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*Research Article*

# Developing Warrants for Designing Continuous Flow Intersection

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The increase in the left-turn demand is the main cause of congestion at conventional intersections, and the traditional countermeasures are inadequate to solve this congestion due to the high changes in demand. This paper looks at the justifying threshold to redesign a signalized intersection from a conventional intersection (CI) to a continuous flow intersection (CFI) and create performance guidelines for decision-makers and professionals deciding to consider the alternative. A performance comparison between the CI and CFI was conducted to define the main parameters affecting the operational performance. To accomplish the paper's objective, candidate locations that have already implemented the CFI were identified, and the location with sufficient data for analysis was selected. After the consideration of different evaluation tools, microsimulation (VISSIM 8) was utilized to model the before and after conditions of the location. Using the field data, signal optimization and driving behavior parameter sensitivity analysis were performed to calibrate the models to replicate real-life conditions. Afterwards, an experiment was designed to examine the different factors that affect the efficiency of each design. The experiment involved 72 different configurations of CFI and CI with 5 different volume levels and used two measures of effectiveness, average vehicle delay, and capacity to assess the results. The results were used to develop guidelines that will help the decision-makers to decide which design should be considered, which will result in developing a decision support system that will accelerate finding which design is superior to others.

## 1. Introduction

The aging US highway system is quickly failing to accommodate the increase in demand. The rise in demand throughout the past couple of decades has been primarily a function of the exponential growth of the population. The increased demand to travel long distances is pushing for more time spent on the road, further deteriorating the current system and causing heavy congestion. The congestion at signalized intersections is directly proportional to the high left-turn volume. Operational and safety performances at congested intersections have been identified as the primary focuses of the transportation engineering community working on solutions and countermeasures to enhance the driving experience. Some of the conventional countermeasures to mitigate congestion at an intersection due to high left-turn volume include using the double left-turn lanes, increasing the cycle length, improving the coordination, and synchronizing the signals. At saturated intersections, the adjustment of cycle lengths and improving the signal coordination result in insignificant improvements [1]. Other countermeasures that rely on modifying existing designs such as widening the right-of-way and enhancing alternative routes have proven to be expensive and disruptive to the network [2]. Grade separation has been one of the countermeasures considered to minimize congestion at major intersections. Although grade separation usually leads to significant improvements, it is not always feasible to implement mainly due to the time and costs of construction [3].

When the previous countermeasures failed to deliver their intended purpose, transportation professionals resorted to the development of more innovative designs for intersections. One of the most popular intersection designs is the continuous flow intersection (CFI). The application of this design as a countermeasure for congestion has proven to be superior to others in terms of traffic operation and safety [4, 5]. Another appeal of this design is that it is cost-effective in comparison to other countermeasures.

Many of the previous studies on CFIs and other innovative designs primarily focused on the analysis and evaluation of the operational and safety performance of these designs [6]. The studies compared different designs to one another or to a conventional design [7]. Some studies examined when to convert from the conventional design to the improved innovative design [8, 9]. Other studies compared different CFI designs [10]. However, none of these studies technically examined the justification of a redesign to build their guidelines.

This research will take a closer look at CFIs and the various factors that affect intersection performance due to the increased left-turn demand and examine the justification and need to redesign the intersection in order to enhance their operational efficiency. Using these guidelines, traffic engineers would be able to make a decision which design will meet their operational needs. To build these guidelines, the paper assesses the current strategies for left-turn management at a signalized intersection for their compliance with the intended purposes. It will also assess the effectiveness of CFIs with regard to operational performance. To do so, locations were carefully selected, and the field data were collected from the concerned organizations. The research evaluated different simulation tools that have the ability to imitate the new design configurations and selected a microsimulation tool to model the selected location before and after the implementation of the CFI. The simulation was then complimented with the field data to accurately resemble real-life conditions through the calibration of the models. In addition, an experiment was designed to look at the different factors that affect the efficiency of the designs. The appropriate measures of effectiveness were chosen to look at the threshold at which it is most effective to convert a conventional intersection to a CFI. The paper then evaluates the significance and effectiveness of these measures and the developed warrants.

## 2. Background

*2.1. Current Countermeasure Strategies.* There are several conventional countermeasure strategies available to improve the congestion due to high left-turn volume at signalized intersections. One of these countermeasures is the addition of lanes to the approach causing the congestion. Other countermeasures include adding more green time to the cycle length and enhancing the coordination and synchronization of the signals. These countermeasures result in insignificant improvements at saturated intersections [1]. Moreover, another countermeasure is to create a grade separation between the intersecting approaches. These countermeasures are very effective when it comes to small increases in the left-turn volume demand. However, they are not effective when the changes in volume are drastic [11].

*2.2. Understanding Continuous Flow Intersections.* The first continuous flow intersection in the United States, with ramps in a single quadrant at a T-intersection, was opened in 1994 in Long Island, New York, at an entrance to Dowling College [10]. The main idea of the continuous flow intersections, also known as the crossover displaced left turn (XDL) and displaced left turn (DLT) [1], is to shift the left-turn lanes from the main intersection to a left-turn bay that is placed to the left side of the road by crossing the oncoming through lanes during a protected phase. This arrangement is achieved through the addition of a signalized intersection about 300–700 ft upstream of the main intersection as seen in Figure 1 [2, 11]. Three-phase intersection will be operated if one set of paired subintersections is implemented. If the CFI was implemented with four subintersections ahead of the primary intersection, the intersection will be operated with two signal phases that reduce the conflicts between the movements, as seen in Figure 2; improves the intersection capacity; and reduces the delay [12].

The use of a two-phase signal allows the through and left-turn movements to avoid conflict with oncoming traffic at the main intersection [13]. The right-turn traffic bypasses the main intersection and merges onto the mainstream traffic through the use of a channelized right-turn lane. This in turn allows the through, left-turn, and right-turn movements to operate simultaneously without any potential conflicts. The additional green time, reduced delay, and reduced conflicts can potentially improve the capacity of an intersection between 30% and 70%, as identified in operational and observational studies performed by UDOT [12]. The implementation of the CFI will result in the improvement of the traffic operations and the safety performance.

Table 1 summarizes the advantages and disadvantages of implementing the CFI design. There are two considerations that were agreed upon by most literature regarding the construction of the CFI on an arterial road. The first consideration is when the volume demand is at or over the intersection capacity, and the second is when there is additional right-of-way available along the arterial road near the intersection [4, 11, 13, 14].

## 3. Methodology

*3.1. Study Location and Data Collection.* Since CFIs are relatively new, there were not any innovative intersections implemented in Florida when this study started. The candidate locations that were being considered for this study are outside the state of Florida. There are several new locations in various regions of the United States that have implemented different innovative designs. However, not all of

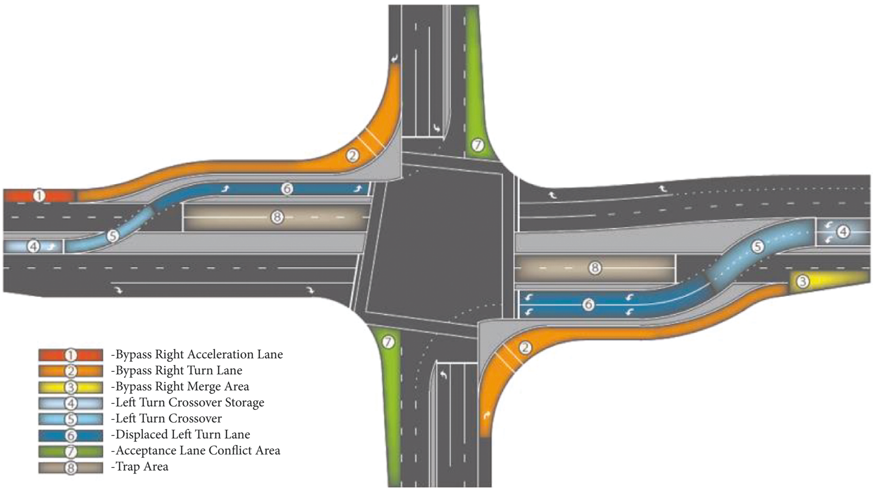


Figure 1: Layout of a two-leg continuous flow intersection [12].

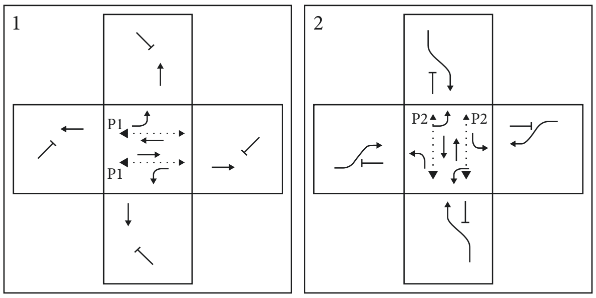


Figure 2: The full CFI signal phasing schemes [12].

Table 1: Advantages and disadvantages of implementing a CFI design.

| Advantages | Disadvantages |
|---|---|
| Reduced delay and travel time for all the movements | Confusion between the driver and pedestrian |
| Reduced number of stops for through arterial traffic | Prohibited U-turn possibilities |
| Increased capacity | Pedestrians cross the intersection in two or more stages |
| Lower cost in comparison to some alternatives | Additional right-of-way |
| Better progression for all movements | Lack of access control |
| Improved safety performance in the intersection | Higher cost in comparison to some alternatives |

them had sufficient data, or it was very hard to get access to the data available for these locations. The candidate pool thus shifted to locations that had already implemented CFIs a while back. Agencies (or authorities) implemented these designs and collected the data at these locations, which was shared for this research. On request, the Federal Highway Admiration (FHWA) provided a list of suggested locations for innovative designs. However, only two of these locations had CFIs that were under construction and were not expected to be ready within the next two years. Later, a professor at Utah University was then contacted, and he was able to provide five different locations for CFIs along the Utah State Route 152 (Bangerter Highway) as follows:

(1) 3100 South in West Valley City, Utah (Implemented)
(2) 3500 South (SR-171) in West Valley City, Utah (Implemented)
(3) 4100 South in West Valley City, Utah (Implemented)
(4) 4700 South in Taylorsville and West Valley City, Utah (Implemented)
(5) 5400 South (SR-173) in Taylorsville, Utah (Implemented)

Since the only intersection that has a four-leg CFI was Bangerter Highway and 4100 S. Rd, this location was chosen as the candidate for the study. This CFI was the first four-leg CFI in the USA; it was built in 2011. The geometric configuration of this CFI is not symmetric; the eastbound and westbound (EB/WB) approaches are similar, while the northbound and southbound (NB/SB) approaches are slightly different. The EB/WB approaches both have one left-turn lane bay, two through lanes, and one shared lane: through and right. While the NB/SB approaches both have three through lanes and one designated right-turn lane, the

NB approach has one left-turn lane bay, but the SB approach has two left-turn lane bays. Consequently, the NB approach was selected for detailed analysis in the following sections. The provided data for this location were for the AM peak hour, which were turning movement counts (TMC), the calculated network performance, average calculated delay, and traffic volumes.

*3.2. Simulation Tool.* Although there are many microsimulation tools available for traffic analysis, none of them can accurately handle the design variations, travel paths, signal timing implications, driver behaviors, and queues. One of the most commonly used microsimulation software is VISSIM, which was mainly selected for its reliability and flexibility. VISSIM V.8 is a microscopic time-based, behavior-based, stochastic simulation tool. It has the ability to: (i) imitate new designs; (ii) simulate signal control plans and/or import signal plans from other tools; (iii) be easily replicated; (iv) run the simulation for random seeds and other factors; (v) collect various measurements throughout the network, allowing a closer look at a different measure of effectiveness; and (vi) develop animated two- and three-dimensional models.

There are numerous simulation parameters that were taken into consideration. One of these parameters is the simulation period. Previous studies have used simulation periods that vary between 15 and 360 minutes; in this study, however, 60 minutes was used for the simulation period, and it was the most used period plus 15 minutes in the beginning to warm up and ensure the system is fully operated and simulate the real life. In order to produce reliable simulation outputs, the models were run using varying replication and seeding numbers [4]. The models should be run using varying replication and seeding numbers; however, one replication number was enough for this study because of the factorial design.

*3.3. Calibration.* Using VISSIM and the aid of images found on Google Earth, two initial models were built for the location at Bangerter Highway and 4100 S. Rd. The first model was for the conventional intersection (CI), which was observed using the images of the period (6/17/2010) before the location was converted into a CFI. The second model was for the continuous flow intersection (CFI), using images dated 7/8/2016 after that location was converted into a CFI (see Figure 3). Using field data, both the models were calibrated through several steps to ensure the model outputs are 95% or higher matched with field data. Since the signal timings of the existing location were not available, the signal optimization step was necessary to be done while calibrating the models. The optimization of the signal plan was done by running the simulation models using different cycle lengths and different splits and looking at the delay time and the capacity. Taking into consideration these two parameters and comparing each of the simulations runs to one another, the signal time splits with the least delay and highest capacity were picked. For the CI 5 signal timings were picked out of 19 different signal timing splits, when comparing them together, the best split showed an 85% match to real life. That signal timing had a cycle length of 90 seconds and split 50 seconds to the NB/SB split equally into 25 seconds for the through and 25 seconds for the left-turn movement and 40 seconds for the WB/EB split equally into 20 seconds for the through and 20 seconds for the left-turn [12, 15]. As for the CFI, 4 signal timings were picked out of 6 different signal timing splits, and when comparing them together, the top-performing signal split showed a 97% match to real life. The signal timing used for the CFI had a 60-second cycle length, split equally 30 seconds and 30 seconds.

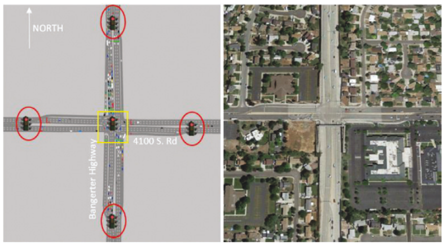


Figure 3: The coded model and screenshot for the CFI location obtained from VISSIM 8.0 and Google Earth.

*3.4. Sensitivity Analysis of Driving Behavior Parameters.* For the driving behavior parameters, a sensitivity analysis was conducted for different levels using the optimal signal timing for both designs. The driving behaviors that were varied were the Wiedmann 99 parameters CC0, CC1, CC2, CC7, and CC8. The CC0 is responsible for the standstill distance [14, 15]. The CC1 is responsible for the headway time, and the CC2 is responsible for the following variation. The CC7 is responsible for the oscillation acceleration. The CC8 is responsible for the standstill acceleration. These five parameters have shown to have the highest effect on the performance of the model [16, 17]. A simulation run was completed as each parameter was varied, while the rest remain in their default values, and the change in throughput was recorded for each run. The parameters were varied by setting two higher and lower points around the default values that result in 25 different simulation runs. The value of each parameter that had the highest positive impact on the throughput was picked for each parameter, and then a final simulation run was completed using all of the new values. For the CI, the new Wiedmann 99 parameters values were $CC0 = 1.64$ ft, $CC1 = 0.7$ sec, $CC2 = 6.56$ ft, $CC7 = 0.66\ \text{ft/s}^2$, and $CC8 = 14.76\ \text{ft/s}^2$.

That led to an increase in capacity from 85% to 95% for the CI. As for the CFI the variation in the Wiedmann 99 parameter, the capacity was either changed negatively or remained the same, leading to the use of the default values, including $CC0 = 4.92$ ft, $CC1 = 0.90$ sec, $CC2 = 13.12$ ft, $CC7 = 0.82\ \text{ft/s}^2$, and $CC8 = 11.48\ \text{ft/s}^2$.

*3.5. Experimental Design.* In order to search for conditions that make a CFI design better than a CI design, it was deemed necessary to design an experiment that encompasses the critical measures of effectiveness. The measures of effectiveness that were used in previous studies included vehicle trips, total delay, moving/total time, delay per vehicle, average speed, storage, phase failure, fuel, HC emissions, NOX emissions, percentage demand, operational and safety performance, average control delay, number of stops, partial and overall capacity, delay for all movements, and CO emissions [8, 13, 18, 19]. The present research selected, for the experiment, the delay time and the capacity of the intersection as the measures of effectiveness. The average delay per vehicle along with the capacity is among the most used measures of effectiveness in the past studies. Using these two measures to compare the before and after conditions of the location would allow for a better understanding of the conditions where conversion to a CFI from a CI is justified. The experiment included a multilevel factorial design that evaluates changes in multiple factors and compares the results using the measures of effectiveness [20]. Five main parameters were considered in the experimental design based on the literature review that proved their effect on the CFI performance. The parameters included the spacing between the main and secondary intersection, number of lanes for the left and through movements, adjacent intersection distance, and volume per hour per lane. The experiment resulted in $3 \times 2$ x $3 \times 2$ x $5 = 180$ scenarios.

The first factor that was varied in the experiment is the spacing distance. In the cases of the CI, the spacing distance was defined as the distance that encapsulates the left lane, while it was defined as the distance between the main intersection and the crossover intersection in the case of the CFI. The spacing distances used in the experiment were 500 ft, 700 ft, and 900 ft to identify the effect of the spacing distance on such design. The second factor that was varied in the experiment is the number of lanes in the intersection. For each spacing distance, the number of lanes was changed for different geometric configurations, which are 1 or 2 left-turn lanes, paired with 2, 3, or 4 through lanes. The NB/SB approaches still had a dedicated right-turn lane, while the WB/EB approaches had one of the through lanes as a shared through and right-turn lane. The third factor that was changed between the scenarios was the distance between the main and adjacent intersections.

The spacing distance of 500 ft, 1,320 ft, and 2,640 ft was used for each configuration. The spacing distance of 700 ft, 1,535 ft, and 2,640 ft was used for each configuration. The spacing distance of 900 ft, 1,750 ft, and 2,640 ft was used for each configuration. The distances 1,320, 1,535, and 1,750 were different for each spacing distance. The spacing between the main and secondary intersections increased the distance to the adjacent intersection and became insufficient to clear the traffic, which resulted in blocking the intersection. So it was needed to increase the distance between the adjacent intersections. However, the 2,640 ft distance between the adjacent intersections was enough to clear the traffic for all three spacing distances. For each of these scenarios, the theoretical capacity was vehicle per hour per lane and was varied between 250, 500, 750, 1,000, and 1,250 vehicles per hour per lane while allotting 5% of the total volume to the right turners. Each volume per lane scenario multiplied by the number of lanes per approach resulted in the total volume per approach. The distances on which the delay was measured varied relative to the distance between the main intersection and the adjacent intersection. For an adjacent intersection at 1,320 ft, the left-turn delay was measured based on 800 ft distance. For an adjacent intersection at 1,535 ft, 1,200 ft was used to measure the left-turn delay. For an adjacent distance of 1,750 ft, the left-turn delay was measured based on 1,470 ft distance. As for all the configurations with 2,640 ft of adjacent distance, the same distances were used for them from their shorter counterparts. As the adjacent distance increases, the distance to measure the delay increases resulting in the variation of the distance between the three scenarios.

During the design of each experiment for this study, a balanced condition and an unbalanced condition were considered. The unbalanced condition means the volume per lane for the minor road is the percentage (25%, 50%, and 75%) of the volume per lane of the major road, and the balanced condition means the same volume per lane used for the four approaches. In order to come up with a conclusive study, the unbalanced condition was first tested, through multiple runs at different volumes. The unbalanced conditions did not show any significant advantage over the balanced condition, as the capacity of each unbalanced condition was very close to each other over the varying volume. The experiment proceeded using only the balanced conditions. Table 2 summarizes the design experiment that was carried out for both the CI and the CFI.

## 4. Results and Analysis

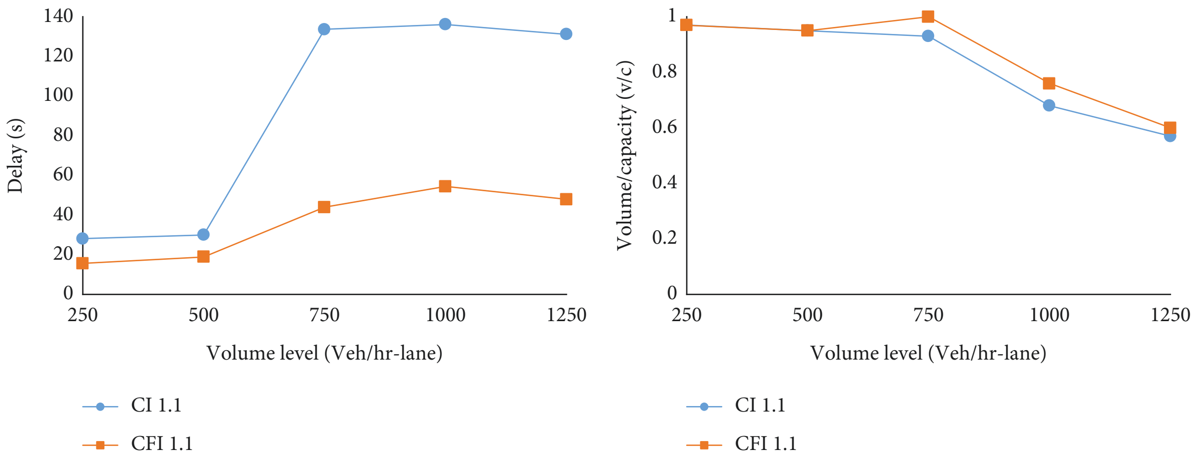
The output from the simulation runs was then used to evaluate the conditions that warrant a CFI design. The analysis focused on the two measures of effectiveness that were the NB left-turn (LT) delay and NB LT capacity. Table 1 was used as a reference for this analysis for group 1.

Looking at the results from scenario group 1, comparing iterations 1.1, 2.1, and 3.1 with regard to NB LT delay and NB LT capacity, the CFI outperforms the CI (see Figure 4). When comparing the results for 4.1, 5.1, and 6.1 from group 1, the CFI outperforms the CI with regard to delay time in iterations 4.1, 5.1, and 6.1; on the other hand, the CFI outperformed the CI at the first two volume levels. However, the CI and CFI had similar performance with regard to NB LT capacity. When comparing the results for 1.2, 2.2, and 3.2 from group 1, the CFI outperformed the CI with respect to NB LT delay and NB LT capacity. When comparing the results for 4.2, 5.2, and 6.2 from group 1, both designs performed the same with respect to NB LT capacity. With respect to the NB LT delay, for 4.2 and 5.2 scenarios, the CI and CFI performed the same at volume level of 750 vehicles per hour per lane and higher, while the CFI outperformed the CI in the 6.2 iteration. The results from 1.1, 2.1, 3.1, 4.1, 5.1, and 6.1 were very similar to their counterparts, except for 5.1 and 5.2 with regard to NB LT delay [16].

Comparing the results for iterations 1.1, 2.1, and 3.1 from group 2, CFI outperformed the CI in terms of delay and capacity. When comparing the results for 4.1, 5.1, and 6.1 from group 2, with respect to the NB LT delay, the 5.1 and 6.1

Table 2: Experimental design description.

| L | Subgroup | Iteration | Scenario no. | Spacing distances (ft) | Number of lanes |  | Adjacent intersection distance (ft) | 250 | 500 | 750 | 1,000 | 1,250 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  |  |  | LT | Thru |  | Total volume (veh/hr) |  |  |  |  |
| 1 | 1 | 1.1 | 1 | 500 | 1 | 2 | 1,320 | 790 | 1,579 | 2,368 | 3,158 | 3,947 |
|  |  | 1.2 | 2 | 500 | 1 | 2 | 2,640 |  |  |  |  |  |
|  | 2 | 2.1 | 3 | 500 | 1 | 3 | 1,320 | 1,053 | 2,105 | 3,158 | 4,211 | 5,263 |
|  |  | 2.2 | 4 | 500 | 1 | 3 | 2,640 |  |  |  |  |  |
|  | 3 | 3.1 | 5 | 500 | 1 | 4 | 1,320 | 1,316 | 2,362 | 3,947 | 5,263 | 6,579 |
|  |  | 3.2 | 6 | 500 | 1 | 4 | 2,640 |  |  |  |  |  |
|  | 4 | 4.1 | 7 | 500 | 2 | 2 | 1,320 | 1,053 | 2,105 | 3,158 | 4,211 | 5,263 |
|  |  | 4.2 | 8 | 500 | 2 | 2 | 2,640 |  |  |  |  |  |
|  | 5 | 5.1 | 9 | 500 | 2 | 3 | 1,320 | 1,316 | 2,362 | 3,947 | 5,263 | 6,579 |
|  |  | 5.2 | 10 | 500 | 2 | 3 | 2,640 |  |  |  |  |  |
|  | 6 | 6.1 | 11 | 500 | 2 | 4 | 1,320 | 1,579 | 3,158 | 4,737 | 6,316 | 7,895 |
|  |  | 6.2 | 12 | 500 | 2 | 4 | 2,640 |  |  |  |  |  |
| 2 | 1 | 1.1 | 13 | 700 | 1 | 2 | 1,535 | 790 | 1,579 | 2,368 | 3,158 | 3,947 |
|  |  | 1.2 | 14 | 700 | 1 | 2 | 2,640 |  |  |  |  |  |
|  | 2 | 2.1 | 15 | 700 | 1 | 3 | 1,535 | 1,053 | 2,105 | 3,158 | 4,211 | 5,263 |
|  |  | 2.2 | 16 | 700 | 1 | 3 | 2,640 |  |  |  |  |  |
|  | 3 | 3.1 | 17 | 700 | 1 | 4 | 1,535 | 1,316 | 2,362 | 3,947 | 5,263 | 6,579 |
|  |  | 3.2 | 18 | 700 | 1 | 4 | 2,640 |  |  |  |  |  |
|  | 4 | 4.1 | 19 | 700 | 2 | 2 | 1,535 | 1,053 | 2,105 | 3,158 | 4,211 | 5,263 |
|  |  | 4.2 | 20 | 700 | 2 | 2 | 2,640 |  |  |  |  |  |
|  | 5 | 5.1 | 21 | 700 | 2 | 3 | 1,535 | 1,316 | 2,362 | 3,947 | 5,263 | 6,579 |
|  |  | 5.2 | 22 | 700 | 2 | 3 | 2,640 |  |  |  |  |  |
|  | 6 | 6.1 | 23 | 700 | 2 | 4 | 1,535 | 1,579 | 3,158 | 4,737 | 6,316 | 7,895 |
|  |  | 6.2 | 24 | 700 | 2 | 4 | 2,640 |  |  |  |  |  |
| 3 | 1 | 1.1 | 25 | 900 | 1 | 2 | 1,750 | 790 | 1,579 | 2,368 | 3,158 | 3,947 |
|  |  | 1.2 | 26 | 900 | 1 | 2 | 2,640 |  |  |  |  |  |
|  | 2 | 2.1 | 27 | 900 | 1 | 3 | 1,750 | 1,053 | 2,105 | 3,158 | 4,211 | 5,263 |
|  |  | 2.2 | 28 | 900 | 1 | 3 | 2,640 |  |  |  |  |  |
|  | 3 | 3.1 | 29 | 900 | 1 | 4 | 1,750 | 1,316 | 2,362 | 3,947 | 5,263 | 6,579 |
|  |  | 3.2 | 30 | 900 | 1 | 4 | 2,640 |  |  |  |  |  |
|  | 4 | 4.1 | 31 | 900 | 2 | 2 | 1,750 | 1,053 | 2,105 | 3,158 | 4,211 | 5,263 |
|  |  | 4.2 | 32 | 900 | 2 | 2 | 2,640 |  |  |  |  |  |
|  | 5 | 5.1 | 33 | 900 | 2 | 3 | 1,750 | 1,316 | 2,362 | 3,947 | 5,263 | 6,579 |
|  |  | 5.2 | 34 | 900 | 2 | 3 | 2,640 |  |  |  |  |  |
|  | 6 | 6.1 | 35 | 900 | 2 | 4 | 1,750 | 1,579 | 3,158 | 4,737 | 6,316 | 7,895 |
|  |  | 6.2 | 36 | 900 | 2 | 4 | 2,640 |  |  |  |  |  |



(a)

(b)

Figure 4: Delay and capacity for the CI versus CFI, group 1: (a) left-turn delay and (b) left-turn capacity.

Table 3: Scenario group 1 results.

| Scenario Gr.3 | | Adjacent distance, 1,320 ft | | | | | | Adjacent distance, 2,640 ft | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spacing distance 900 ft | Volume per lane | Single LT lane | | | Double LT lane | | | Single LT lane | | | Double LT lane | | |
| | | 1.1 | 2.1 | 3.1 | 4.1 | 5.1 | 6.1 | 1.2 | 2.2 | 3.2 | 4.2 | 5.2 | 6.2 |
| NB LT delay | 250 | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes |
| | 500 | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes |
| | 750 | Yes | Yes | Yes | No | Yes | Yes | Yes | Yes | Yes | No | No | Yes |
| | 1,000 | Yes | Yes | Yes | No | Yes | Yes | Yes | Yes | Yes | No | No | Yes |
| | 1,250 | Yes | Yes | Yes | No | Yes | Yes | Yes | Yes | Yes | No | No | Yes |
| NB LT capacity | 250 | No | Yes | Yes | No | No | No | No | Yes | Yes | No | No | No |
| | 500 | Yes | Yes | Yes | No | E | E | Yes | Yes | Yes | No | E | Yes |
| | 750 | Yes | Yes | Yes | No | No | No | Yes | Yes | Yes | No | No | No |
| | 1,000 | Yes | Yes | Yes | No | No | No | Yes | Yes | Yes | No | No | No |
| | 1,250 | Yes | Yes | Yes | No | No | No | Yes | Yes | Yes | No | No | No |
| Yes | | CFI outperforms CI | | | | | | | | | | | |
| No | | CFI does not outperform CI | | | | | | | | | | | |
| E | | CFI and CI are performing equally | | | | | | | | | | | |

Table 4: Scenario group 3 results.

| Scenario gr. 3 | | Adjacent distance 1,320 ft | | | | | | Adjacent distance 2,640 ft | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spacing distance 900 ft | Volume per lane | Single LT lane | | | Double LT lane | | | Single LT lane | | | Double LT lane | | |
| | | 1.1 | 2.1 | 3.1 | 4.1 | 5.1 | 6.1 | 1.2 | 2.2 | 3.2 | 4.2 | 5.2 | 6.2 |
| NB LT delay | 250 | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes |
| | 500 | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes |
| | 750 | Yes | Yes | Yes | No | Yes | Yes | Yes | Yes | Yes | No | No | Yes |
| | 1,000 | Yes | Yes | Yes | No | Yes | Yes | Yes | Yes | Yes | No | No | Yes |
| | 1,250 | Yes | Yes | Yes | No | Yes | Yes | Yes | Yes | Yes | No | No | Yes |
| NB LT capacity | 250 | No | Yes | Yes | No | No | No | No | Yes | Yes | No | No | No |
| | 500 | Yes | Yes | Yes | No | E | E | Yes | Yes | Yes | No | E | Yes |
| | 750 | Yes | Yes | Yes | No | No | No | Yes | Yes | Yes | No | No | No |
| | 1,000 | Yes | Yes | Yes | No | No | No | Yes | Yes | Yes | No | No | No |
| | 1,250 | Yes | Yes | Yes | No | No | No | Yes | Yes | Yes | No | No | No |
| Yes | | CFI outperforms CI | | | | | | | | | | | |
| No | | CFI does not outperform CI | | | | | | | | | | | |
| E | | CFI and CI are performing equally | | | | | | | | | | | |

iterations showed that the CFI outperformed the CI, while, in 4.1, there was no significant difference between the two designs was observed. With respect to NB LT capacity, both designs performed the same. When comparing iterations 1.2, 2.2, and 3.2, with regard to NB LT delay and NB LT capacity, the CFI outperformed the CI. The comparison between the results for iterations 4.2, 5.2, and 6.2 with regard to delay 4.2 and 5.2 shows that both the designs had the same performance when they reached 750 vehicles per hour per lane level; however, the CFI outperformed the CI at 6.2. With regard to NB LT capacity, the CI and CFI performed the same in most of the 4.2, 5.2, and 6.2 scenarios. Table 3 presents the results for scenario group 1. When looking at the group 2 iterations that are similar to group 1 results, the CFI outperformed the CI with a single left-turn lane in most of the scenarios with respect to the delay. The CFI outperformed the CI in terms of capacity for most of the iterations with a single left-turn lane; however, there was no significant difference between the CI and CFI performance with double left-turn lanes, which could be attributed to the signal optimization and the coordination between the main and secondary intersections. Also, the balanced approach may contribute to this insignificance between the CI and CFI due to the fact that the intersection is heavily congested at the 750 vehicles per hour per lane volume level on all four approaches.

The results for iterations 1.1, 1.2, 2.1, 2.2, 3.1, and 3.2 from group 3 all show that the CFI outperformed the CI with regard to all NB LT delay and NB LT capacity (see Table 4). Evaluating iterations 4.1, 5.1, and 6.1 from group 3 with regard to NB LT delay in 4.1 showed no significant difference between the two design performances when reached 750 volume per lane or more due to the reasons mentioned above, while, for iterations 5.1 and 6.1, the CFI outperformed the CI. However, both the designs showed no significant difference regarding the LT capacities for all three iterations. The results for scenarios 4.2, 5.2, and 6.2 from group 3, with regard to NB LT delay, showed that both the designs had the same delay when they reached 750 vehicles per hour per lane level in iterations 4.2 and 5.2, while, in 6.2, the CFI outperformed the CI. In all three iterations, the CI performed the same as the CFI with respect to NB LT capacity. The only difference between 1.1, 2.1, 3.1, 4.1, 5.1, and 6.1 compared to their counterparts was that iterations 5.1 and 5.2 with regard

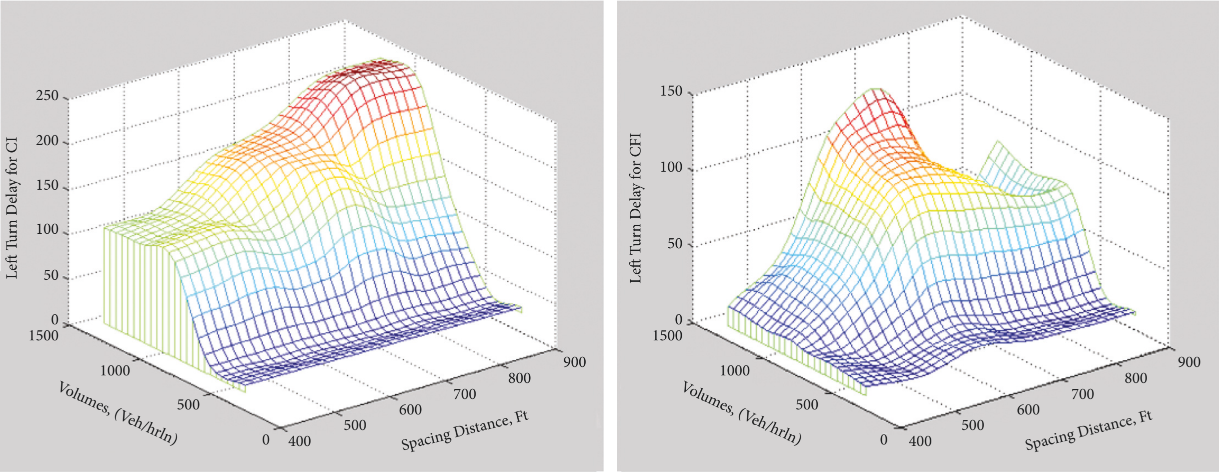


Figure 5: Left-turn delay for CI and CFI for various spacing distances.

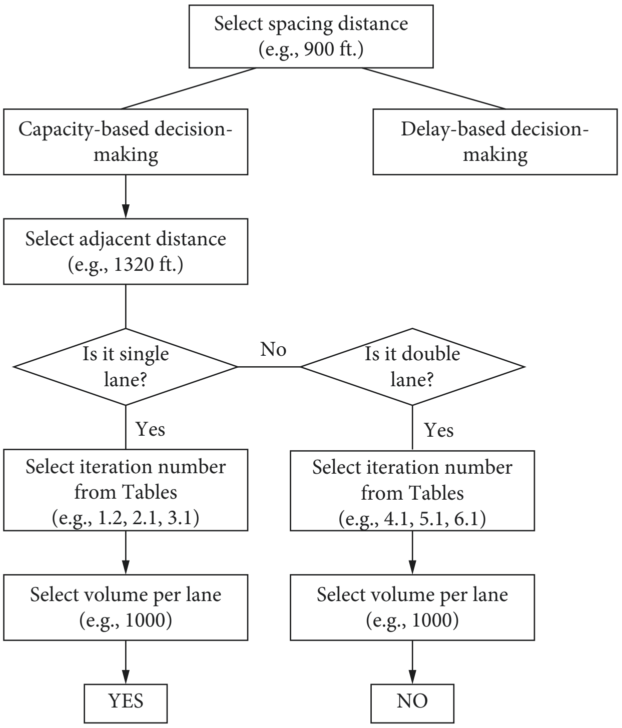


Figure 6: Decision-making framework for switching from conventional intersection to the CFI. Delay-based decision-making also follows the same flow of activities as capacity-based decision-making (see Tables 3 and 4 for details).

to NB LT delay. The CFI LT delay and capacity performed better than the CI on all the single left-turn lane scenarios; however, there was no significant difference between the two designs' performances in the high volume scenarios that are attributed to the balanced approach effect and the signal optimization and coordination between the main and secondary intersections since all other parameters were constant.

## 5. Discussion and Conclusion

Taking into consideration the results and analysis, the apparent trend seems to be that when comparing a single conventional left-turn lane and a single left-turn CFI, the CFI seems to have better performance in terms of delay and capacity than the CI. However, there was no significant difference between the double left-turn lane CI and the double left-turn CFI in terms of capacity for most of the scenarios. However, when comparing a double left-turn lane CI and a double left-turn lane CFI in terms of delay, the CFI seems to outperform the CI as the number of through lanes increases. The similarity between the CI and CFI in some LT capacity results was attributed to the signal optimization and/or coordination between the main and secondary intersections. Also, the balanced approach might cause these fluctuations in the results between the CI and CFI capacity results since the same volume per hour per lane was assumed for all the four approaches. In addition, the results show that the CFI is improving the delay in most of the cases compared to the other design. The results show that increasing the spacing distance between the main and secondary intersection will increase the delay (see Figure 5). The distance between the main intersection and the adjacent intersection seemed to have a significant effect on the performance of the CFI. However, when taking the queue length into consideration, the intersections with longer adjacent distances were able to accommodate the long queue lengths. When looking at iterations 1.1, the trend seems to support past literature that suggest the CFI outperforms the CI at higher left-turn volumes. The results of this study show that the cross points between the CI and CFI capacities happened at the volume level range from 500 to 750 vehicles per hour per lane, the range increases as the spacing distance increases with single left-turn scenarios, and the difference between the CI and CFI delay increases at the same volume range with more superiority to the CFI design. The results are summarized in Figures 4 and 5 showing the contour surface for the volumes, spacing distance, and the response variable of left-turn delay.

The results of the analysis would help the decision-makers make their decision. Figure 6 illustrates a decision-making framework for switching from conventional intersection to the CFI. These guidelines would help the traffic engineers pick the best configuration of such design that would enhance their existing condition design. Based on the framework shown in Figure 6, a decision support system can be designed to facilitate the decision-makers by entering the site-specific information into the system and generate the results for the given situation. It is also expected that the developed methodology will be validated for scenarios different from the present study. [21–23].

## Data Availability

The data that are not included in the manuscript cannot be shared due to the confidentiality agreement between the research and data-sharing organizations.

## Disclosure

A part of this paper belongs to the PhD dissertation of the first author titled: Developing Warrants for Designing Continuous Flow Intersection and Diverging Diamond Interchange completed at the University of Central Florida, USA.

## Conflicts of Interest

The authors declare that they have no conflicts of interest.

## Authors' Contributions

Hatem Abou-Senna was involved in supervision. Essam Radwan contributed to conceptualization and funding. Tarek Lotfy Kamal contributed to data preparation. Fawaz Al Harbi, Mohammed Alfawzan, and Husnain Haider assisted in conceptualization, presentation, and proof reading.

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