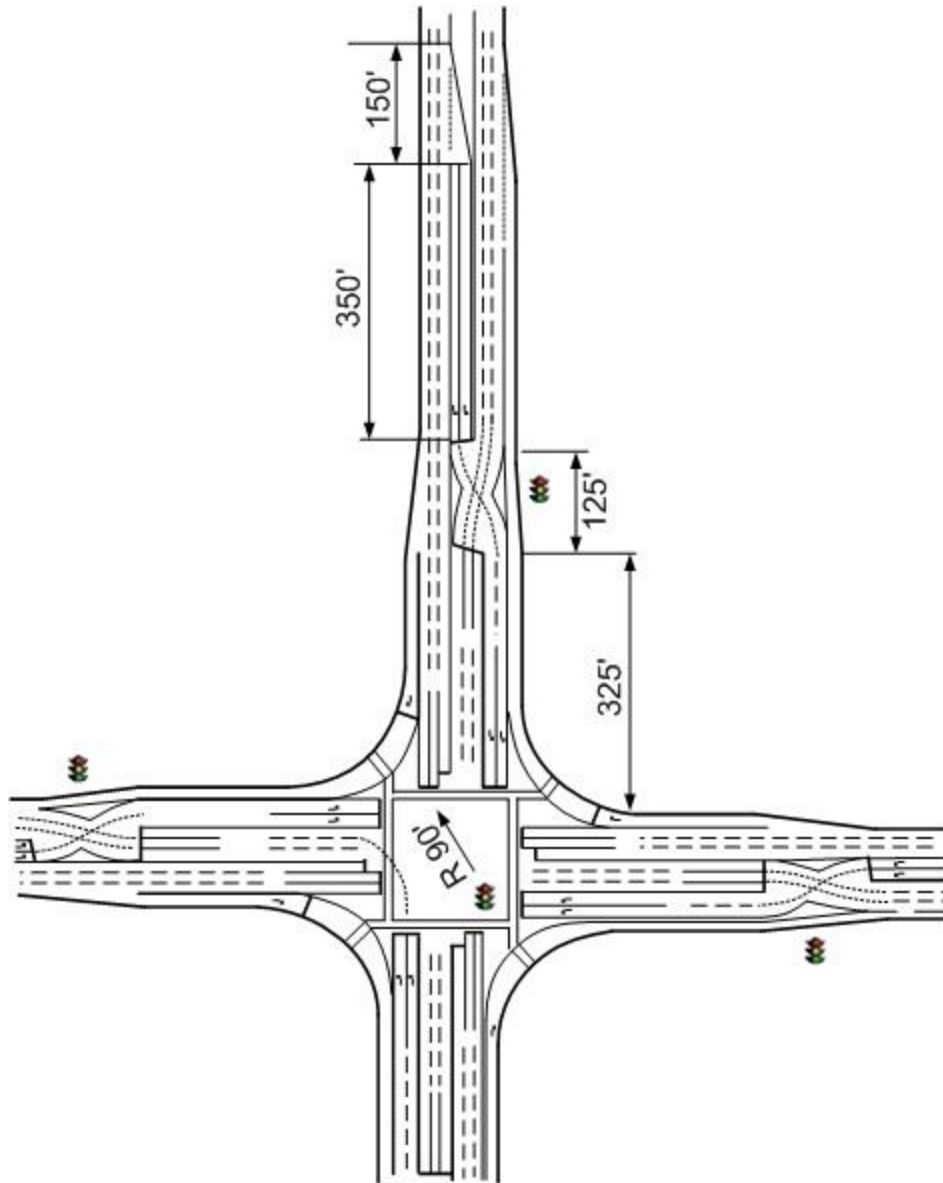
Figure 11: Typical Full CFI with DLTs on all Approaches

Removal of conflict between the left-turn movement and the oncoming traffic at the main intersection is the primary design element in a CFI. The DLT vehicles typically cross the opposing through traffic approximately 300–400 ft upstream of the main intersection under the control of another traffic signal as shown in Figure 12. Research performed by the Maryland State Highway Administration (MDSHA) shows that the appropriate upstream distance is dependent on queuing from the main intersection and on costs involved in constructing a left-turn storage area for the crossed-over left turn movement. Radii of the crossover movements can range from 150 to 200 ft, while the radius of the next left-turn movement at the main intersection is dependent on the turning movement of the design vehicle. Lane widths at the crossover reverse curve should be wider than 12 ft to accommodate larger design vehicles. Consideration should also be given to having wider lane widths (up to 15 ft) for the receiving crossroad. The

angle between the CFI left-turn lanes and the main through lanes is referred to as the crossover angle and is influenced by the median width and the alignment of the mainline lanes. The Louisiana Department of Transportation and Development (LA DOTD) recommends an angle of 10–15 degrees.
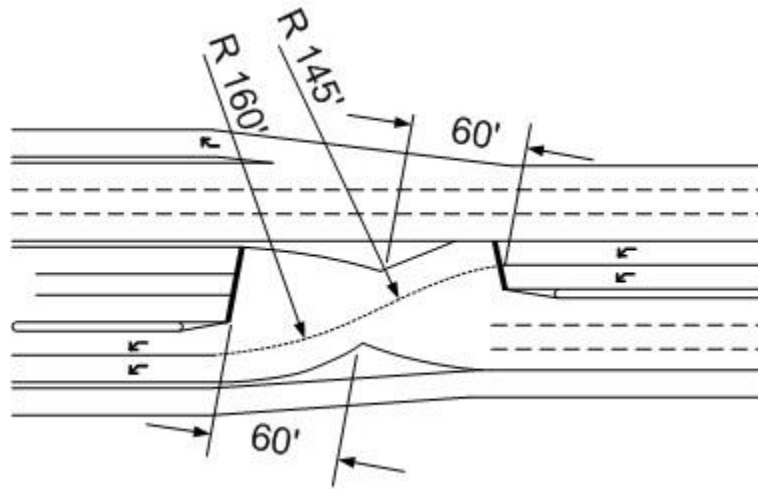
Figure 12: Left-turn Crossover Movement in a CFI

Right-of-way constraints are an issue common in urban environments. The CFI design helps minimize right-of-way acquisition by occupying far less space compared to grade separated interchanges. However, due to the presence of left-turn crossovers, a CFI has a larger footprint compared to a conventional at-grade intersection. To minimize the footprint, median widths can be reduced, but they still need to be adequate to accommodate signs. Designers can obtain minimum median widths from the AASHTO Green Book. Designers should also take into account the possibility of installing post-mounted signs in these medians for safe and effective channelization of traffic. Offsets for signs should be in accordance with the MUTCD. A wide median can be counterproductive for several reasons, including the following:

- Wide medians can result in large walking distances for pedestrians at the intersection. This can result in long pedestrian clearance intervals, which can be counterproductive to the efficient signal operation.
- Wide medians resulting in a wide intersection footprint lead to longer yellow and all-red clearance times for the intersection and consequently longer cycle lengths.

If the existing arterial has a wide median, the median can be narrowed through the use of transition curves and guidance from the AASHTO Green Book. Similarly, minimum turning radius criteria for the appropriate design vehicles and shoulder placement can be obtained from the AASHTO Green Book and applied as appropriate. NCHRP Synthesis 225, "Left-Turn Treatments at Intersections—A Synthesis of Highway Practice," describes several design features for CFIs including channelizing islands, overhead lane controls, and raised pavement markers for lane delineation and traffic flow separation. With the elimination of left-turn

movements at the main intersection, U-turns should also be prohibited at the main intersection of a CFI. However, if the median's width is sufficient, then U-turn movements on the major road can be executed at the left-turn crossover. Designers of the CFI in Baton Rouge, LA, implemented a U-turn crossover with truck restrictions between the main intersection and the left-turn crossover. Sight distance and driver expectancy are other issues related to the design of a CFI. Left-turning drivers may be confused when they negotiate the CFI. This can be counterintuitive to unfamiliar drivers. Hence, unambiguous signing is needed. The CFI in Louisiana was designed and constructed based on the following criteria:

- Design speed of 50 mph with 12-ft lanes and 8-ft shoulders on U.S. 61 (Airline Highway).
- Lane width of 12 ft was on all lanes except the frontage roads.
- The median width on U.S. 61 (Airline Highway) was 43 ft.
- Shoulders of 8 ft in width on both sides of U.S. 61.
- The separation between the left-turn crossover and the opposing through traffic was 20 ft.

A 12-ft-wide separation was maintained between the left-turn crossover and the opposing right turning traffic. Some of the other design guidelines used in the Louisiana CFI were as follows:

- The angle of crossing for DLT vehicles was as great as possible to help reduce the possibility of wrong-way entry and to reduce crossing time.
- Right-turn lanes were provided on intersection legs approaching DLT roadways.

Widening or adding lanes at a CFI in the future could be difficult. Additional lanes that may be needed in the future should be planned during the initial design of a CFI.

In summary, the key characteristics of the implemented CFI designs are as follows (*11*):

- Left-turning vehicles are removed from conflict at the main intersection by having them move across the opposing through traffic stream at a signal-controlled crossover 300 to 400 ft upstream of the main intersection.
- Crossover movement radii can range from 200 to 400 ft.
- Access limitations in the vicinity of CFI inter-sections are likely, as some State design manuals preclude median breaks within 600 to 700 ft of the intersection. Also, driveways near the inter-section have to be right-in and right-out.
- Pedestrians can be accommodated at CFIs at the main intersection

Median U-Turn Intersection (MUT)

Basic Geometric Design Considerations

The Median U-Turn (MUT) intersection (known in Utah as Thru-Turn intersection) performs well on arterials that have sufficient median width to accommodate the U-turn maneuver (*9*). In

general, corridors with MUT intersections have median widths ranging from 60 to 100 ft. This design is used as a corridor treatment in Michigan, although it has been used successfully for isolated intersections. Figure 13 shows a design for a typical four-legged MUT intersection.
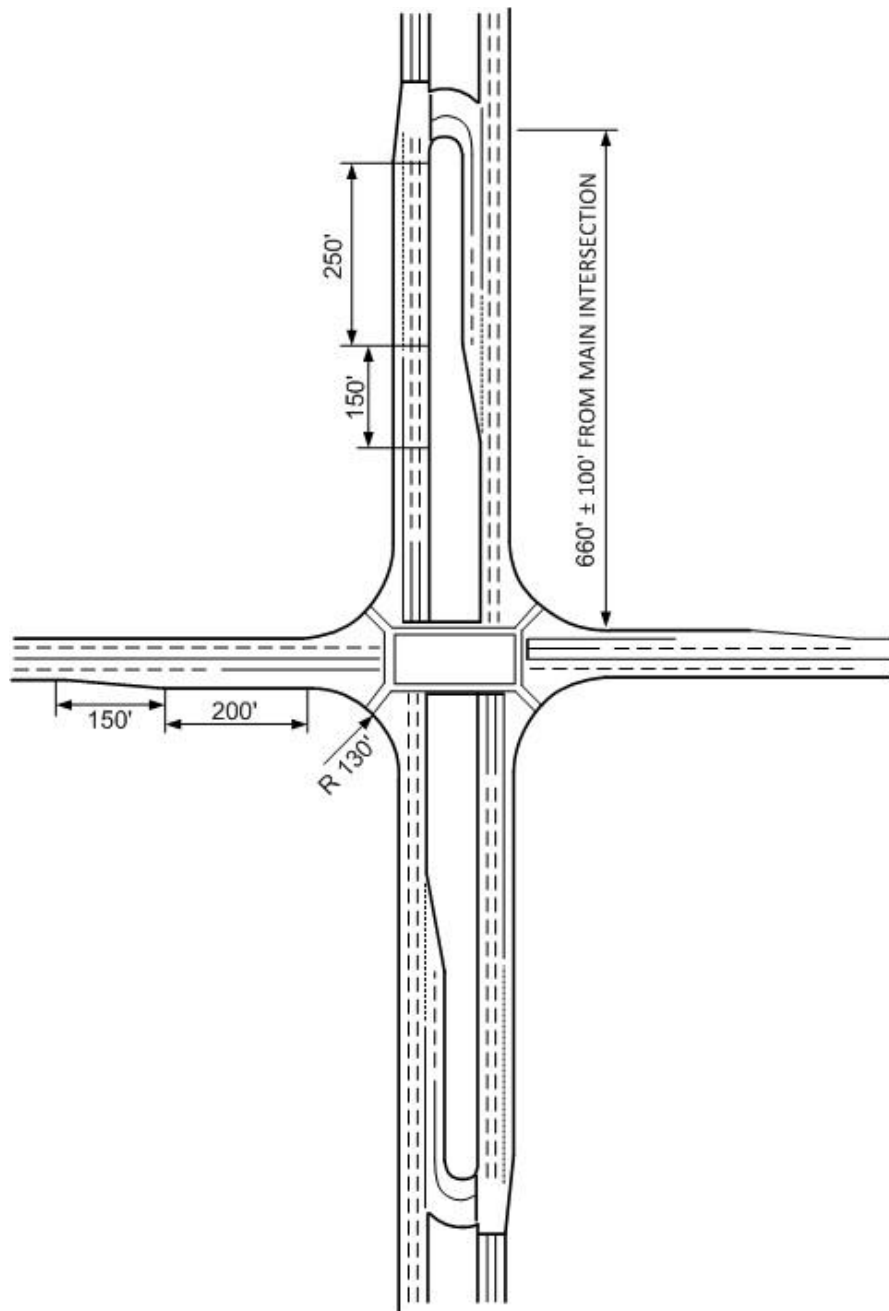
Figure 13: Layout of an MUT Intersection

At an MUT intersection, the design of the main intersection is similar to the design of a conventional intersection. The main intersection is designed for larger volumes of right-turn movements than a conventional intersection serving the same total volumes since the left-turning vehicles become right-turning vehicles. Because of this, the intersection must be designed with

right-turn bays of sufficient width and length to accommodate the volume of turning vehicles. Depending on the right-turn volume, dual right-turn lanes or an exclusive right-turn lane and an adjacent shared-use through and right-turn lane may be needed.

Channelized right turns at an MUT intersection are rarely used, because they may require even more right-of-way, present a multistage pedestrian crossing, and create a more difficult driving maneuver for a driver turning right from the minor street and weaving over to use the U-turn crossover. At some MUT intersections (e.g., at partial MUT intersections), left turns from the side road are allowed as well as left-turn bays provided on the minor road approaches.

The MUT intersection has secondary intersections at each of the crossover locations. One-way crossovers with deceleration/storage lanes are highly recommended. Several studies have found that one-way (directional) median crossovers provide better traffic operations and safety performance than two-way (bidirectional) crossovers.

The Michigan DOT (MDOT) has developed design guidelines for directional median crossovers. Figure 14 and Figure 15 illustrate MDOT guidelines for designing directional median crossovers and show one-lane crossovers. In Michigan, it is customary for drivers of passenger vehicles to queue side-by-side in a 30-ft wide crossover and treat it as if it had two lanes. However, large trucks and other heavy vehicles typically use the entire width of the crossover. MDOT uses striped two-lane crossovers (with two lanes of storage leading up to the crossover) in some places. These crossovers are typically 36 ft wide.
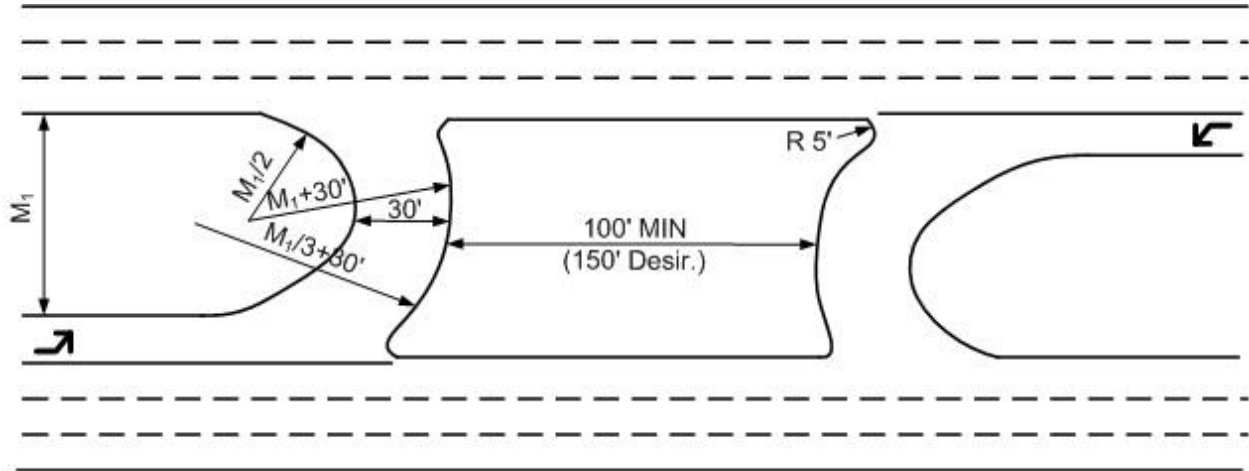
Figure 14: Directional Crossover Design on Highway with Curbs
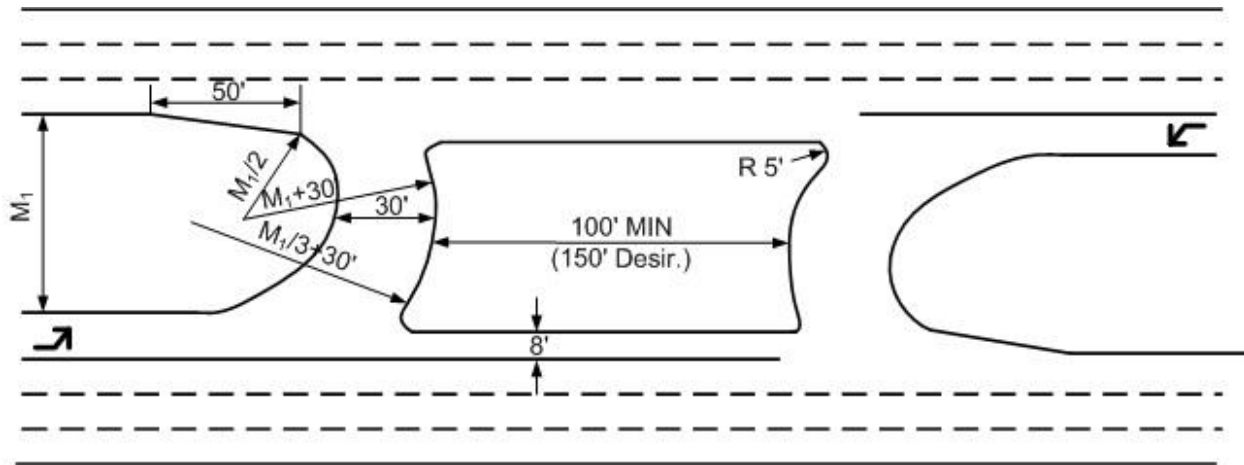
Figure 15: Directional Crossover Design on Highway without Curbs

The AASHTO Green Book provides values for minimum median width based on the needs of U-turning design vehicles. The design vehicle and number of opposing lanes directly govern the required median width at the median crossover junction. Median widths between 47 and 71 ft typically result from the choice of a large design vehicle and the desire to accommodate a U-turn maneuver of that vehicle without encroaching on outside curbs or shoulders. Assuming 12-ft-wide lanes and right-of-way limits that are 10 ft wide beyond the edge of the travelway, the right-of-way for boulevards with U-turns can range from 139 ft for four-lane arterials to 163 ft for eight-lane arterials.

There are several ways to accommodate these MUT intersections if sufficient right-of-way is not available to accommodate a wide median. One method of reducing the median width is to allow vehicles to turn onto the existing or widened shoulder, which could have strengthened pavement. Another method is to add pavement outside the travel lane to allow the design vehicle to complete the U-turn maneuver and merge back into the traffic stream. The additional pavement is typically referred to as loon. Loons are generally defined as expanded paved aprons opposite a median crossover. Figure 16 shows a schematic diagram of a loon design.
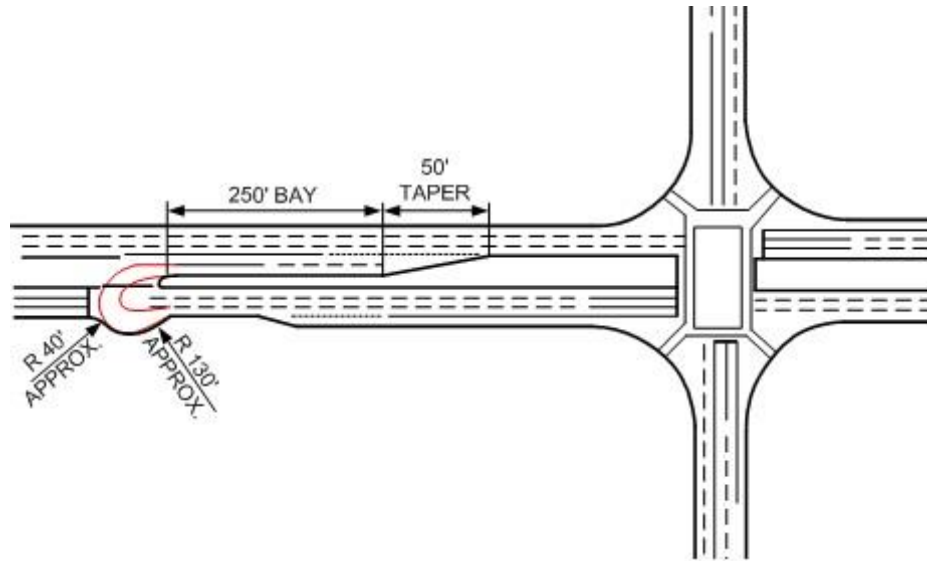
Figure 16: Loon Implementation for an MUT Intersection

Figure 17 shows a design in which the median widens in the vicinity of the crossover to better accommodate U-turns. The reverse curves used to accomplish the widening and narrowing should be gentle enough so as to not force drivers to execute unexpected sharp maneuvers as they proceed through the curves.
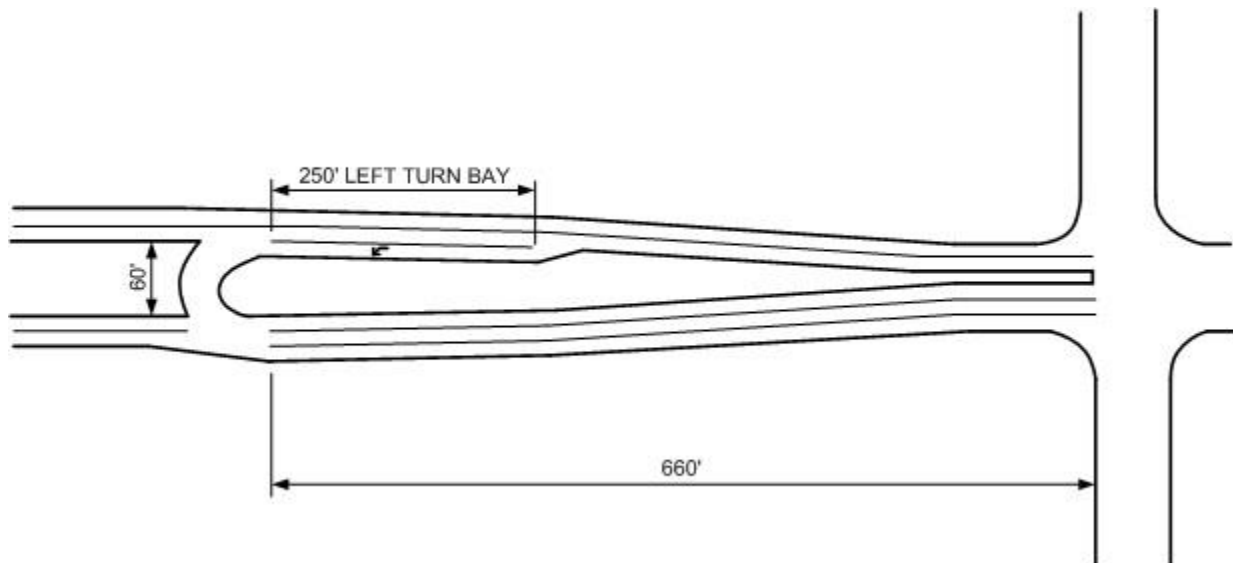
Figure 17: Example of a Transition from a Wide Median Section to a Narrow Median Section on MUT Intersection Corridors

Another way to use an MUT intersection design while keeping the main street median narrow is to place the U-turn crossovers on the minor street. Studies showed that this variation may introduce travel time benefits compared to the common design with crossovers on the main road. U-turn crossovers on the minor street mean that left turns from the main street are initiated with a

right turn, which may violate driver expectations. As a result, adequate signing is critical in these cases.

The AASHTO Green Book recommends a distance of 400 to 600 ft for the minimum spacing between the median crossover and the main intersection. MDOT recommends a distance of 660 ft ±100 ft for the median crossover from the MUT intersection. The distances recommended by MDOT were established to accommodate drivers desiring to turn left from the crossroad. The longer distance facilitates the completion of the U-turn maneuver at the median crossover and subsequent right-turn maneuver at the intersection of the major road and cross street for a 45 mph posted speed limit on the major road. The Access Management Manual recommends an access spacing of 660 ft on minor arterials and 1,320 ft on principal arterials between consecutive directional median openings on divided highways.

Designers should consider several issues when determining the distance from a main intersection to the median U-turn crossover. Longer distances to crossovers decrease probability of main road queues at the main intersection for the opposing direction of travel to block the crossover. They also provide more time and space for signs to be seen and read and for drivers to maneuver into the proper lane. Shorter distances to crossovers mean shorter driving distances and travel times and lower volumes at each crossover because each serve fewer driveways between the main intersection and the crossover. The selection of the spacing from the median crossover to the intersection is also a tradeoff between preventing spillback from the main intersection and the adverse impacts of additional travel for the left-turning vehicles.

Turn bays leading into U-turn crossovers are typically at least 250 ft long to provide adequate deceleration and storage. They may be longer when speeds are higher and U-turning demands are greater. In Michigan, to provide adequate storage, there are some MUT intersections where the turn bay for the crossover actually begins prior to the main intersection, at the prior crossover, or even before the prior crossover. Careful consideration of curb radii design, signing, and marking are needed at these locations so that drivers do not attempt to execute direct left turns at the main intersection.

Median U-turn designs also require significantly larger rights-of-way along the major street (AASHTO recommends a 60 foot median to accommodate large trucks) and require the use of multiple signal installations (typically three, one for the main intersection and one for each of the two median crossovers) instead of just one (*12*). Otherwise additional pavement should be added to the outside travel lane to safely complete the U-turn maneuver.

Spacing of Median Opening

The Green Book makes the following recommendations on the spacing of median openings (*13*):

- Spacing between median openings should be adequate to allow for introduction of left-turn lanes.

- Median openings should reflect street or block spacing and the access classification of the roadway.
- Full median openings should be consistent with traffic signal spacing criteria.
- Spacing of openings should be consistent with access management classifications of criteria.

Research reported in NCHRP Report 348 (*14*) indicates that several states have set median opening spacing criteria that range from 330 to 2,640 ft. These criteria are mainly applicable in suburban and rural environments. The report also presents minimum desired spacing of unsignalized median openings at driveways as a function of speed. This spacing ranges from 370 ft at 30 mph to 910 ft at 55 mph. In addition, the report suggests the following guidelines be considered for the spacing and design of median openings on divided roadways:

- The spacing of median openings for signalized driveways should reflect traffic signal coordination requirements and the storage space needed for left turns.
- The spacing of median openings for unsignalized driveways should be based on a roadway's function or access level and the environment in which the roadway is located (e.g., rural) and should be conducive to signalization.
- Median openings for left-turn entrances should be spaced to allow sufficient storage for left-turning vehicles.
- Median openings at driveways could be subject to closure where volumes warrant signals, but signal spacing would be inappropriate.
- Median openings should be set back far enough from nearby signalized intersections to avoid possible interference with intersection queues, and storage for left turns must be adequate.

TRB Circular 456 (*15*) indicates that median openings generally should relate to the street or block spacing. Thus, where cross-streets are placed at regular intervals, these intervals will influence median opening spacing. The Circular recommends that access points on both sides of the road should be aligned on undivided highways. Where this is not possible, sufficient left-turn storage should be provided by establishing a minimum offset distance. Driveways should be offset from median openings by the following:

- At least 200 ft when two low-volume traffic generators are involved,
- The greater of 200 ft or the established median opening spacing interval when one major traffic generator is involved, and
- At least two times the established median opening spacing interval when two major traffic generators are involved.

NCHRP Report 375 (*16*) found that very few state highway agencies have formal policies on the minimum spacing between median openings. Those agencies that do have criteria generally use a

spacing between median openings in the range from 0.25 to 0.5 mi. The Florida DOT identifies the following factors that should be considered in determining the spacing of median openings:

- Deceleration length,
- Queue storage,
- Turn radius, and
- Perception/reaction distance.

Based on consideration of all of these factors, Florida has identified a 1,070-ft spacing between median openings as being a realistic minimum for urban arterials.

Median Width

The findings of the analysis concerning median width are as follows:

- At rural, four-leg, unsignalized intersections, accident frequency decreases as median width increases.
- At rural, three-leg, unsignalized intersections, no statistically significant relationship exists between accident frequency and median width.
- At urban/suburban, four-leg, unsignalized intersections, accident frequency increases with increasing median width over the range of median widths from 14 to 80 ft.
- At urban/suburban, three-leg, unsignalized intersections, the intersection accident frequency increases with increasing median width.

The Florida DOT suggests that the appropriate median width is a function of the purpose which the median is to serve in a particular application, such as the following:

- Separation of opposing traffic streams,
- Pedestrian refuge,
- Left turn to side street,
- Left turn out of side street,
- Crossing vehicles,
- U-turns, and
- Aesthetics and maintenance.

Table 1: Minimum and Recommended Median Widths Based on AASHTO Green Book

Roadway type	Speed (mph)	Median width (ft)	
Reconstruction project	≤ 40	15.5	Minimum
Reconstruction project	45	19.5	Minimum
Reconstruction project	50	22.0	Minimum
Four lane highways with medians expecting significant U-turns and directional median openings with excellent positive guidance	All	30.0 – single left turn 42.0 – dual left turns	Recommended
Six lane highways with medians expecting significant U-turns and directional median openings with excellent positive guidance	All	22.0 – single left turn 34.0 – dual left turns	Recommended

Superstreet Intersection

Geometric Design

The key difference between an MUT intersection and a Superstreet intersection (also known as a Restricted Crossing U-Turn intersection – RCUT) is that an MUT intersection allows through movements from the side street. A Superstreet intersection has either no median openings at the intersection or has one-way directional median openings to accommodate traffic making left turns from the main street onto the side street (*9, 17*).

Typical Applications of Superstreet Intersection

Figure 18 shows a design layout for typical four-legged Superstreet intersections. This design is for the more complex version which is more suitable for arterials with higher volumes. Should pedestrians be expected at intersections, these designs need to be modified to better accommodate them.
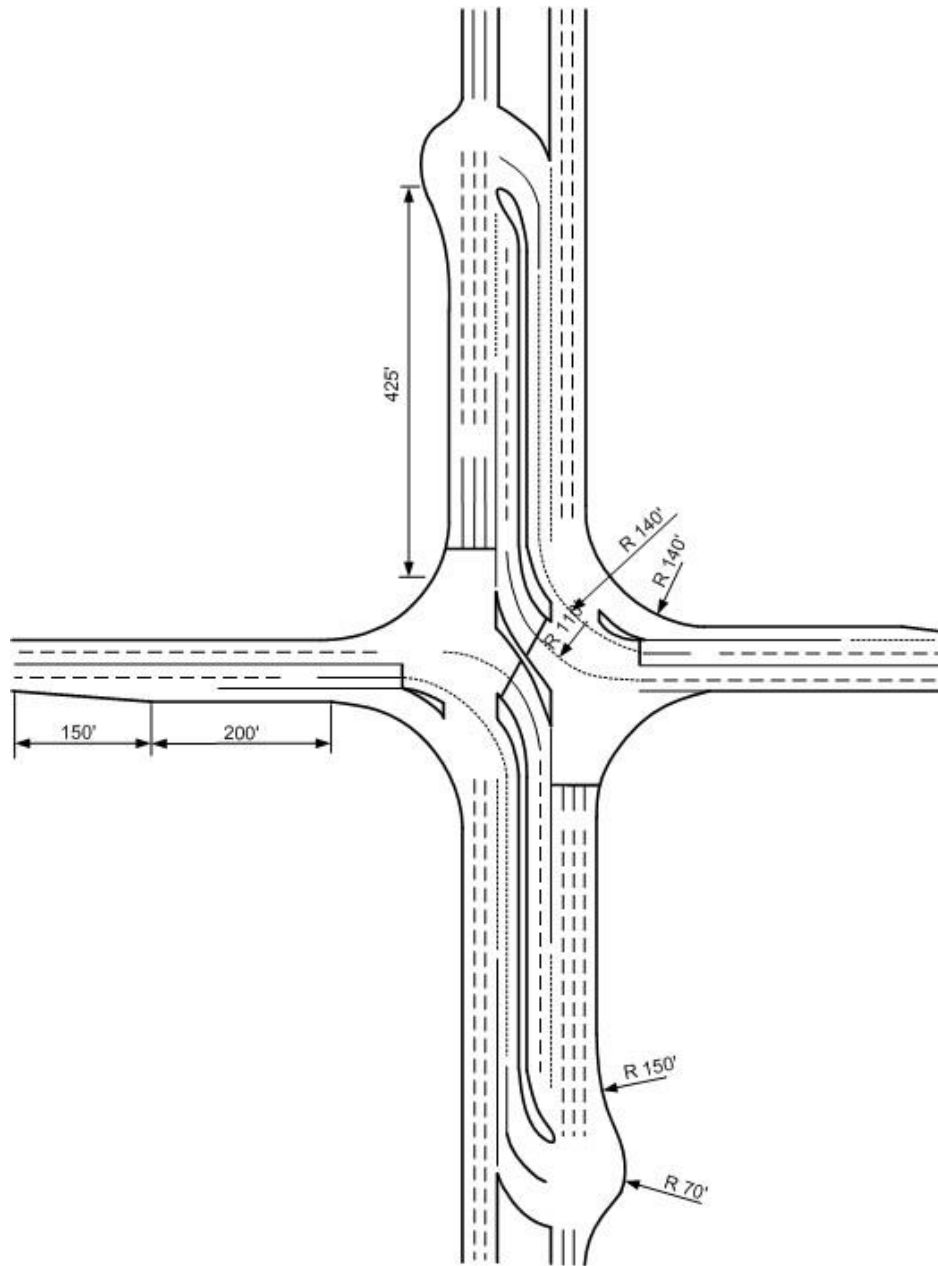
Figure 18: Layout of a Superstreet Intersection

Median Width and Crossover Spacing

Similar to the MUT intersection, the median width is a crucial design element for a Superstreet intersection. The desirable right-of-way widths needed to accommodate large trucks without allowing vehicles to encroach on curbs or shoulders, assuming 12-ft-wide lanes and 10 ft of shoulder, range from approximately 140 ft for four-lane arterials to approximately 165 ft for eight-lane arterials. For this same situation, desirable minimum median widths between 47 and 71 ft are typically needed. Much of the discussion of crossover spacing provided for MUTs applies to Superstreet intersections. The main points of the discussion included the following:

1. The first method of reducing right-of-way needs is to provide some median openings that only accommodate smaller vehicles. Proper highway signs need to be placed in appropriate locations to prohibit trucks at these crossovers.
2. A second method of reducing the amount of needed right-of-way is to allow vehicles to turn onto a shoulder, which has been strengthened with full-depth pavement.
3. A third way to reduce right-of-way is to provide bulb-outs or loons at the U-turn crossovers. A loon is an expanded paved apron opposite a median crossover. The purpose is to provide additional space to facilitate the larger turning path of a commercial vehicle along narrow medians.
4. A fourth method to reduce right-of-way width throughout a Superstreet intersection corridor is to use reverse curves on the main street through roadways to widen the median for a short distance at a crossover and then narrow it back down beyond the crossover. Drivers may not initially expect these alignment changes but could quickly adapt to the design. Using any of these methods means that medians do not have to be wider than 16 ft, which accommodates a minimum 4-ft-wide median and a 12-ft-wide turn bay along much of the length of a Superstreet intersection design. For these cases, the overall right-of-way required for a corridor of Superstreet intersections can be as narrow as 84 ft for four-lane arterials and as wide as 132 ft for eight-lane arterials.

Several factors should be considered when selecting the appropriate spacing from a main intersection to a U-turn crossover. Longer spacing between the main intersection and crossovers decreases spillback probabilities, providing more time and space for drivers to maneuver into the proper lane and read and respond to highway signs. Shorter spacing between the main intersection and crossovers translates into shorter driving distances and travel times. AASHTO recommends spacing from 400 to 600 ft for MUT designs based on signal timing. MDOT's experience with MUTs has led it to establish 660 ±100 ft as the standard spacing. NCDOT's standard minimum spacing between main Superstreet intersections and crossovers is 800 ft.

Designers have flexibility in selecting the crossover spacing. To accommodate constraints related to drainage, sight distances, or available right-of-way, crossovers are shifted toward or away from a main intersection with relatively minimal adverse effects on traffic operations. Locating a crossover so that vehicles can make U-turns or left turns into the driveway or side street is common practice. This treatment can prove beneficial at Superstreet intersections where the combination of main road turning volumes and driveway volumes do not have a significant impact on the major road through traffic.

Crossover Design

Designers may use one-lane or two-lane crossovers for U-turns depending on traffic volume demands and the number of receiving lanes. AASHTO's Green Book and the MDOT Geometric Design Guide 670 provide U-turn crossover design details for MUTs that also apply to Superstreet intersections. Figure 19 shows a typical movement of a heavy vehicle in a loon.

NCDOT recommends an outside turning radius of 100 ft for the major road left-turn crossover, as shown in Figure 20.
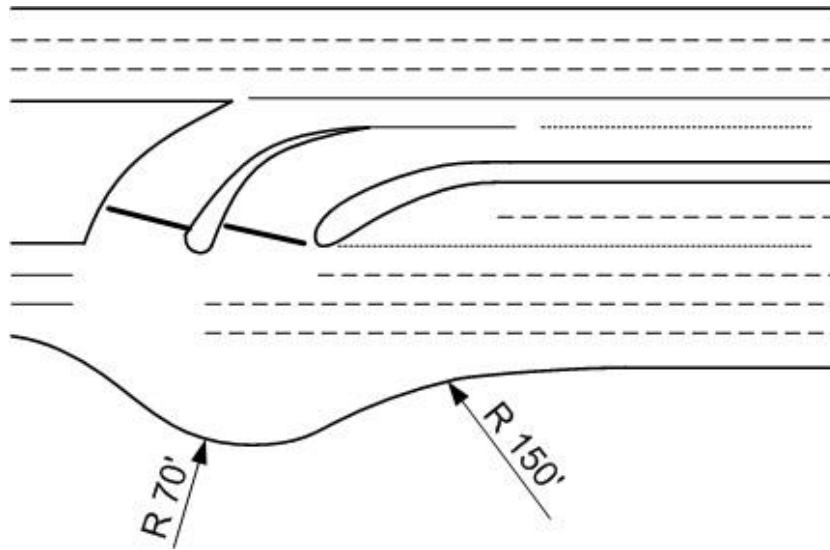
Figure 19: Loon at Crossover that Features two U-turn Lanes
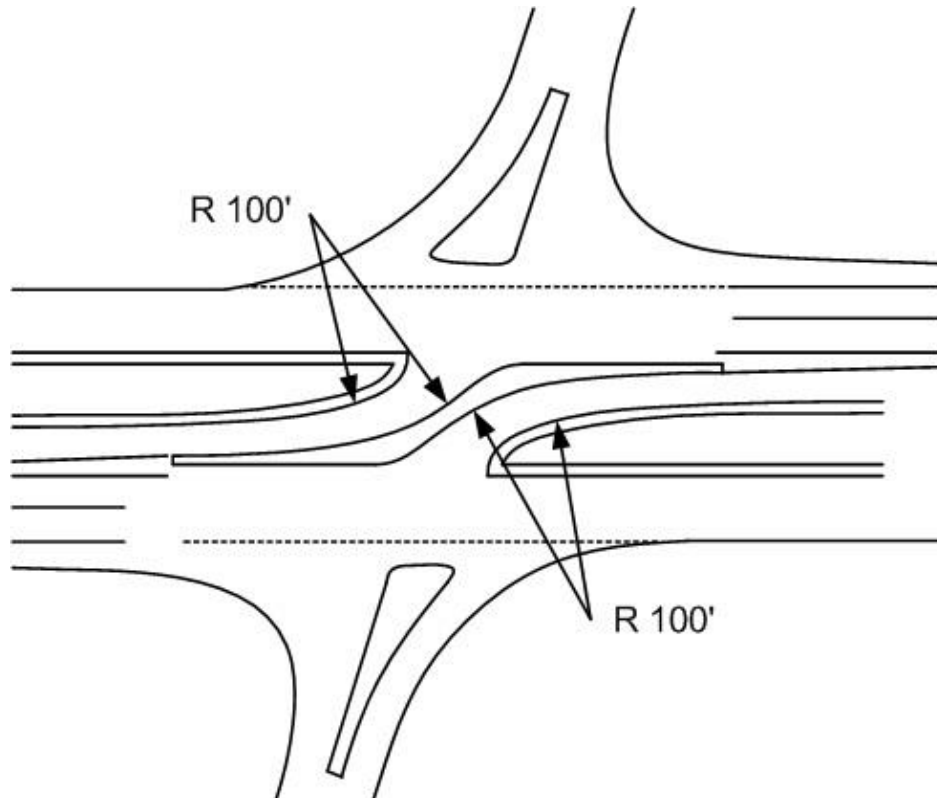
Figure 20: NCDOT Superstreet Intersection Left-turn Crossover Design Recommendation

OPERATIONAL PERFORMANCE EVALUATION METHODOLOGIES

General Considerations

Due to unavailability of empirical data on innovative designs performance, the operational performance of these designs is usually being assessed through microsimulation (*8, 9, 18, 19, 20*). Various studies confirmed the advantages of innovative designs, reporting decrease in delays and increase in throughout capacities anywhere between 10% and 90%, depending on the implemented design and location. Certain research efforts have been made to develop deterministic models for these evaluations, but with limited testing of these models. This research is using some of the newly developed models, as well as traditional HCM-type methods, to assess the performance of innovative designs on the deterministic level.

DDI Performance Evaluation

One candidate methodology for DDI performance evaluation was presented in (*21*). This methodology is an extension of the traditional HCM methodology for signalized intersections delay calculation, which takes into consideration the specific operations at a DDI. Since it is derived from the HCM methodology, it can easily be applied in the proposed algorithms for performance evaluation. The focus of this methodology in this application will be on the delay calculation of external movements at a DDI, which are defined as the through traffic on arterial streets and left turn traffic onto or off ramps. The basis of this model is the calculation of the lost green time *b* for external movements, which is caused by an internal queue that occurs during the overlap times. With the calculated lost green time b, the effective green time is obtained by the following formula:

$$g' = g - b \qquad (1)$$

where g is the actual green time of the external arterial movement, while g' is the effective green time of the arterial through movement.

$$c' = c \cdot \frac{g'}{g} \qquad (2)$$

Where:

$$c = s \cdot n \cdot \frac{g}{C} \qquad (3)$$

And:

c' = effective capacity of the lane group of the external arterial through movement,

c = capacity of the lane group of the external arterial through movement, and

C = cycle length of the signal timing.

The effective v/c ratio of the external arterial through movement X' is calculated as follows:

$$X' = \frac{g}{c'} = \frac{g \cdot q}{c \cdot g'} \qquad (4)$$

where q represents the arrival traffic flow rate of the external arterial through movement.

With the changed values of the effective green time and v/c ratio of the external arterial through movement, the control delay calculation formulas are adjusted to reflect the actual control delay at DDIs.

Uniform delay d_1:

$$d_1 = \frac{0.5C\left(1 - \frac{g'}{C}\right)^2}{1 - \left[\min(1, X') \cdot \frac{g'}{C}\right]} \qquad (5)$$

Progression factor PF:

$$PF = \frac{(1 - P) \cdot f_{PA}}{1 - \frac{g'}{C}} \qquad (6)$$

Where:

PF = uniform delay progression adjustment factor, which accounts for effects of signal progression

P = proportion of vehicles arriving at the external arterial movement on green

f_{PA} = supplemental adjustment factor for a platoon arriving during green

Incremental delay d_2:

$$d_2 = 900T\left[(X' - 1) + \sqrt{(X' - 1)^2 + \frac{8klX'}{c'T}}\right] \qquad (7)$$

Where:

d_2 = incremental delay to account for the effect of random arrivals and oversaturation queues, adjusted for the duration of the analysis period and type of signal control (s/veh)

T = analysis duration

k = incremental delay factor that is dependent on controller settings

l = upstream filtering/metering adjustment factor

Initial queue delay d_3:

$$d_3 = \frac{1,800 Q_b (1+u)t}{c'T} \qquad (8)$$

Where:

d_3 = initial queue delay, which accounts for delay to all vehicles in the analysis period caused by initial queue at the start of the analysis period (s/veh);

Q_b = initial queue at the start of period T (veh)

u = delay parameter

t = duration of unmet demand in T (h).

Control delay per vehicle for external movements is then calculated as follows:

$$d = d_1(PF) + d_2 + d_3 \left(\frac{s}{veh} \right) \qquad (9)$$

By applying the given methodology, the movement delay at a DDI can be calculated deterministically, and then the conventional LOS analysis can be performed for each movement, as well as for the DDI as a whole. For comparison purposes, the delay and LOS of a conventional diamond interchange or a SPUI can be obtained by directly applying the HCM methodology. These methodologies will further be checked for the order of magnitude by comparing them to results from microsimulation models.

CFI Performance Evaluation

The operational performance evaluation of CFIs has also been mainly done through microsimulation. Some examples of these studies can be found in (*9, 22, 23*). Deterministic models, such as Synchro or HCS still do not have the ability to accurately model and evaluate CFIs. This research is looking into some modifications of the conventional HCM methodology that can be applied to CFIs. The conventional Quick Estimation HCM methodology (QEM) has already been successfully applied in an MS Excel-based tool for regular signalized intersections (*24*). A modification of the existing Excel-based tool is currently underway to account for specifics of a CFI operation. In order to upgrade the existing tool for CFI evaluation, the following assumptions have been made:

- All turning lanes are 12 ft wide, with extra 20 ft buffer
- The crossovers are approximately 350 ft from the main intersection

- The through movements at the main intersection are the critical movements for cycle length calculation
- If pedestrian crossings exist, pedestrian clearance times are used for minimum cycle calculation
- There is no stopping/delay for left turns at the main intersection
- The maximum green times for crossover left turns depend on the lane configuration and volumes on one side, and the available buffet time during phase changes at the main intersection on the other

The cycle length and green time estimation is based on the HCM QEM methodology, and applied to the main intersection as described in (*24*). The left turn green times at the crossovers depend on the available buffer time during phase changes, calculated according to the methodologies described in (*22, 23*). The algorithm also check the required green times for these turns based on the volumes and lane configuration, and selects the green time that is higher in order to minimize delays and number of stops for left turns. Once the signal timing parameters have been calculated, the algorithm determines delays and LOS for each movement and the intersection as a whole. The application for conventional intersections was already developed and verified, so the algorithm has the ability to directly compare the conventional design with a CFI for given traffic inputs.

The base prototype of the application for comparing CFIs with conventional intersections has already been developed based on the methodologies and assumptions previously described. It has been tested on the Bangerter and 4100 S full CFI, and compared to a VISSIM microsimulation CFI model. The comparison results between VISSIM and application for the same inputs is given in Tables 2 – 4.

Table 2: Bangerter @ 4100 S Full CFI: Main Intersection Performance Comparison

	Main Intersection			
	VISSIM		**Application**	
	Vehicles	Delay (s)	Vehicles	Delay (s)
SBT	2623	20.7	2604	24.8
WBT	1243	41.0	1302	47.8
NBT	869	18.2	890	10.2
EBT	832	40.5	1024	37.0
Avg.		**25.5**		**28.4**

Table 3: Bangerter @ 4100 S Full CFI: Crossover Intersections Performance Comparison

	Crossovers			
	VISSIM		**Application**	
	Vehicles	Delay (s)	Vehicles	Delay (s)
SBL	109	54.9	110	84.4
NBT	1044	1.9	1066	2.1
NBL	332	56.0	322	65.1
SBT	3102	3.3	3078	9.2
WBL	267	65.0	272	59.7
EBT	1233	27.3	1414	7.1
EBL	100	55.1	108	55.8
WBT	1988	2.6	2042	4.6
Avg.		**12.0**		**12.2**

Table 4: Bangerter @ 4100 S Full CFI: Entire Intersection Performance Comparison

VISSIM		**Application**	
Delay (s)	**LOS**	**Delay (s)**	**LOS**
18.4	**B**	**18.7**	**B**

SAFETY EVALUATION METHODOLOGIES

General Considerations for DDIs and CFIs

Deterministic methodologies for safety evaluations of innovative designs can be based on the safety surrogates in the form of conflict points and differences in speeds at certain location compared to conventional designs, as well as existing data on safety performance of these designs. Using conflicts and speed reductions as safety surrogate measures is common in the recently emerging road safety evaluation research. Studies based on surrogate measures particularly target prediction of safety outcomes for designs that have not been implemented long enough to collect the empirical crash data for a valid before-after safety analysis. The potential benefit of these emerging studies is exploring the ability to integrate safety evaluation with microsimulation models. Because of the specific treatments for left turns and through movements, the number of conflict points is reduced with DDI and CFI designs, as shown in Figures 21 and 22 adapted from (*9*):
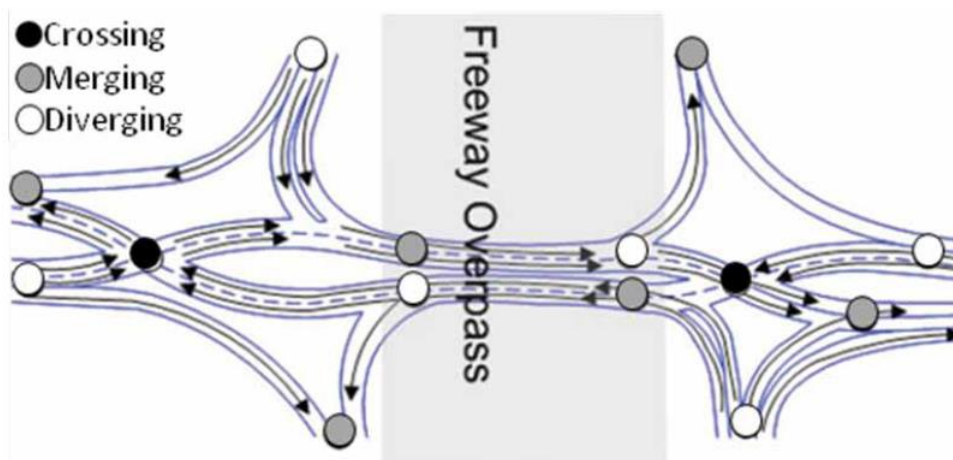
Figure 21: Conflict Points at DDI (Adapted from (*9*))
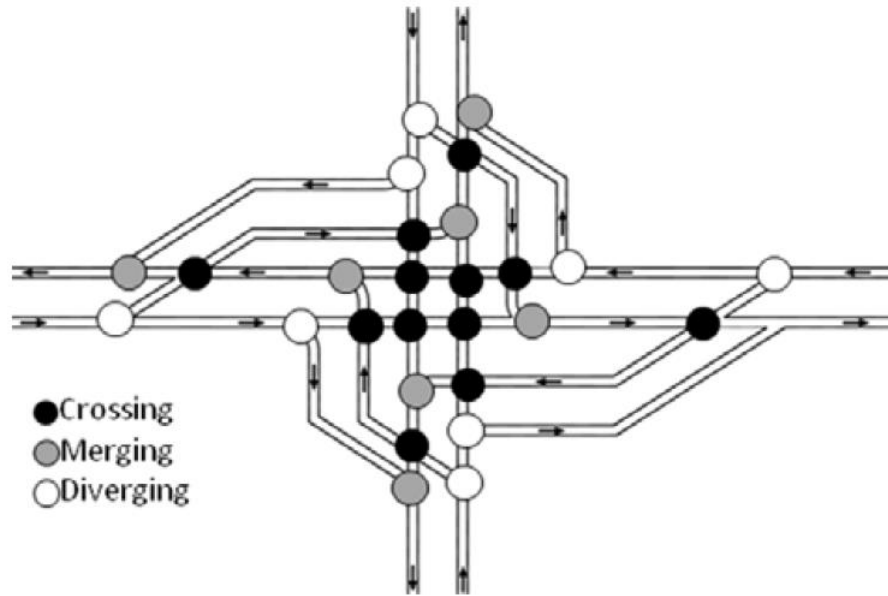
Figure 22: Conflict Points at CFI (Adapted from (*9*))

The number of conflict points within a DDI is 14, compared to 26 with a regular diamond interchange. The CFI design reduces the number of total conflict points from 32 (regular 4-legged intersection) to 30 (CFI). The number and type of conflict points can be used to directly compare different designs.

Speed is also different within the DDIs and CFI compared to conventional designs, as noted in the Geometric Design Considerations chapter. Speed can be used as another surrogate to estimate the safety of these designs. It can be directly used to assess impacts of vehicle speeds on safety in a case of vehicle-pedestrian collision. Two parameters can be used in this case: the Abbreviated Injury Scale (AIS), and the probability of pedestrian fatality.

Some researchers evaluated the effects of vehicle impact speed on the level of pedestrian injury (*26*). The output is given in the AIS as a function of the impact speed. The relationship is given in Figure 23. The degrees of injury on the AIS scale are as follows:

0 – Not injured

1 – Minor injury

2 – Moderate injury

3 – Serious injury

4 – Severe injury

5 – Critical injury

6 – Maximum injury

9 – Not specified

The literature provided basic inputs for creating a mathematical model that relates vehicle impact speeds with pedestrian injury in the case of a pedestrian/vehicle crash, given in the AIS form. This model is given in Figure 24.

The function that relates vehicle impact speed V(mph) with the AIS scale is a polynomial quadratic equation that has the following form:

$$AIS\ degree = 0.0041V^2 + 0.0079V + 0.0843 \qquad (10)$$
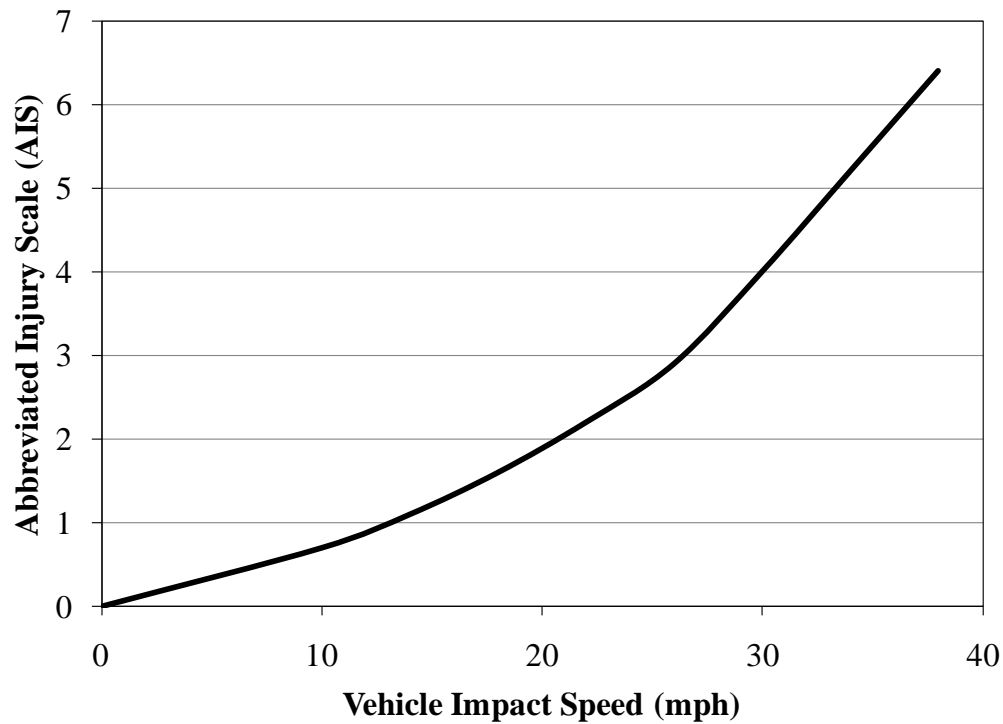
Figure 23: Abbreviated Injury Scale (AIS) as a Function of Vehicle Impact Speed
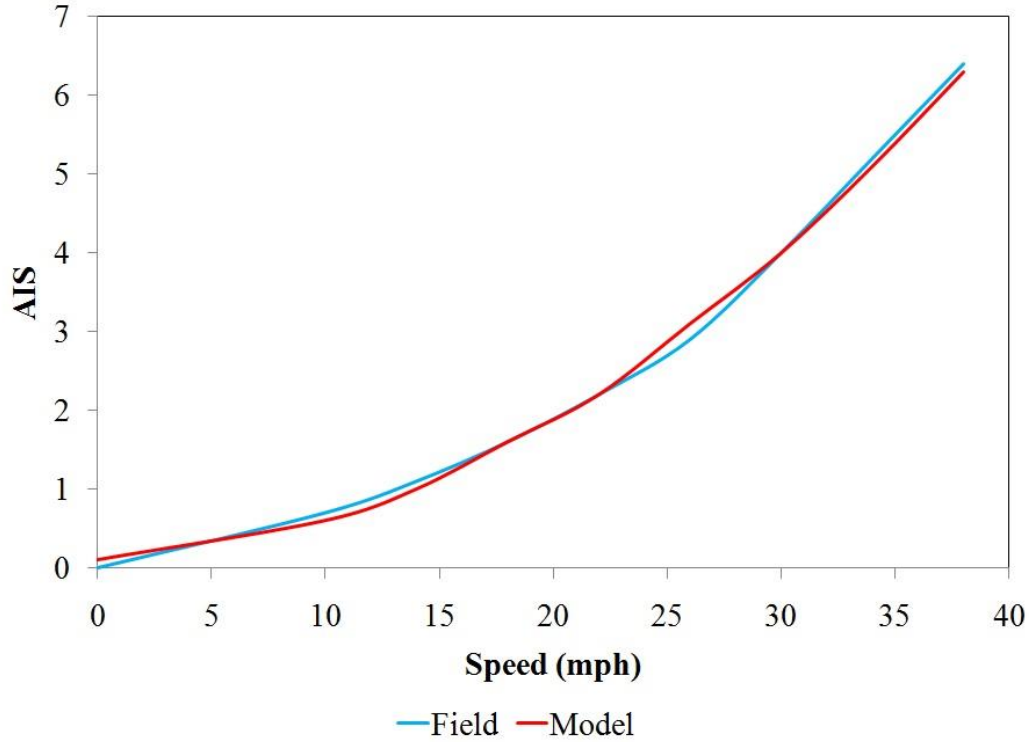
Figure 24: AIS Model

A high R² value of 0.9979 shows a good correlation between the observed and model results. This model can be applied to different sections of alternative intersection designs to assess the severity of pedestrian injury in the case of a pedestrian/vehicle crash.

Previous research studies analyzed the probability of pedestrian fatality in the case of pedestrian/vehicle crashes and developed a function that looks like follows (*27*):

$$P(v) = \frac{1}{1 + \exp(6.9 - 0.1448 \cdot v)} \qquad (11)$$

v – impact speed given in mph

Using the results from the reviewed studies it can be seen these models can be implemented to estimate safety effects of different intersection designs. The given literature provides basic guidelines for this process.

Interchange Safety Performance Functions

The Highway Safety Manual (HSM) (*34*) provides comprehensive guidelines for safety evaluations of different road and street facilities. The NCHRP report 17-45 (*35*) develops Safety Performance Functions (SPFs) for freeways and interchanges, so the methodology described in this report is used to analyze safety performance of diamond interchanges and single-point urban interchanges (SPUIs), as well as the starting point for analyzing safety performance of DDIs.

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The safety performance analysis of crossroad ramp terminals depends on the type of terminal. For conventional diamond interchanges and SPUIs, the four-leg ramp terminal with diagonal ramps is selected, which is coded as D4 in (*35*). The SPF that predicts the number of crashes per year for this type of configuration has the following form:

$$N_{spf,D4} = e^{b_{0,D4} + b_{xrd,D4} \ln\left(\frac{AADT_{xrd}}{1000}\right) + b_{rmp,D4} \ln\left(\frac{AADT_{ex}}{1000} + \frac{AADT_{en}}{1000}\right)} \qquad (12)$$

Where:

$N_{spf,D4}$ – predicted number of crashes (crashes/yr) for D4 interchange configuration under base conditions, applied to each ramp separately (if more than one)

$AADT_{xrd}$ – Average Annual Daily Traffic (AADT) of the crossroad (veh/day)

$AADT_{ex}$ – AADT of the exit ramp (veh/day)

$AADT_{en}$ – AADT of the entering ramp (veh/day)

$b_{0,D4}$, $b_{xrd,D4}$, $b_{rmp,D4}$ – calibrated coefficients for the D4 ramp terminal configuration, crossroad AADT and ramp AADT, respectively

When applying this SPF for local conditions, one needs to apply the corresponding Crash Modifications Factors (CMFs), as well as the calibration factor for local conditions C_{UT} (as a calibration factor for Utah). So the predicted average crash frequency for local (UT) conditions for D4 ramp configurations can be expressed as:

$$N_{rt,D4} = C_{UT} \cdot CMF_{comb} \cdot N_{spf,D4} \qquad (13)$$

Where:

$N_{rt,D4}$ – predicted average crash frequency for D4 configuration (crashes/yr)

C_{UT} – calibration factor for local (UT) conditions

CMF_{comb} – combined Crash Modification Factor, which is a product of CMFs for channelized turns, location, access points and segment length

Table 5 provides the values for calibrated coefficients, CMF_{comb} for typical local layouts of diamond interchanges and SPUIs, as well as the value for the overdispersion parameter k for total and property damage only (PDO) crashes.

Table 5: SPF Coefficients for Typical UT D4 Interchange Configurations

Crash type	$b_{0,D4}$	$b_{xrd,D4}$	$b_{rmp,D4}$	CMF_{comb}	k
Total	-3.044	1.255	0.114	0.87	0.087
PDO	-3.058	0.879	0.545	0.87	0.087

SPF Calibration for Local Conditions for D4 Interchange Configurations

The calibration factor used in the equation for predicting the average crash frequency has to be determined for local conditions. For that purpose, an analysis was performed using crash data for six interchanges (three diamond and three SPUIs), for a total duration of six years (2008 – 2013).

The crash data were obtained from UDOT's Traffic and Safety Division, the AADT data through the UDOT Open Data portal, while the interchange geometry was recorded through Google Earth and site visits. The crash data also contains exact location (lat/long coordinates) for each crash, and this piece of information was used to filter out all crashes that happened in the vicinity of the interchange ramps, for each of the six years. Since the provided SPF only captures interchange-related crashes, the ramp influence area was defined as the minimum of 250 ft (which is generally the distance from a ramp/intersection where the turn lanes begin), or one half of the distance between the two ramps in a diamond interchange (so that some crashes that occurred between the ramps would not be double-counted). Using the exact coordinates of each ramp, their influence areas and the crash coordinates, a search application was developed to count the number of crashes for each interchange. The data on used interchanges, their respective AADTs and the number of crashes for each of the six years is provided in Table 6. Since separate AADT data for exit and entrance ramps is not available, it is estimated that these AADTs are approximately 15% of the crossroad AADT. The results shown here are for total crashes only.

Table 6: Observed Interchange Crash and AADT Data

Diamond Interchanges	SPUIs
I-80 700 E	I-15 4500 S
I-80 State Street	I-15 3300 S
SR-201 5600 W	I-15 600 N

Diamond I-80 700 E	2008		2009		2010		2011		2012		2013	
	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp
$AADT_{xrd}$	40,535	34,040	39,645	33,295	38,295	32,160	47,270	32,710	45,235	31,300	45,100	31,250
$AADT_{ex}$	6,080	5,106	5,947	4,994	5,744	4,824	7,091	4,907	6,785	4,695	6,765	4,688
$AADT_{en}$	6,080	5,106	5,947	4,994	5,744	4,824	7,091	4,907	6,785	4,695	6,765	4,688
Crashes/yr	22		11		16		17		17		17	

Diamond I-80 State	2008		2009		2010		2011		2012		2013	
	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp
$AADT_{xrd}$	33,040	34,520	32,845	34,310	32,710	34,175	32,615	34,070	31,960	33,390	33,665	32,555
$AADT_{ex}$	4,956	5,178	4,927	5,147	4,907	5,126	4,892	5,111	4,794	5,009	5,050	4,883
$AADT_{en}$	4,956	5,178	4,927	5,147	4,907	5,126	4,892	5,111	4,794	5,009	5,050	4,883
Crashes/yr	26		18		28		42		39		49	

Diamond SR-201 5600 W	2008		2009		2010		2011		2012		2013	
	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp
$AADT_{xrd}$	19,580	38,060	19,715	38,235	19,735	38,365	19,165	40,400	17,960	40,280	18,425	41,325
$AADT_{ex}$	2,937	5,709	2,957	5,735	2,960	5,755	2,875	6,060	2,694	6,042	2,764	6,199
$AADT_{en}$	2,937	5,709	2,957	5,735	2,960	5,755	2,875	6,060	2,694	6,042	2,764	6,199
Crashes/yr	14		3		8		14		17		29	

Table 6 Continued

SPUI I-15 4500 S	2008		2009		2010		2011		2012		2013	
	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp
$AADT_{xrd}$	28,675	39,390	28,875	39,665	28,905	39,705	28,065	38,555	35,150	38,440	36,065	39,435
$AADT_{ex}$	4,301	5,909	4,331	5,950	4,336	5,956	4,210	5,783	5,273	5,766	5,410	5,915
$AADT_{en}$	4,301	5,909	4,331	5,950	4,336	5,956	4,210	5,783	5,273	5,766	5,410	5,915
Crashes/yr	37		17		33		33		27		41	

SPUI I-15 3300 S	2008		2009		2010		2011		2012		2013	
	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp
$AADT_{xrd}$	23,360	44,370	23,125	44,680	24,585	44,725	24,805	43,430	24,580	43,300	30,280	44,425
$AADT_{ex}$	3,504	6,656	3,469	6,702	3,688	6,709	3,721	6,515	3,687	6,495	4,542	6,664
$AADT_{en}$	3,504	6,656	3,469	6,702	3,688	6,709	3,721	6,515	3,687	6,495	4,542	6,664
Crashes/yr	26		21		27		37		30		38	

SPUI I-15 600 N	2008		2009		2010		2011		2012		2013	
	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp	West ramp	East ramp
$AADT_{xrd}$	13,055	13,055	12,980	12,980	12,925	12,925	12,885	12,885	12,630	12,630	12,315	12,315
$AADT_{ex}$	1,958	1,958	1,947	1,947	1,939	1,939	1,933	1,933	1,895	1,895	1,847	1,847
$AADT_{en}$	1,958	1,958	1,947	1,947	1,939	1,939	1,933	1,933	1,895	1,895	1,847	1,847
Crashes/yr	23		17		15		16		21		20	

The total predicted number of crashes for these interchanges was calculated using equation (13), coefficients from Table 5, and AADT values for crossroads and ramps from Table 6. The results for the predicted number of crashes are given in Table 7.

Table 7: Predicted Number of Crashes for D4 Configuration Interchanges

Interchange	Total Predicted Number of Crashes (crash/year)					
	2008	**2009**	**2010**	**2011**	**2012**	**2013**
I-80 700 E	10.26	9.96	9.49	11.37	10.70	10.67
I-80 State	8.95	8.87	8.83	8.79	8.55	8.71
SR-201 5600 W	7.39	7.44	7.47	7.77	7.58	7.85
I-15 4500 S	9.10	9.18	9.20	8.83	10.06	10.42
I-15 3300 S	9.20	9.22	9.47	9.24	9.18	10.36
I-15 600 N	2.43	2.42	2.40	2.39	2.33	2.25

Comparing the results from Table 7 with field data provided in Table 6, it can be seen that the given SPF underestimates the total number of crashes for local conditions. The obtained results however are used to determine the SPF calibration factor for local conditions, C_{UT}. According to the HSM (*34*), the calibration factor is the ratio between all observed crashes for all sites and all years to all predicted crashes, and in this case is defined as:

$$C_{UT} = \frac{\sum_{2008}^{2013} N_{observed}}{\sum_{2008}^{2013} N_{rt,D4}} \qquad (14)$$

Using the data from Tables 6 and 7, it is determined that the calibration factor for local conditions has a value of:

$C_{UT} = 3.00$

Multiplying the results from Table 7 by the calibration factor, one can get the calibrated results for the total predicted number of crashes for these sites. These results are given in Table 8. It should be noted that this analysis is based on a limited number of sites, and as a part of a future effort, it should be repeated for a larger population size. It should also be performed separately for diamond interchanges and SPUIs because of the subtle differences in their configurations. In the current form, for all sites and all years, the R^2 value for the total number of observed and predicted crashes is close to 0.72. These six interchanges represent the comparison group for the safety analysis described in the following section.

Table 8: Calibrated Predicted Number of Crashes for D4 Configuration Interchanges

Interchange	Total Predicted Number of Crashes (crash/yr)					
	2008	**2009**	**2010**	**2011**	**2012**	**2013**
I-80 700 E	30.82	29.90	28.52	34.14	32.15	32.04
I-80 State	26.88	26.66	26.51	26.40	25.68	26.15
SR-201 5600 W	22.19	22.35	22.43	23.35	22.76	23.57
I-15 4500 S	27.32	27.58	27.62	26.53	30.23	31.31
I-15 3300 S	27.63	27.70	28.43	27.76	27.57	31.12
I-15 600 N	7.31	7.26	7.21	7.18	6.99	6.75

Another parameter that can be calculated from the observed and predicted crashes is the yearly modification factor (a_y), and it represents a ratio between observed and predicted crashes during one year. This factor shows year-to-year fluctuations in the number of predicted crashes (for all sites combined), and is used in the next step to adjust the number of predicted crashes for the treatment sites. It is provided in Table 9 for each analyzed year.

Table 9: Yearly Modification Factor a_y for Interchange Crashes

Year	**2008**	**2009**	**2010**	**2011**	**2012**	**2013**
a_y	1.041	0.615	0.902	1.094	1.039	1.285

DDI Crash Modification Factor: Analysis through Empirical Bayes Methodology

A DDI conversion represents a significant change in interchange geometry and operations, so it can be expected to have some impact on safety also. This impact can be assessed and described through a CMF for DDI conversion. For local conditions, the CMF is calculated using the available crash data for DDI locations for several years before and after the DDI conversion. However, these data are still very scarce, especially for the after period. This can have impacts on the obtained results, so it is recommended to take the results presented here with a reserve, and repeat the analysis using this methodology as a part of a future effort when more data become available. Three DDIs in Utah are used for CMF analysis: SR-201 and Bangerter in West Valley City (built in 2011), I-15 and Pioneer Crossing in American Fork (built in 2011), and I-15 and 500 E in American Fork (built in 2012). The crash and AADT data are obtained the same way as for the diamond interchanges and SPUIs described previously, for a period between 2008 and 2013. These data are given in Table 10.

Table 10: Before and After DDI Crash and AADT Data

SR-201 Bangerter	2008		2009		2010		2011*		2012*		2013*	
	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp
$AADT_{xrd}$	33,590	39,240	33,560	39,200	31,445	36,730	28,960	33,830	29,075	33,965	30,645	35,800
$AADT_{ex}$	5,039	5,886	5,034	5,880	4,717	5,510	4,344	5,075	4,361	5,095	4,597	5,370
$AADT_{en}$	5,039	5,886	5,034	5,880	4,717	5,510	4,344	5,075	4,361	5,095	4,597	5,370
Crashes/yr	18		23		22		26		23		34	

I-15 Pioneer Crossing	2008		2009		2010		2011*		2012*		2013*	
	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp
$AADT_{xrd}$	23,740	31,027	23,910	31,249	23,810	31,118	23,740	31,027	23,265	30,406	24,500	32,020
$AADT_{ex}$	3,561	4,654	3,587	4,687	3,572	4,668	3,561	4,654	3,490	4,561	3,675	4,803
$AADT_{en}$	3,561	4,654	3,587	4,687	3,572	4,668	3,561	4,654	3,490	4,561	3,675	4,803
Crashes/yr	13		8		9		8		9		9	

I-15 500 E AF	2008		2009		2010		2011		2012*		2013*	
	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp	North ramp	South ramp
$AADT_{xrd}$	19,080	19,080	18,965	18,965	18,890	18,890	18,830	18,830	18,455	18,455	17,995	17,995
$AADT_{ex}$	2,862	2,862	2,845	2,845	2,834	2,834	2,825	2,825	2,768	2,768	2,699	2,699
$AADT_{en}$	2,862	2,862	2,845	2,845	2,834	2,834	2,825	2,825	2,768	2,768	2,699	2,699
Crashes/yr	15		8		7		5		3		0	

* DDI

The predicted number of crashes for the treatment sites (including both before and after periods) can be obtained by using Equation 13, with the inclusion of the yearly modification factor, as follows:

$$N_{rt,D4} = a_y \cdot C_{UT} \cdot CMF_{comb} \cdot N_{spf,D4} \qquad (15)$$

This predicted number of crashes for the treatment sites represents a starting point in the Empirical Bayes (EB) analysis. Note that this result does not include the CMF for DDI conversion, which is to be determined based on these data. The results for total predicted crashes for these sites are given in Table 11.

Table 11: Predicted Number of Crashes for Treatment Sites (No DDI Conversion)

Interchange	Total Predicted Number of Crashes (crash/yr)					
	2008	**2009**	**2010**	**2011**	**2012**	**2013**
SR-201 Bangerter	31.06	18.32	24.59	26.63	25.43	33.82
I-15 Pioneer Crossing	21.08	12.58	18.35	22.15	20.46	27.18
I-15 500 E AF	12.80	7.50	10.94	13.21	12.20	14.59

The EB method combines an estimation of the study site crash frequency with characteristics of similar sites using SPFs to estimate the predicted number of crashes (*36, 37*). It is shown that the EB method is better suited to estimate safety than more traditional statistical methods. The EB methodology presented here is adapted from (*38*). Major characteristics of the EB method are as follows:

a) It accounts, directly with modeling, for changes in factors that are measured and understood
b) It accounts for changes in unmeasured factors
c) It accounts for "regression-to-the-mean": the "before" crash counts alone may not be a good basis for predicting "what would have been"

The main parameters used in the presented analysis are as follows:

a) "before" period: from the start year of analysis (2008 in this case) until the year before the treatment was implemented
b) "after" period: from one year after the treatment was implemented until the end year of analysis (2013 in this case)
c) $\hat{\pi}$ - predicted number of crashes in the "after" period: what would have been if the treatment had not been implemented, all sites

$$\hat{\pi}_j = C_j \cdot M_j \qquad (16)$$

where:

j – site code

$$C_j = \frac{\sum predicted\ crashes\ in\ after\ period\ for\ site\ j}{\sum predicted\ crashes\ in\ before\ period\ \ for\ site\ j} \qquad (17)$$

$$M_j = w \cdot P_j + (1 - w) \cdot K_j \qquad (18)$$

with the following:

w – weight, computed as

$$w = \frac{1}{1 + k \cdot P_j} \qquad (19)$$

k – overdispersion parameter, which was calculated when SPFs were developed, provided in HSM, and given in Table 5 for D4 interchange configuration SPFs

$$K_j = \sum observed\ crashes\ in\ before\ period\ for\ site\ j \qquad (20)$$
$$P_j = \sum predicted\ crashes\ in\ before\ period\ \ for\ site\ j \qquad (21)$$

$$\hat{\pi} = \sum_j \hat{\pi}_j \qquad (22)$$

d) $VAR\{\hat{\pi}_j\} = C_j^2 \cdot VAR\{M_j\} \qquad (23)$

where:

$$VAR\{M_j\} = M_j \cdot (1 - w) \qquad (24)$$

$$VAR\{\hat{\pi}\} = \sum_j VAR\{\hat{\pi}_j\} \qquad (25)$$

e) $\hat{\lambda}$ - observed number of crashes in the "after" period: what actually was after the treatment had been implemented, all sites

$$\hat{\lambda}_j = \sum observed\ crashes\ in\ after\ period\ for\ site\ j \qquad (26)$$

$$\hat{\lambda} = \sum_j \hat{\lambda}_j \qquad (27)$$

f) $VAR\{\hat{\lambda}_j\} = \hat{\lambda}_j \qquad (28)$

$$VAR\{\hat{\lambda}\} = \sum_{j} VAR\{\hat{\lambda}_j\} \qquad (29)$$

g) $\hat{\theta}$ - the ratio of the observed number of crashes in the after period with the treatment, to the predicted number of crashes in the after period if the treatment had not been implemented, applied to all treated sites, computed as:

$$\hat{\theta} = \frac{\hat{\lambda}}{\hat{\pi} \cdot \left(1 + \frac{VAR\{\hat{\pi}\}}{\hat{\pi}^2}\right)} \qquad (30)$$

$$VAR\{\hat{\theta}\} = \hat{\theta}^2 \cdot \frac{\frac{1}{\hat{\lambda}} + \frac{VAR\{\hat{\pi}\}}{\hat{\pi}^2}}{\left(1 + \frac{VAR\{\hat{\pi}\}}{\hat{\pi}^2}\right)^2} \qquad (31)$$

$\hat{\theta}$ actually represents the CMF for the applied treatment.

h) $\sigma\{\hat{\theta}\} = \sqrt{VAR\{\hat{\theta}\}}$ – standard deviation of $\hat{\theta}$ $\qquad (32)$

i) $\delta = \hat{\pi} - \hat{\lambda}$ - the reduction in number of crashes in the after period (what would have been minus what actually was) $\qquad (33)$

j) $VAR\{\delta\} = VAR\{\hat{\pi}\} + VAR\{\hat{\lambda}\}$ $\qquad (34)$

The described EB methodology is applied to the DDI treated sites, using inputs from Tables 10 and 11, and the following results were obtained (Table 12):

Table 12: EB Analysis Results for DDI Treated Sites

Parameter	Value
$\hat{\pi}$	98.63
$VAR\{\hat{\pi}\}$	64.56
$\hat{\lambda}$	75
$VAR\{\hat{\lambda}\}$	75
$\hat{\theta}$	0.755
$VAR\{\hat{\theta}\}$	0.01
$\sigma\{\hat{\theta}\}$	0.106
δ	23.63
$VAR\{\delta\}$	139.56

It can be seen that the crash modification factor for DDI conversion for local conditions is:

$CMF_{DDI} = 0.755$

The Crash Reduction Factor (CRF) for DDI conversion is in this case:

$CRF_{DDI} = 24.5\% \pm 10.6\%$

Again, it should be noted that this analysis is based on a limited number of sites and few years (only one or two) of the "after" period. This methodology can be used as the general guidance to repeat the analysis as a part of a future effort when more DDI crash data become available. The obtained CMF was used in the DDI safety module, as described later in the report.

Intersection Safety Performance Functions

Chapter 12 of the HSM (*34*) describes the predictive methodology for analyzing crashes at signalized intersections. This methodology contains SPFs and CMFs for different geometrical and operational characteristics of signalized intersections. The effect of traffic volumes on major and minor intersection approaches on predicted crash frequency is incorporated through SPFs, while the effects of geometric and traffic control features are incorporated through CMFs. The SPFs address four types of crashes: multiple-vehicle, single-vehicle, vehicle-pedestrian and vehicle-bicycle crashes.

The SPF for total multiple-vehicle crashes has the following form:

$$N_{bimv} = \exp\left(a + b \cdot \ln\left(AADT_{maj}\right) + c \cdot \ln\left(AADT_{min}\right)\right) \qquad (35)$$

Where:

N_{bimv} – predicted crash frequency for total multiple-vehicle crashes (crashes/yr)

AADT$_{maj}$ – AADT on the major intersection approach (veh/day)

AADT$_{min}$ – AADT on the minor intersection approach (veh/day)

a, b, c – regression coefficients

Equation (35) is first applied to determine the predicted number of total multiple-vehicle crashes, and is then divided into components by severity level, N$_{bimv(FI)}$ for fatal and injury crashes, and N$_{bimv(PDO)}$ for property damage only crashes. Equation (35) is used to determine the preliminary values for FI and PDO crashes using calibrated regression coefficients, designated as N'$_{bimv(FI)}$ and N'$_{bimv(PDO)}$, and then the adjustments are made using the following equations:

$$N_{bimv(FI)} = N_{bimv(total)} \cdot \left(\frac{N'_{bimv(FI)}}{N'_{bimv(FI)} + N'_{bimv(PDO)}} \right) \qquad (36)$$

$$N_{bimv(PDO)} = N_{bimv(total)} - N_{bimv(FI)} \qquad (37)$$

The SPF coefficients for 3-leg (3SG) and 4-leg (4SG) signalized intersections, used in Equations (35) – (37) for multiple-vehicle collisions, are given in Table 13.

Table 13: SPF Coefficients for Multiple-Vehicle Crashes at Signalized Intersections

Intersection Type	Intercept (a)	AADT$_{maj}$ (b)	AADT$_{min}$ (c)	Overdispersion Parameter (k)
Total Crashes				
3SG	-12.13	1.11	0.26	0.33
4SG	-10.99	1.07	0.23	0.39
Fatal and Injury Crashes				
3SG	-11.58	1.02	0.17	0.30
4SG	-13.14	1.18	0.22	0.33
Property-Damage-Only Crashes				
3SG	-13.24	1.14	0.30	0.36
4SG	-11.02	1.02	0.24	0.44

The SPFs for single-vehicle crashes have the same forms, but with different coefficients. The SPFs are given in Equations (38) – (40), while the regression coefficients are provided in Table 14.

$$N_{bisv} = \exp\left(a + b \cdot \ln(AADT_{maj}) + c \cdot \ln(AADT_{min})\right) \qquad (38)$$

$$N_{bisv(FI)} = N_{bisv(total)} \cdot \left(\frac{N'_{bisv(FI)}}{N'_{bisv(FI)} + N'_{bisv(PDO)}} \right) \qquad (39)$$

$$N_{bisv(PDO)} = N_{bisv(total)} - N_{bisv(FI)} \qquad (40)$$

Table 14: SPF Coefficients for Single-Vehicle Crashes at Signalized Intersections

Intersection Type	Intercept (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)	Overdispersion Parameter (k)
Total Crashes				
3SG	-9.02	0.42	0.40	0.36
4SG	-10.21	0.68	0.27	0.36
Fatal and Injury Crashes				
3SG	-9.75	0.27	0.51	0.24
4SG	-9.25	0.43	0.29	0.09
Property-Damage-Only Crashes				
3SG	-9.08	0.45	0.33	0.53
4SG	-11.34	0.78	0.25	0.44

The HSM also provides SPFs for vehicle-pedestrian and vehicle-bicycle collisions. However, pedestrian and bicycle volumes at intersections are in most cases not available. For that reason it is usually assumed that vehicle-pedestrian and vehicle-bicycle crashes combined are approximately 4% of the multiple-vehicle crashes:

$$N_{bi(ped/bike)} = 0.04 \cdot N_{bimv} \qquad (41)$$

The available CMFs for signalized intersections include CMF for exclusive left turn lanes (CMF_{LT}), CMF for exclusive right turn lanes (CMF_{RT}), and CMF for left turn phasing ($CMF_{LTphase}$). CMF_{LT} and CMF_{RT} are determined based on the intersection type (3SG or 4SG) and the number of approaches with LT or RT lanes. $CMF_{LTphase}$ is determined separately for protected, permitted or protected/permitted left turn signal phasing for each intersection approach separately, and then the obtained values are multiplied to determine $CMF_{LTphase}$ for the entire intersection. For local conditions, it is assumed that intersection lighting exists ($CMF_{light} = 0.91$) and that there are no red-light cameras ($CMF_{rl\,cam} = 1.0$). The combined CMF_{comb} therefore can be computed as:

$$CMF_{comb} = CMF_{LT} \cdot CMF_{RT} \cdot CMF_{LTphase} \qquad (42)$$

Table 15 provides the HSM calibrated values for the three CMFs.

Table 15: CMFs for Geometry and Signal Phasing at Signalized Intersections

CMF_{LT} for number of approaches with LT lanes				
# approaches	1	2	3	4
3SG	0.93	0.86	0.80	-
4SG	0.90	0.81	0.73	0.66
CMF_{RT} for number of approaches with RT lanes				
# approaches	1	2	3	4
3SG	0.96	0.92	-	-
4SG	0.96	0.92	0.88	0.85
$CMF_{LTphase}$ for type of LT treatment				
Protected LT		Permitted LT		Protected/permitted LT
0.94		1.00		0.99

The number of predicted crashes for an entire intersection can then be determined with Equation (43). This equation can be used for total FI and PDO crashes.

$$N_{pred} = CMF_{comb} \cdot \left(N_{bimv} + N_{bisv} + N_{bi(ped/bike)}\right) \qquad (43)$$

SPF Calibration for Local Conditions for Signalized Intersections

Equation (43) represents a starting point in predicting signalized intersection crash frequencies. It still needs to be calibrated for use in local conditions, where it would have the following form:

$$N_{pred} = C_{UT} \cdot CMF_{comb} \cdot \left(N_{bimv} + N_{bisv} + N_{bi(ped/bike)}\right) \qquad (44)$$

To determine the local calibration factor C_{UT}, an analysis was performed using crash data for five 4-leg signalized intersections, for a total duration of six years (2008 – 2013). These intersections represent the comparison group for the EB analysis described in the next section.

As in the previous case for interchange analysis, the crash data were obtained from UDOT's Traffic and Safety Division, the AADT data through the UDOT Open Data portal, and the intersection geometry and signal phasing was recorded through Google Earth and site visits. The crash data contains the exact location (lat/long coordinates) for each crash, and this piece of information was used to filter out all crashes that happened in the vicinity of the intersection, for each of the six years. Since the provided intersection SPFs only capture intersection-related crashes, the

intersection influence area was defined as the radius of 250 ft from the center of the intersection (which is generally the distance where the turn lanes begin). Using the exact coordinates of each intersection, their influence areas and the crash coordinates, a search application was developed to count the number of crashes for each intersection. The data on analyzed intersections, their respective AADTs and the number of crashes for each of the six years is provided in Table 16. The results shown here are for total crashes only.

Table 16: Observed Signalized Intersection Crashes and AADT Data

Redwood Rd. @ 3500 S	2008	2009	2010	2011	2012	2013
$AADT_{maj}$	40,865	40,620	40,455	40,335	39,530	30,260
$AADT_{min}$	27,460	27,195	28,915	29,190	28,920	29,180
crashes/yr	41	48	30	45	46	59

5600 W @ 3500 S	2008	2009	2010	2011	2012	2013
$AADT_{maj}$	38,060	38,325	38,365	40,400	40,280	41,325
$AADT_{min}$	26,885	27,070	27,100	26,315	26,235	26,915
crashes/yr	57	43	44	38	77	57

State St. @ 4500 S	2008	2009	2010	2011	2012	2013
$AADT_{maj}$	32,555	32,785	32,815	38,705	38,590	39,590
$AADT_{min}$	30,190	30,005	29,885	32,695	32,045	31,240
crashes/yr	47	38	35	42	46	94

State St. @ 3300 S	2008	2009	2010	2011	2012	2013
$AADT_{maj}$	37,445	37,705	37,745	36,650	36,540	37,490
$AADT_{min}$	30,075	29,895	29,775	27,425	26,875	26,500
crashes/yr	44	39	40	53	43	34

700 E @ 3300 S	2008	2009	2010	2011	2012	2013
$AADT_{maj}$	38,820	37,970	36,675	37,300	35,695	35,590
$AADT_{min}$	27,035	27,225	27,250	24,045	23,970	24,595
crashes/yr	30	36	25	60	35	43

The predicted number of total crashes for these sites was calculated using Equations (35), (38), (41), (42) and (43). Each of the CMFs for each site was determined separately, and then used to compute CMF_{comb}. The results for predicted number of total crashes are provided in Table 17.

Table 17: Predicted Number of Crashes for Signalized Intersections in Comparison Group

Intersection	CMF$_{comb}$	Total Predicted Number of Crashes (crash/yr)					
		2008	2009	2010	2011	2012	2013
Redwood 3500 S	0.47	7.81	7.74	7.82	7.81	7.63	5.78
5600 W 3500 S	0.48	7.36	7.43	7.44	7.80	7.77	8.03
State 4500 S	0.48	6.42	6.46	6.46	7.84	7.78	7.95
State 3300 S	0.44	6.81	6.85	6.85	6.52	6.47	6.62
700 E 3300 S	0.52	8.16	7.98	7.70	7.61	7.26	7.28

Comparing the results from Table 17 with field data provided in Table 16, it can be seen that the given SPF underestimates the total number of crashes for local conditions. The obtained results however are used to determine the SPF calibration factor for local conditions, C$_{UT}$, using the following equation:

$$C_{UT} = \frac{\sum_{2008}^{2013} N_{total\ observed}}{\sum_{2008}^{2013} N_{total\ predicted}} \qquad (45)$$

Using the data from Tables 16 and 17, it is determined that the calibration factor for local conditions has a value of:

C$_{UT}$ = 4.52

Multiplying the results from Table 17 by the calibration factor, one can get the calibrated results for the total predicted number of crashes for these sites. These results are given in Table 18. Similar as for interchange analysis, these results are based on a limited number of sites and crash data. This methodology should serve as guidance for future analysis, with more sites and crash data.

Table 18: Calibrated Predicted Number of Crashes for Signalized Intersections in Comparison Group

Intersection	Total Calibrated Predicted Number of Crashes (crash/yr)					
	2008	2009	2010	2011	2012	2013
Redwood 3500 S	35.26	34.96	35.31	35.28	34.46	26.08
5600 W 3500 S	33.26	33.55	33.60	35.23	35.10	36.27
State 4500 S	28.99	29.17	29.17	35.42	35.14	35.89
State 3300 S	30.76	30.94	30.94	29.43	29.20	29.90
700 E 3300 S	36.83	36.04	34.76	34.37	32.79	32.89

The yearly modification factor a_y is also calculated using results from Tables 16 and 18, for each year separately for all sites. This factor is given in Table 19.

Table 19: Yearly Modification Factor a_y for Signalized Intersection Crashes

Year	2008	2009	2010	2011	2012	2013
a_y	1.030	0.911	0.763	1.078	1.092	1.124

CFI Crash Modification Factor: Analysis through Empirical Bayes Methodology

A conversion to CFI is expected to have some impacts on intersection crashes, since it modifies intersection geometry, operations, and impacts driver expectancy. This impact can be assessed and described through a CMF for CFI conversion. For local conditions, the CMF is calculated using the available crash data for eight locations for several years before and after the CFI conversion. However, these data are still very scarce, especially for the after period. This can have impacts on the obtained results, so it is recommended to take the results presented here with a reserve, and repeat the analysis using this methodology as a part of a future effort when more data become available. The crash and AADT data are obtained the same way as for the comparison sites, for a period between 2008 and 2013. However, since a CFI introduces crossover intersections, crashes that occurred in the vicinity of the crossovers are also added to the main intersection crashes. In this case, it is assumed that the intersection influence area is 250 ft for the main intersection, and 100 ft for each crossover. The observed crash data for total crashes for CFI sites is given in Table 20.

Table 20: Observed Total Crashes for CFI Sites (Before and After Conversion)

Bangerter @ 3100 S	2008	2009	2010	2011*	2012*	2013*
AADT$_{maj}$	47,390	47,345	44,360	48,124	47,980	49,230
AADT$_{min}$	17,440	17,335	17,265	17,207	16,870	16,445
crashes/yr	23	31	11	26	18	23

Bangerter @ 4100 S	2008	2009	2010	2011*	2012*	2013*
AADT$_{maj}$	50,710	51,065	51,115	49,633	49,485	50,770
AADT$_{min}$	30,885	30,700	30,580	30,473	29,875	29,130
crashes/yr	49	51	34	47	57	63

Bangerter @ 4700 S	2008	2009	2010*	2011*	2012*	2013*
AADT$_{maj}$	51,265	51,625	51,675	54,724	54,560	55,980
AADT$_{min}$	33,085	33,320	31,840	30,917	30,825	31,625
crashes/yr	66	62	48	47	47	63

Bangerter @ 5400 S	2008	2009	2010*	2011*	2012*	2013*
AADT$_{maj}$	55,915	56,305	56,360	54,724	54,560	55,980
AADT$_{min}$	30,715	33,600	33,465	40,086	39,300	38,320
crashes/yr	108	70	109	91	95	96

Bangerter @ 6200 S	2008	2009	2010	2011*	2012*	2013*
AADTmaj	55,840	56,230	56,285	54,653	54,490	55,905
AADTmin	30,460	30,280	30,160	30,054	29,465	28,730
crashes/yr	46	57	47	82	102	94

* CFI

Table 20 Continued

Bangerter @ 7000 S	2008	2009	2010	2011*	2012*	2013*
AADT$_{maj}$	55,840	56,230	56,285	54,653	54,490	55,905
AADT$_{min}$	18,120	18,010	19,855	19,795	19,400	18,915
crashes/yr	36	43	33	39	42	57

Redwood @ 5400 S	2008	2009	2010*	2011*	2012*	2013*
AADT$_{maj}$	63,115	62,735	62,485	62,271	61,050	59,525
AADT$_{min}$	39,960	43,700	41,635	39,798	38,865	39,195
crashes/yr	66	73	68	75	64	53

Redwood @ 6200 S	2008	2009	2010*	2011*	2012*	2013*
AADT$_{maj}$	37,360	37,620	37,660	36,760	36,650	37,605
AADT$_{min}$	22,430	22,295	22,210	22,134	21,700	21,155
crashes/yr	74	70	85	47	79	44

* CFI

The predicted number of total crashes for the treatment sites (including both before and after periods) can be obtained as follows:

$$N_{pred} = a_y \cdot C_{UT} \cdot CMF_{comb} \cdot \left(N_{bimv} + N_{bisv} + N_{bi(ped/bike)}\right) \qquad (46)$$

This predicted number of crashes for the treatment sites represents a starting point in the Empirical Bayes (EB) analysis. Note that this result does not include the CMF for CFI conversion, which is to be determined based on these data. The results for total predicted crashes for these sites are given in Table 21.

Table 21: Calibrated Predicted Number of Crashes for Signalized Intersections in Treatment Group

Intersection	Total Calibrated Predicted Number of Crashes (crash/yr)					
	2008	2009	2010	2011	2012	2013
Bangerter 3100 S	32.51	28.69	22.42	34.48	34.65	36.43
Bangerter 4100 S	39.85	35.47	29.72	40.67	40.86	42.97
Bangerter 4700 S	40.96	36.56	30.34	45.22	45.62	48.54
Bangerter 5400 S	44.11	40.14	33.63	48.03	48.26	50.75
Bangerter 6200 S	43.97	39.13	32.79	44.87	45.08	47.40
Bangerter 7000 S	38.98	34.69	29.76	40.73	40.92	43.03
Redwood 5400 S	53.27	47.81	39.44	54.94	54.18	54.42
Redwood 6200 S	26.83	23.88	20.01	27.54	27.67	29.09

The same EB methodology is applied to signalized intersections as for the previously described interchange analysis. Equations (16) – (34) are used to calculate all parameters and determine the CMF for CFI conversion. The results of the EB analysis are given in Table 22.

Table 22: EB Analysis Results for CFI Treated Sites

Parameter	Value
$\hat{\pi}$	945.08
$VAR\{\hat{\pi}\}$	1372.25
$\hat{\lambda}$	830
$VAR\{\hat{\lambda}\}$	830
$\hat{\theta}$	0.877
$VAR\{\hat{\theta}\}$	0.0021
$\sigma\{\hat{\theta}\}$	0.045
δ	115.08
$VAR\{\delta\}$	2202.25

It can be seen that the crash modification factor for CFI conversion for local conditions is:

$CMF_{CFI} = 0.877$

The Crash Reduction Factor (CRF) for CFI conversion is in this case:

$CRF_{CFI} = 12.3\% \pm 4.6\%$

Again, it should be noted that this analysis is based on limited data, so this methodology can be used as the general guidance to repeat the analysis as a part of a future effort when more CFI crash data become available. The obtained CMF was used in the CFI safety module, as described later in the report.

ACCESSIBILITY CONSIDERATIONS

Access management involves controlling vehicle movement efficiently by maximizing capacity and reducing major access conflicts with the land adjacent to the intersection (*29*). This can be especially important when implementing innovative intersections in order to address the existing needs of the area, and to encourage economic growth and development.

Access Management Considerations for DDIs and CFIs

Many of the access concerns with DDIs and CFIs have been similar based upon the large layout of these types of intersections along with the types of vehicle movements involved in these intersections. An advantage of DDIs in terms of their access management is that they provide full access control through an interchange. However, not all movements can be accessed through a DDI, it does not allow movement between an exit ramp and an entrance ramp (*30*). CFIs also have this problem with U-turn movements. A countermeasure that has been implemented to improve this issue is a U-turn crossover between the main intersection and the left-turn crossover (*9*).

Another accessibility issue for DDIs and CFIs has to do with adjacent intersections. Due to the layout of these intersections, they can require removing adjacent streets or driveways. The functional area of an intersection is defined as the area upstream and downstream of the intersection. This area can be variable depending on the intersection based upon different factors including the distance travelled during the perception-reaction time, and deceleration distance along with the amount of queuing at the intersection (*31*). This area, also known as the corner clearance, is the area in which there should not be any driveways abutting the road and is measured from the stopline of the intersection to the point of curvature of the driveway. According to the Alabama DOT Access Management Manual, any full access signalized intersection without a median and a design speed of less than 45 mph requires a corner clearance of 1,320 feet (*32*). This area can be seen on a DDI in Figure 25.
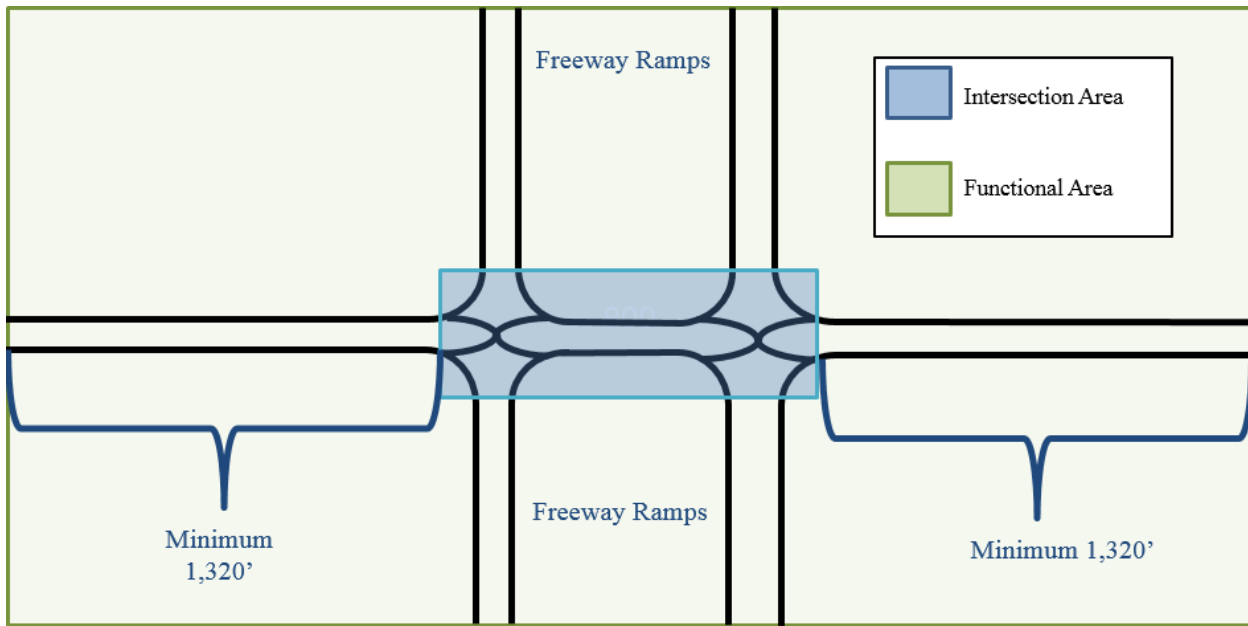
Figure 25: Functional area for a DDI

The Functional area of a CFI differs from this due to the crossover intersection spacing before the intersection. The spacing between the main intersection and the crossover intersection generally ranges from 300 to 500 feet; however, the distance in which any shared access can be placed must be another 900 feet past the crossover intersection in order to allow for vehicles to have time to complete the turning movements at the intersection (*32,33*).
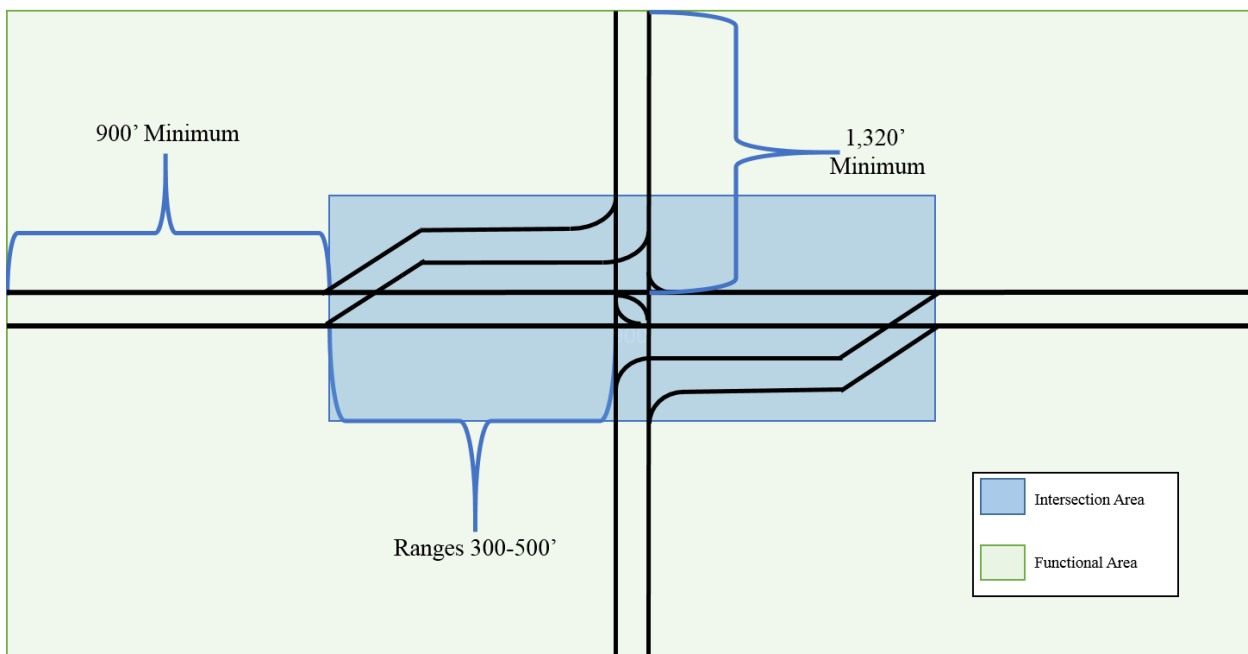

Figure 26. Functional Area for a CFI

A method that has been used for CFIs to accommodate existing driveways around the intersection is the implementation of a frontage road alongside the intersection. In Baton Rouge, LA, a frontage road was incorporated alongside the CFI in order to provide access to the existing businesses without impacting the flow of the CFI (*9*). The DDIs two-phase signal is also highly efficient, and can often cause backups at adjacent intersections due to the higher traffic flow in the DDIs that cannot be accommodated by other intersections. This can often cause congestion and backflow into the DDI (*30*).

Pedestrian access is an additional concern with DDIs. The two common methods for pedestrian access are either along the vehicle travel way or through the middle of the intersection as seen in Figures 27 and 28 below from DDIs in Missouri.

Figure 27: Pedestrian access outside of a DDI in Maryland Heights, MO (Adapted from (30))
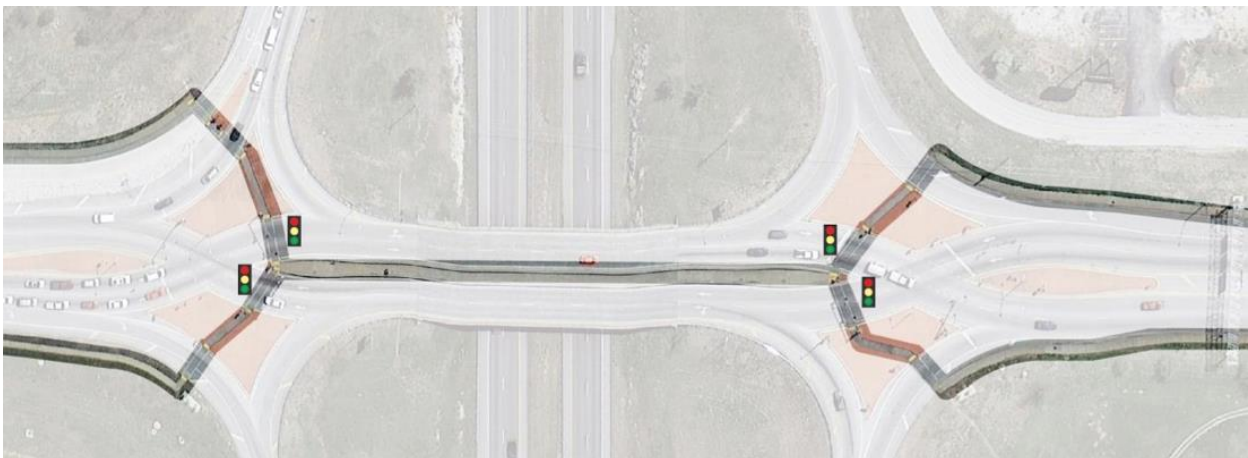

Figure 28: Pedestrian access inside a DDI in Springfield, MO (Adapted from (30))

Both types create issues for many pedestrians. Each of these crosswalks needs to be signalized due to high traffic volumes and vehicle speeds. Pedestrians need to cross the arterial streets as well as the highway exit ramps, which creates safety concerns as well as additional timing in order to accommodate the pedestrian movement.

UDOT's CFI guidelines (*39*) recognize four general categories of CFI access accommodations:

1) Access accommodation at the crossover
2) Access accommodation prior to the crossover
3) Access accommodation at the displaced left turn
4) Access accommodation at the bypass right turn

These access points require special attention when being designed. Most of the time they are restricted to right-in/right-out, or left-in/left-out movements only. Traffic movements at these access points can potentially be detrimental to the CFI operations, since they can slow down the traffic in the main lanes, which causes some safety concerns. For that reason it is not recommended to have direct access points within the CFI functional area. State DOTs usually do not recommend having median breaks within 600 – 700 ft from the main intersection, which reduces accessibility in the vicinity of CFI intersections.

One way of measuring the quality of access along urban streets in the vicinity of intersections or interchange ramps is to determine the available driveway and cross street density, using the State highway access management spacing standards. UDOT provides these standards for local highways in (*40*). Based on the highway category, the standard defines minimum signal spacing, minimum street spacing (for cross streets), and minimum driveway spacing. These values are provided in Table 23, filtered only for the highway categories applicable to DDI and CFI locations. These values are used in the accessibility module for computing signal, street and driveway, combined with the minimum required clear zone in the vicinity of a DDI or a CFI.

Table 23: State Highway Access Management Spacing Standards (*40*)

Cat	Name	Speed (mph)	Min street spacing (ft)	Min driveway spacing (ft)	Signal spacing (ft)
8	C-U	< 30	300	150	1320
6	R-U	30 – 45	350	200	1320
5	R-PU	> 45	660	350	2640
C-U – Community – urban importance R-U – Regional – urban importance R-PU – Regional priority – urban importance					

PERFORMANCE OF TRANSIT AND NON-MOTORIZED MODES

The alternative intersection/interchange types are placed mostly in urban areas, where the right-of-way is shared by different modes of transportation. This section provides an overview of performance measuring for transit, pedestrian and bicycle modes.

Transit Consideration at DDIs and CFIs

Since the alternative intersection designs are constructed at locations where high traffic volumes are observed or expected, there is a great probability that one or more transit lines will traverse these locations. Highway transit modes can be accommodated at alternative designs without any special provisions. However, attention is required when planning and designing transit stops in the vicinity of these intersections/interchanges, especially at CFIs. The footprint of a DDI is similar as for a conventional diamond interchange, so the same guidelines can apply. One should only consider the turning movements of transit vehicles at crossovers, and the minimum distance required for these vehicles to be able to pull into a transit bay at the exit of the DDI. For CFIs, similar consideration for transit stop locations can be followed as described in access management section. It is possible however to have a transit stop located next to the through lane between the crossover and the main intersections, but this has to be carefully considered, since it can pose some safety concerns.

Re-location of left turn movements and reduced number of signal phases could benefit transit operations in terms of overall intersection and corridor delays, as well as the number of stops. Studies on the effects of alternative designs mostly focus on traffic operations, without considering transit operations. Some more recent research efforts show that innovative designs have the ability to improve both vehicular traffic and transit performance, but this research is focused on intersections more adequate for higher density urban environments (*45*). There is a need for further research to explore how transit performs in DDI and CFI environment.

Certain Transit Signal Priority (TSP) strategies can be available at DDIs and CFIs. They also need to be designed with special attention, especially at CFIs, because of the complex signal timing and phasing. Green extension and Early green for transit can be implemented without major modifications, while other strategies may not always be available, and this will depend on a case-by-case basis.

Rail transit modes present a much bigger challenge for implementation at DDIs and CFIs. Within DDIs, the train tracks can follow the main crossover movements (which is not ideal because the tracks need to cross each other twice, if double-tracked), or can be placed in the middle of the roadway. In any case, the designer must consider the space required for train tracks, longitudinal separation from other traffic, location of the overhead power lines, as well as traffic signal control with separate phasing and timing for trains. The performance of rail vehicles (acceleration, deceleration, speed, turning radius) must also be considered in track and signal design. Within a CFI, the train tracks can be placed to the sides of the road, or can run through the middle. Again

special attention must be given to the longitudinal separation and signal phasing and timing, as well as the additional space required for the tracks. The possible location of transit stops will also depend on the location of the tracks, which can have some impacts on passenger accessibility to transit stops. Until the conclusion of this report, the author is aware of one field implementation of a center-running light rail transit line through a DDI in Bloomington, MN (DDI at 34th Ave and I-494). The author is not familiar with any implementations of rail transit modes within CFIs.

The operation of highway transit modes, if there is no special signal treatment, will be similar to vehicular traffic (i.e. similar delay experienced at the intersection/interchange). By knowing the transit frequency and the average transit ridership, one can estimate the total person-based delay for transit users, and therefore the costs associated with it. This approach is used in the performance modules for analyzing performance of transit modes at DDIs and CFIs.

Pedestrian Consideration at DDIs and CFIs

In most cases, pedestrians are also expected to be present at DDIs and CFIs. Depending on the actual solution and locations of pedestrian paths, walkways and crossings, clear guidance should be provided for pedestrians who traverse these locations. Longitudinal separation within DDIs is recommended, since it can improve both objective and subjective safety. Signal phasing and timing for pedestrian phases at DDI crosswalks will depend on the allowable pedestrian movements and location of the crosswalks. In some implementations, pedestrians cannot cross the arterial street within the DDI, so they need to walk to the next available crosswalk. This may encourage unlawful crossings and pose serious safety concerns. For that reason the allowable movements and the location of the available crosswalks in the vicinity of a DDI are very important.

The most significant impact of a CFI design is on pedestrian crossing distance and available crossing time. Because of the larger footprint, pedestrian crossings at a CFI are much longer, since the pedestrians need to cross more travel lanes and medians. Pedestrian crossing times are also longer, which can sometime impact the traffic operations because of the longer required signal cycle length. Pedestrian times should also be separated from left turns at the main intersection, because they are in a direct conflict, and left turns are generally faster at a CFI then within a conventional intersection. Sometimes it may be required to have double-phased pedestrian crossing times, which significantly increases pedestrian delays, and requires additional space within the median for pedestrian refuge, which should also be protected to improve objective and subjective safety. The maximum travel distance for pedestrians is included in the pedestrian performance module, described later in the report.

Pedestrian delay calculation at signalized intersections (which is also applicable to DDIs and CFIs) is described in the Highway Capacity Manual (HCM) (*41*). It is calculated using the following equation:

$$d_p = \frac{(C - g_{walk})^2}{2C} \qquad (47)$$

Where:

d_p – pedestrian delay (s/ped)

C – intersection cycle length (s)

g_{walk} – effective pedestrian green time (s), generally calculated as:

$$g_{walk} = g_{disp} + 4.0 \qquad (48)$$

g_{disp} – pedestrian displayed green time (s)

The calculation of pedestrian delay for different intersection configurations is also included in the pedestrian performance module.

Bicycle Consideration at DDIs and CFIs

Similar disadvantages exist for bicycles at DDIs and CFIs as discussed for pedestrians. If bicycle traffic exists or is expected, it is recommended to design separate bicycle lanes separated from other traffic to increase objective and subjective safety.

Computation of the bicycle delay is provided in the HCM (41). The first step is to compute the bicycle lane capacity as follows:

$$c_b = s_b \cdot \frac{g_b}{C} \qquad (49)$$

Where:

c_b – capacity of the bicycle lane (bicycles/hr)

s_b – saturation flow rate of the bicycle lane = 2,000 (bicycles/hr)

g_b – effective green time for the bicycle lane (s) – assumed here to be the same as the green time for the through movement

C – intersection cycle length (s)

The next step is to compute the bicycle delay as follows:

$$d_b = \frac{0.5 \cdot C \cdot \left(1 - \frac{g_b}{C}\right)^2}{1 - min\left[\frac{v_{bic}}{c_b}, 1.0\right] \cdot \frac{g_b}{C}} \qquad (50)$$

Where d_b is bicycle delay (s/bicycle), v_{bic} is bicycle flow rate (bicycles/h), and other variables as previously defined. Bicycle delay calculation is included in the bicycle performance module.

User Costs and Economic Impacts

User costs at intersection/interchange locations primarily depend on the operational performance, but can also depend on safety and accessibility. User costs can be computed for each mode separately (cars and trucks, transit, pedestrians and bicycles) if the input data are known. The module that computes these impacts uses inputs on the Value-of-Time per person (same for cars, transit riders, pedestrians and bicycles, and a different one for trucks), truck traffic percentage, and all calculated performance measures to determine the user cost delay. Costs associated with safety and accessibility can be indirectly estimated based on the results from the modules.

INNOVATIVE INTERSECTION AND INTERCHANGES PERFORMANCE MODULES

A part of the effort described in this report is a development of Excel-based deterministic applications that analyze performance of different alternatives from various standpoints and provide a direct comparison. Two applications are currently available: an interchange application, which compares conventional diamond interchanges, SPUIs and DDIs; and an intersection application, which compares conventional intersections and CFIs.

Interchange Application

The interchange application consists of six interconnected Excel spreadsheets which take user inputs and perform analysis and comparison among a conventional diamond interchange, a SPUI and a DDI alternative for the given inputs. The names of the spreadsheets must not be changed before the analysis, otherwise the application will not run. Once the analysis is completed, the resulting spreadsheets can be renamed. The following spreadsheets comprise the interchange application.

1. **Diamond DDI SPUI.xlsm**

This is the user input spreadsheet for the interchange application and it is shown in Figure 29. In order to use the spreadsheet, macros have to be enabled upon opening, since the spreadsheet is using Visual Basic for Applications (VBA) to communicate to other modules.

Figure 29: Diamond DDI SPUI.xlsm Spreadsheet Layout

In the top left corner the user needs to input the data for VOT and truck percentage, which is used to determine user costs. The gray boxes are for inputting street/freeway names. The blue boxes are geometry inputs, where the user inputs the number of lanes for each ramp approach. The speed of each approach on the crossroad and exit ramps is inputted in the orange boxes. The red boxes are for volume inputs, which are vehicular turning volumes (that correspond to the intersection geometry), pedestrian volumes, bicycle volumes and transit frequencies. The Right-Turn-On-Red (RTOR) is for assumed right turn percentage during red, and the spreadsheet automatically sets these values to 50% if separate right turn lane is available, or 10% if not. The user may change these values to correspond to the actual field conditions or estimations. In the yellow box, the user has the option to manually select left turn treatment for each left turn movement separately. This is done by selecting "Yes" from the drop-down menu, and then selecting the option for left turn treatment in the corresponding boxes next to each left turn, which will appear once "Yes" has been selected. The options are Protected, Permitted, Protected + Permitted or blank (in which case the corresponding modules will automatically determine left turn treatment). If "No" is selected, than the modules automatically determine left turn treatment for the entire interchange based on geometry and volume inputs.

Once the input boxes have been filled out, the user starts the application by activating the "Evaluate" button. This will transfer the input data to all other modules that estimate different performance metrics. After this is completed, the user will see links in the spreadsheet that will open each of those modules. The "Clear Input" button will remove all entries.

2. QEM_Diamond.xlsm

This is the spreadsheet that contains the operational, transit, pedestrian, bicycle, accessibility and user cost analysis modules. The user inputs from the Diamond DDI SPUI spreadsheet are transferred here for further analysis. The spreadsheet first performs estimation and simple optimization of signal phasing and timing parameters, by determining the interchange cycle length, phase times, delays, LOS and other parameters, and then computes all other performance indicators. The underlying methodology is the HCM Quick Estimation Methodology (QEM, hence the name), as well as additional methodologies described in the Signal Timing Manual (*42*) and UDOT's Signalized Intersection Design Guidelines (*43*). A detailed description of the QEM spreadsheet can be found in (24), while the guide is available online at www.learning-transportation.org, Lecture 4. The original QEM application was developed as a part of a FHWA project "The Effective Integration of Analysis, Modeling and Simulation Tools" (*44*), where it was successfully validated against HCS and Synchro software. For this purpose, it was modified to account for signalized operations at a conventional diamond intersection. The transit, pedestrian, bicycle, accessibility and user cost analysis modules are developed as described previously in this report. The main calculations in this spreadsheet can be seen under the "Phase calculation" tab. A visual representation of performance measures can be seen under the "Summary sheet" tab. It should be noted that the performance measures are calculated for one hour only, typically the peak hour entered in the input spreadsheet.

3. QEM_DDI.xlsm

This is an upgrade of the conventional diamond QEM spreadsheet. It is using calculations described previously in the "Performance Evaluation Methodologies" section for DDI phasing, timing and delay calculation. It is also using user defined inputs from the Diamond DDI SPUI spreadsheet and rearranges them automatically to correspond to the DDI inputs. It provides operational, transit, pedestrian, bicycle, accessibility and user cost results for DDIs.

4. QEM_SPUI.xlsm

Same as the previous two spreadsheets, this one performs analysis for a SPUI alternative based on the given inputs. Signal parameters estimation is customized for SPUI operations.

In the "Input sheet" of the three QEM modules, the user also has an option to change some of the default parameters, such as ideal saturation flow rate, peak hour factor (PHF), lost time per phase, area type, minimum and maximum cycle lengths and pedestrian speed, based on the local existing or estimated conditions.

5. Safety_DDI.xlsx

This is the safety module that calculates crash frequencies for conventional diamond interchanges, DDIs and SPUIs (for which the results are the same as for diamond, since separate SPF is not available). It is using the inputs from the Diamond DDI SPUI spreadsheet and applies the methodologies described in the "Interchange Safety Performance Functions" section. For DDI safety performance, the module uses the CMF for DDI conversion calculated previously. This module provides the total, fatal and injury and property damage only crashes per year for each of the three alternatives. It uses the volume inputs to compute the AADT values, by assuming that the peak hour volumes are equal to 9% of the AADT. This value can be changed in the corresponding AADT cells if needed.

6. Performance Matrix.xlsx

This is the output spreadsheet which combines the results of all previous modules. It is consisted of eight tabs that show different performance measures. The first tab, "Performance matrix", is the general overview of the performance for the three alternatives (conventional diamond, DDI and SPUI interchanges). It is using the Relative Performance Index (RPI) as the main indicator of performance. RPI is based on the best-performing alternative for each performance measure, which gets the value of 100, and the performance of other alternatives is computed based on the optimal. The interface is also color-coded for easier assessment. The remaining seven tabs, "Operational", "Safety", "Peds", "Bikes", "Transit", "Access" and "Econom_impact" provide detailed results from the corresponding modules, as well as the RPI calculation. In this tabs the user can see all the details for each performance measure.

When the user performs the evaluation (by activating "Evaluate" in the Diamond DDI SPUI spreadsheet), the application automatically generates operational and safety result spreadsheets, which are placed in the same folder as the application and can be recognized by the prefix "Interchange [interchange name, according to the crossroad/freeway names given in the input sheet]". These result spreadsheet show all detailed calculations. They can also be opened through the links that will appear in the Diamond DDI SPUI spreadsheet once the analysis is completed.

Intersection Application

The intersection application consists of seven interconnected Excel spreadsheets which take user inputs and perform analysis and comparison among a conventional intersection and a CFI alternative for the given inputs. The names of the spreadsheets must not be changed before the analysis, otherwise the application will not run. Once the analysis is completed, the resulting spreadsheets can be renamed. The following spreadsheets comprise the interchange application.

1. **Conventional CFI.xlsm**

This is the user input spreadsheet for the intersection application, shown in Figure 30. Macros have to be enabled upon opening for the application to run properly. The layout is similar as for the interchange application, with the same color code for input boxes.

Figure 30: Conventional CFI.xlsm Spreadsheet Layout

Additional inputs that a user can select in this module are related to the CFI geometry. From the drop-down menu near the bottom, the user can select a Full or Partial CFI option. Full CFI means

displaced left turns at all intersection approaches, while Partial CFI means displaced left turns at two approaches. For the Partial CFI option, the module automatically selects which approach should be transformed into CFI (E/W or N/S), based on the entered volumes. Another additional CFI input is the presence of the right turn bypass lanes. This can be defined for any of the approaches by selecting "Yes" or "No" from the drop-down menu under the corresponding RT bypass cell.

2. QEM_conventional.xlsm

This is the spreadsheet that contains the operational, transit, pedestrian, bicycle, accessibility and user cost analysis modules for a conventional signalized intersection. It is the same as the QEM modules previously described, only in this case all calculations are customized for a typical 3-leg or 4-leg signalized intersection.

3. QEM_CFI.xlsm, QEM_EBWBCFI.xlsm, QEM_NBSBCFI.xlsm

These are the QEM modules for a full, partial EW and partial NS CFI respectively. They are customized to perform signal phasing and timing estimation for the corresponding CFI configuration. They contain separate calculations for the main intersection and crossovers. First the input data are aggregated into CFI separate inputs, and then the parameters for the main intersection are calculated. Using the same cycle length, the parameters for crossovers are calculated next. Calculation of parameters and performance measures is based on the CFI calculation methodology described previously in the "Performance Evaluation Methodologies" section for CFI phasing, timing and delay calculation. Performance measures for transit, pedestrian, bicycle, accessibility and user costs are then calculated correspondingly.

4. Safety_analysis.xlsx

This is the safety module that calculates crash frequencies for the conventional intersection and CFI alternatives. It uses the methodology described previously in the "Intersection Safety Performance Functions" section. Since intersections have more complex CMF calculation, this module introduces additional input values from HSM and CMF calculation for the given intersection geometry and operations. For CFI safety performance, the module uses the CMF for CFI conversion calculated previously. This module provides the total, fatal and injury and property damage only crashes per year for each of the three alternatives. It uses the volume inputs to compute the AADT values, by assuming that the peak hour volumes are equal to 9% of the AADT. This value can be changed in the corresponding AADT cells if needed.

5. Performance Matrix.xlsx

This is the output spreadsheet which combines the results of all previous modules. It is the same as the spreadsheet for interchange evaluation. It uses RPI and detailed performance measures to compare conventional intersection and CFI alternatives for the given inputs.

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