MAXIMIZING INTERSECTION CAPACITY AND MINIMIZING DELAY
THROUGH UNCONVENTIONAL GEOMETRIC DESIGN OF CONTINUOUS-FLOW
INTERSECTIONS

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INTERSECTIONS

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Thesis

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ABSTRACT

Typical four phase four legged intersection often operate inefficiently restricting the vehicle throughput thus resulting in larger delays. The impact of left turns is high for the larger delays and is also the most significant factor contributing to the level of service of the conventional intersections. So, by altering the geometric design of the conventional intersection, capacity can be increased and intersection delay can be decreased.

Two phase intersections (2-PI) utilize an unconventional lane arrangement in order to maximize the vehicular throughput. This arrangement involves displacing left turn lanes across opposing through traffic before the main intersection is reached. Such an alteration allows left and through vehicles to proceed simultaneously, and consequently, both the intersection capacity and delay are improved. Numerous studies have validated the operational improvements associated with such a geometric design, but in-depth analysis of this unconventional system can provide a maximized effect.

The main aim of the research is to show how split 2-phase unconventional intersections can minimize delay and maximize capacity for the same given cycle length when compared to the conventional intersections and also how pedestrian safety is increased when compared to the regular two phase and conventional intersections. The main research goal is obtained by setting two objectives. The first objective is to minimize delay of the proposed unconventional intersection when the volume is low. The
second objective is to maximize capacity when the volume is high and congested. A non-linear programming model for finding optimum cycles times for both the objective functions under different volume conditions is developed using MATLAB and the results were used in VISSIM to run simulations and check the accuracy of model.

After defining the main objective functions and a list of constraints, preliminary tests were performed to show the effectiveness of the method adapting to changes in traffic demand. The results found from the optimization were used to run simulation tests comparing the capacity and delay at a split 2-phase intersection to those from a conventional four-legged geometric design. VISSIM was used for analysis of unconventional intersection and also for conventional intersection for comparison. The results of several trials have strongly showed the optimum performance of the proposed unconventional geometric arrangement.
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CHAPTER I

INTRODUCTION

Transportation engineers are constantly looking for innovative ways to improve the capacity and overall operation of roadway intersections. The number of vehicles within the United States transportation network seems to be growing each year, and intersection functionality is becoming increasingly strained due to traditional geometric designs. The history of roadway interchanges has evolved over time to provide new configurations that have helped ease the situation, with each new concept allowing for a higher capacity and lower delay than its predecessor.

One factor that can significantly hinder the performance of an intersection is heavy left-turn movements. A typical four-phase, four-legged intersection and its accompanying phase diagram are shown in Figures 1.1 and 1.2. Depending on the phase during which the intersection is reached, vehicles and pedestrians alike may have to wait for up to three phases to complete their movements. Neither pedestrians nor through vehicles can cross during Phases 1 and 3, with only left turns being serviced. This limits the capacity of the intersection and may cause most vehicles to experience a longer delay. In addition, pedestrian safety is compromised by permitted right or left turns during Phases 2 and 4. As a result of these shortcomings, unconventional intersection geometries...
have been investigated to help improve operations at four-legged intersections with high left turn volumes.

Figure 1.1: Typical four legged Intersection

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Figure 1.2: Phasing Diagram of typical four legged Intersection

1.1 Statement of Problem

Traffic congestion and increased delay at the signalized intersection is mainly caused due to the high left turn volumes. Research have been done to overcome this
problem and adopted many conventional measures including adding lanes, signal planning and double left turn lanes. The use of the conventional measures is limited as the modifications of the intersection design such as widening and building bypasses is limited due to increase in volume year by year [4]. Due to left turning movements in the typical four legged intersection, the delay is increased and capacity decreased since all other movements are restricted during left turn movements. Also the pedestrian safety is significantly compromised in the conventional intersections due to the permitted right and left turns.

In the typical four legged intersection there are four left turning movements, four through movements, four right turning movements resulting in 16 conflict points, in which 12 conflict points are due to the left turn directional movements alone. Impact of the left turning movements is significantly high in the intersection delay and capacity. All conflict points in the typical four legged intersection with all possible movements are shown in Figure 1.3. Intersections having high left turning volumes will have higher delay due to which the capacity of the intersection significantly decreases.

1.2 Research Objectives and Methodology

Many researchers suggested that safety and operational efficiency are the major benefits of unconventional 2-PI over conventional intersection design. Unconventional intersections not only improve safety and operational efficiency but also decrease stopped delay, queue length and also can achieve significant reduction in total accidents and accident severity [5]. In order to maximize the intersection capacity and minimize the delay, unconventional split 2-phase intersection is introduced. In the two phase
intersections, left turning and through movements takes place simultaneously. In this geometry, the left turn lanes are displaced across the opposing through traffic before the main intersection is reached. In comparison with the 2-PI, pedestrians move in 2 phases where the safety is highly compromised where as in the split 2-phase intersection they move in a single phase as they do in atypical four legged intersection.

![Figure 1.3: Conflict Points in a Typical Four-Legged Intersection](image)

The two main objectives in the research are

- Minimizing Intersection Delay.
- Maximizing Capacity.

A non linear programming model was developed in MATLAB and results were compared in VISSIM with different volume levels in various approaches. Based on the volume levels in each approach, different green times for each phase and total cycle time are obtained from MATLAB and these values are used in VISSIM to check the accuracy
of model developed in MATLAB. The results of the various simulation results showed that the proposed two phase geometry is optimum for minimizing delay and maximizing capacity at high left turn volumes.

1.3. Overview of Thesis

A brief outline of the chapters within this thesis is described in sections 1.3.1 through 1.3.5. Detailed discussion of these chapters II through IV are provided later in the thesis.

1.3.1 Chapter II

The literature review in Chapter II discusses the idea of displaced left turns and several studies that have been conducted to evaluate the effectiveness of unconventional intersections. This chapter also includes the locations of different 2-PI intersections that are implemented in different parts of the United States. The chapter concludes with how the 2-PI is efficient in handling high volumes and also decreasing delay at the same time.

1.3.2 Chapter III

Chapter III describes the objectives of the study and different assumptions made and equations used in the study. Methods implemented in achieving the mentioned objectives are discussed in this chapter.

1.3.3 Chapter IV

Chapter IV describes the results obtained from different models and methods used in achieving the objectives. Comparison between the results obtained from methods is also discussed in this chapter.
1.3.5 Chapter V

In Chapter V, the conclusions are made from the results obtained and are summarized. Some of the recommendations about the methods implemented and future work that can be done to improve the analysis are also discussed in this chapter.
CHAPTER II

REVIEW OF LITERATURE

2.1 Introduction to the Concept of 2-PI

The idea of using displaced left turn lanes was first suggested in 1986 by Francisco D. Mier and Belisario H. Romo (1). In 2-PI, all left-turning vehicles, cross over lanes of opposing through traffic before the main intersection is reached. This movement safely occurs through the operation of four surrounding minor intersections, referred to as satellites in the analysis. With such a configuration, through and left-turn vehicles advance simultaneously while right-turning vehicles from the cross-street are also protected. As a result, generally only two phases are necessary and this is referred to as a two phase intersection (2-PI).

The 2-PI services a much larger volume of vehicles for a significantly lower cost than overpass construction as no structures are needed (2). Other benefits include its contribution to maintaining the quality of the surrounding environment. A typical four-phase intersection can leave most of the vehicles queued for a majority of the cycle length, making idling vehicle emissions a concern. The 2-PI can help reduce fuel consumption and noise levels while increasing the air quality in the immediate area (1).
2.1 Research on 2-PI

Several studies have been conducted to evaluate the effectiveness of the 2-PI and its practicality as a safe and affordable alternative. One such study as performed by Esaway and Tarek (3) sought to compare two unconventional intersection configurations with a traditional four-phase intersection, using the average control delay as a measure of performance. The two intersections analyzed included the 2-PI and the upstream signalized crossover (USC). The main difference between the 2-PI and the USC schemes is that the USC crosses both the left and through traffic to the left side of the roadway. Using VISSIM software, the results found that the 2-PI outperformed both the USC and the traditional intersection under balanced and unbalanced volume conditions. The extensive left turning bays present between the main intersection and each of the four minor intersections were believed to provide additional capacity to the 2-PI over the USC.

Another study performed by Dhatrak, Edara, and Bared (4) compared the 2-PI with the parallel flow intersection (PFI). The PFI differs from the 2-PI in that a bypass turn lane is used to eliminate left turn conflicts. Left vehicles from the main intersection turn into the bypass lane along the cross street and they are merged with the through traffic by means of secondary intersections. These intersection schemes were compared based on the number of vehicles serviced and the intersection delay. Although the throughput values for the through traffic were very similar in nearly every case, the 2-PI intersection serviced a larger number of vehicles and also yielded a smaller delay when
left-turning traffic was considered. These findings were assumed to be the result of an average of one additional stop experienced by vehicles in the PFI configuration.

Cheong, Rahwanji, and Chang (5) conducted research to compare all three unconventional intersections (2-PI, PFI, and USC). The average delay was compared for both through-only and left-only traffic for each of the three configurations, and the findings again showed that the 2-PI generally outperformed the others. However, the study concluded that under certain traffic conditions, the PFI actually performed similarly or slightly better than the 2-PI. For areas that have limited right-of-way or that must provide access to neighboring businesses, it was concluded that the PFI can be a good alternative to the 2-PI.

Park and Rakha (6) looked at the safety and environmental impacts that the 2-PI has on its surroundings. Research was performed to see how drivers responded to an intersection configuration that they were unfamiliar with. The study kept track of the number and type of accidents that occurred in the time period immediately following the installation of the 2-PI. Annual crash data was compared after the first and second years of operation, and the number of incidences was found to decrease by about 50% after one year of operation. This showed that although some problems may emerge from initial driver confusion, this is likely a temporary condition. When environmental factors were compared between the 2-PI and a typical four-phase intersection, the unconventional system was found to provide notable improvements. The energy savings were estimated between 5% and 11% while the reduction in vehicle emissions was estimated at 1% to
6%. These environmental improvements were expected at both low and high volumes of left-turning movements.

Finally, research performed by Autey, Sayed, and Esaway (7) analyzed the intersection delay of the 2-PI and three other unconventional schemes in hopes of compiling guidelines in regards to their implementation. All of the intersections were modeled under high, low, balanced, and unbalanced volume conditions. Each unconventional intersection outperformed a standard intersection, though the 2-PI provided the greatest impact with an increase in capacity reaching over 90%. Based on these findings, the 2-PI was recommended for an intersection with high, balanced volume conditions as well as for all unbalanced volume conditions.

2.2 Existing Two Phase Intersections (2-PIs)

2-PI also known as Continuous Flow Intersection (CFI) or displaced or crossover displaced left turns are first introduced in Mexico over 20 years ago. These intersections are also implemented in United States in different states.

The first 2-PI was built in 1996 at a T-intersection entrance to a research center on the campus of Dowling College in Long Island, New York. Figure 2.1 shows the T-intersection constructed in the campus of Dowling College. The intersection opened in the Dowling college is the prototype of the 2-PI [8]. The second T-intersection the first on a state highway is built in 2000 in Accokeek, Maryland. Figure 2.2 shows the T-intersection constructed in Maryland. The third intersection was built in 2006 in Baton Rouge, Louisiana. It is also stated as one of the first high profile intersection built in United States. The delay before the construction was about 4 minutes per vehicle and was
reduced to 1 minute per vehicle after the construction [9]. Figure 2.3 shows the intersection in Baton Rouge, Louisiana. Figure 2.4 shows the fourth CFI that was built in 2007 in Salt Lake, Utah. In the same year the fifth intersection was built in St. Louis, Missouri [9]. Figure 2.5 shows the CFI in St. Louis, Missouri.

Many other 2-PIs are also built in Ohio, Colorado, Mississippi, New York, and New Jersey. More than 15 intersections are built in United States and many are under construction.

Figure 2.1: T-intersection in Long Island, New York.
Figure 2.2: T-intersection in Accokeek, Maryland.

Figure 2.3: 2-PI in Baton Rouge, Louisiana.
Figure 2.4: 2-PI in Salt Lake, Utah.

Figure 2.5: 2-PI in St. Louis, Missouri.
2.3 Concluding Remarks

Clearly, the studies conducted and increase in the number of 2-PIs over the past few years clearly show that the 2-PI is a very promising geometric and operational improvement and has proved to both increase the capacity and decrease the delay in comparison to most other at-grade intersection schemes. However, there are disadvantages to using this type of configuration. For example, a large amount of right-of-way is required for the construction of a 2-PI. The intersection could potentially be ten or more lanes wide across each leg, consuming a very large amount of space. In addition, the way in which the lanes are configured makes access to surrounding businesses extremely limited. Nonetheless, locations suffering from unsatisfactory operations can overcome these drawbacks and ultimately benefit from the use of this unconventional scheme.

Although the installation of a 2-PI alone is shown to improve the performance of an intersection, taking into account the signal timing ensures optimal performance. Previous studies have mostly focused on comparing different unconventional configurations of an intersection through simulation; very limited effort thus far has been given to the system optimization of a 2-PI for capacity enhancement and reducing delay. Motivated by such a need, research is made to conduct a system analysis of a 2-PI configuration, considering the effects of pedestrians, residual queues, and satellite signalization on the overall performance. By formulating a system of equations to represent this configuration during optimization, objective function of maximizing throughput for congested flows and minimizing delay for less congested flows is used.
CHAPTER III

PROPOSED SIGNAL OPTIMIZATION MODEL

3.1 Introduction

Both the typical four-phase, four-legged intersection and the 2-PI provide less than desirable conditions for pedestrian crossing. As stated previously, permitted right and left turns can conflict with pedestrian crossing at traditional intersections. Crossing at a 2-PI is also difficult and potentially unsafe because pedestrians must wait in islands inside the intersection as vehicles may move in front of or behind them until they are permitted to cross. Figure 3.1 shows the geometry of 2-PI and Figure 3.2 demonstrates the movements that must be made at a 2-PI in order for a pedestrian to completely cross. If a pedestrian begins at point A, they will be able to cross to point C during phase 1. They will utilize an island within the intersection at point C as they wait until the next phase due to right turning vehicles from the cross street. During phase 2, they are able to move to points D and E, completing their movement.

Thus far, very few studies have addressed the unsafe method of pedestrian crossing at a 2-PI. To enhance pedestrian safety and comfort, we propose a split two phase operation with a designated time interval for pedestrians to cross the street. This is done by dividing each phase into two parts as shown in the phase diagram in Figure 3.4. With this alteration, pedestrians do not need to wait inside the intersection and crossing
can be completed in one phase. In addition, pedestrians will not have to worry about vehicles turning through the crosswalk because right turn on red is not permitted. This variation, referred to as a split two phase intersection (split 2-PI), is pictured in Figure 3.3.

Figure 3.1: Geometric Design of 2-PI
Figure 3.2: Pedestrian Movement in 2-PI

Figure 3.3: Split 2-PI

Figure 3.4: Phasing Diagram of Split 2-PI
3.2 Objective Functions

The main aim of the research is to show how the split 2-phase unconventional intersections can minimize and delay and maximize capacity for the same given cycle time when compared to the conventional intersections. The main research goal is obtained by setting two objectives. The first objective is to minimize delay of the proposed unconventional intersection when the volume is low. The second objective is to maximize capacity when the volume is high and congested.

3.2.1 Minimizing Delay

It is commonly agreed that under normal traffic conditions, the primary objective of intersection operation is to minimize delay as a means of improving the level of service. However, when the traffic gets heavily congested, where demand is sustained and delay is excessive, the preference of system operation should be set to maximize the capacity of the intersection so that as many vehicles can clear the intersection as possible to reduce the blockage. Under such context, the following sections discuss the formulation of the system equations and optimization of the two nonlinear systems for delay minimization and capacity maximization, respectively.

The delay experienced by vehicles at an intersection is equal to the difference between the actual travel time and the ideal travel time:

\[ D = C \left[ \sum \left( l_i - \sum \left( \frac{L_i}{U_i} \cdot q_i \right) \right) \right] \]  

\[ \text{Where} \]

\[ D = \text{delay, veh-sec} \]
\( C = \text{total cycle length, sec} \)
\( l_i = \text{queue size for movement } i, \text{ veh} \)
\( L_i = \text{link length for movement } i, \text{ ft} \)
\( U_i = \text{vehicle speed for movement } i, \text{ ft/sec} \)
\( q_i = \text{vehicle arrival rate for movement } i, \text{ veh/sec} \)

Equation (3.1) is the total delay encountered by actual time it takes the vehicle to complete a specific movement based on the number of vehicles accumulated in the queue during one cycle. The queue size can vary for each approach as well as for each movement at that approach. There are four approaches for a four-legged intersection, each with possible through, left turn, and right turn movements. Thus, there are a total of 12 instances to be considered. The queue size developed at any given approach for a certain movement is defined as follows:

\[
    l_i = C \cdot \left( q_i - s_i \cdot \frac{g_i}{C} \right) \quad (3.2)
\]

Where
\( g_i = \text{effective green time for movement } i, \text{ sec} \)
\( s_i = \text{saturation flow rate for movement } i, \text{ veh/sec} \)

The second term in Equation (3.1) represents the time that it would take a vehicle to complete its movement when no signals are encountered. The link length is the total length a vehicle must travel to clear the intersection. Similar to the queue size, the link length can differ between approaches and movements due to the geometry of the intersection. The vehicle speed is also considered to differ, as turning vehicles are
typically traveling at a much slower speed when compared to through vehicles. As a result, the second half of the delay equation is also composed of 12 total terms.

This function is optimized by finding the best combination of variables resulting in the smallest delay. The objective function can be significantly simplified when only the variable terms are taken into account. The link lengths, speeds, and saturation flow rates are intersection specific. For any arrival rate at each intersection approach, the effective green times and the overall cycle length are the key variables to adjust in the optimization process.

3.2.2 Capacity Maximization

In the case of congested flow conditions, a model is needed to maximize the intersection capacity in order to increase the vehicle throughput. The capacity of an intersection is equal to the total number of vehicles that can be accommodated by the individual effective green times:

\[
c = \frac{3600 \cdot \sum \left( \frac{g_i}{\bar{h}} \cdot n_i \right)}{C}
\]  

(3.3)

Where

- \( c \) = capacity, veh/hr
- \( \bar{h} \) = minimum headway, sec/veh
- \( n \) = number of lanes for movement \( i \)

Similar to Equation (3.1), the overall capacity of the intersection is contributed to by all the possible movements at every intersection approach.
3.3 System Constraints

System constraints must be placed on the variables present in the objective function to ensure that a practical result is obtained. These constraints can either be upper or lower bounds or both, and may relate several variables to one another. The constraints used are outlined as follows.

3.3.1 Cycle Time

A practical range of 45 to 150 seconds is considered in the optimization. This range is used to set upper and lower bounds to the value of cycle length.

\[ C \geq 45 \]  
\[ C \leq 150 \]  

3.3.2 Queue Size

The value obtained for the queue size must be a positive number to ensure that no green time is wasted. In addition, the queue cannot be longer than the length of the storage bay. Based on Equation (3.2), these statements result in the following requirements:

\[ q_i \cdot C - s_i \cdot g_i \geq 0 \]  
\[ q_i \cdot C - s_i \cdot g_i \leq \frac{L_i}{s} \]  

Where \( s = \) vehicle spacing, ft

3.3.3 Left Turns

The minimum amount of green time that must be given to left-turning vehicles is prescribed based on the amount of time it will take one vehicle to traverse the intersection
at that location. Left turns at each intersection approach may have a different minimum value.

\[ g_L \geq \frac{L_L}{U_L} \]  

(3.8)

Where

- \( g_L \) = effective green time for left-turning vehicles, sec
- \( L_L \) = turning trajectory length of left-turn, ft
- \( U_L \) = speed of left-turning vehicles, ft/sec

3.3.4 Pedestrians and Right Turns

A sufficient length of green time must be dedicated to the pedestrians in order to ensure safety when crossing the entire intersection by eliminating vehicular conflicts. The amount of time needed can be determined based on the pedestrian walking speed and the total distance to be crossed.

\[ G_{PED} \geq \sum (n_i \cdot w_i) / U_{PED} \]

(3.9)

Where

- \( G_{PED} \) = total green time for pedestrian crossing, sec
- \( w \) = lane width, ft
- \( U_{PED} \) = pedestrian walking speed, ft/sec

With the split 2-PI configuration proposed, a certain amount of green time from a given approach is set aside for the crossing of pedestrians. During this time, left-hand turning movements are restricted from that approach. Therefore, the time dedicated to
pedestrian movements is simply the difference between the through green time and the left turn green time, resulting in:

\[ g_T - g_L \geq \sum \left( \frac{n_i \cdot w_i}{U_{PED}} \right) \]  

(3.10)

Where

\( g_T \) = effective green time for through vehicles, sec

Vehicles turning right from the East-West direction will be able to proceed during the time at which green is given to the North-South left-turning vehicles. Therefore, the green times for each of these movements are set equal. For main intersection notation, the first subscript represents the direction of travel while the second subscript defines the movement made. For example, \( g_{NR} \) represents a northbound right turn.

\[ g_{NR} = g_{WL} \]  

(3.11)

\[ g_{SR} = g_{EL} \]  

(3.12)

\[ g_{ER} = g_{NL} \]  

(3.13)

\[ g_{WR} = g_{SL} \]  

(3.14)

3.3.5 Satellite Signals

The four surrounding satellite signals, which control the traffic leaving the central intersections to join the downstream intersection from each approach, must be included in the optimization model in order for the split 2-PI configuration to work effectively. The signal timing at these secondary intersections must be coordinated with the main intersection in order to allow vehicles to flow throughout the entire network.
uninterrupted. When through movements of the central intersection are given green time, the satellite signal in the downstream direction should also become green after the appropriate offset time, and should remain green for the entire duration of through movements. For satellite intersection notation, the first subscript also represents the direction of travel while the second subscript describes the location of the satellite in question. For example, \( g_{NN} \) represents northbound travel through the northern satellite.

\[
g_{NN} = g_{NT} \quad (3.15)
\]

\[
g_{SS} = g_{ST} \quad (3.16)
\]

\[
g_{EE} = g_{ET} \quad (3.17)
\]

\[
g_{WW} = g_{WT} \quad (3.18)
\]

The offset value at each satellite is based on the distance from the main intersection and vehicle speed.

\[
t_{\text{off},i} = \frac{x_i}{U_i} \quad (3.19)
\]

Where

\( t_{\text{off},i} \) = offset time for satellite location i, sec

\( x_i \) = distance between central intersection and satellite location i, ft

\( U_i \) = vehicle speed from central intersection to satellite location i, ft/sec

When the through movements at such an approach are restricted, the satellite signal will allow left-turning vehicles to crossover and queue in the turning bay while the other approaches are serviced.

\[
g_{ij} = \left( C - g_{jt} - Y - AR - 2d \right) \quad (3.20)
\]
Where

\[ i = \text{direction of travel} \]
\[ j = \text{location of satellite} \]
\[ Y = \text{total intersection yellow time, sec} \]
\[ AR = \text{total intersection all-red time, sec} \]
\[ d = \text{start-up and change-of-phase delay, sec} \]

3.3.6 Residual Queue

The size of the residual queue is equal to the number of vehicles left over from the previous cycle, which must be accommodated during the following cycle. The green times of the next cycle must be adjusted to provide enough time to release the residual queue in addition to newly arriving vehicles. The time necessary for releasing the residual queue is determined based on the timing associated with the preceding cycle and is given as follows:

\[
t_i = \frac{g_i \cdot C - s_i \cdot g_i}{h}
\]

(3.21)

Where

\[ t_i = \text{time to release residual queue for movement } i, \text{ sec} \]

The total green times and cycle length for the following cycle will include the effects of the residual queue as well as the start-up and change-of-phase delay. The cycle length value found will be used as the base for calculating the residual queue in the next cycle.

\[
C = (g_{NS} + t_{NS}) + (g_{EW} + t_{EW}) + Y + AR + 2d
\]

(3.22)
Where

\[\begin{align*}
g_{NS} &= \text{effective green time in next cycle for north or south through movement, sec} \\
t_{NS} &= \text{time to release residual queue for north or south through movement, sec} \\
g_{EW} &= \text{effective green time in next cycle for east or west through movement, sec} \\
t_{EW} &= \text{time to release residual queue for east or west through movement, sec}
\end{align*}\]

All of the constraints outlined above apply to the objective equation for maximizing capacity as well as the function for minimizing delay. Due the nature of the formulations, the application of either objective function requires the solution of a nonlinear system in order to obtain an optimization result.

3.4 Introduction to MATLAB

MATLAB is a GUI programming language in an interactive environment used for various purposes. It is used for algorithm development, data analysis, data visualization, and is also used for performing various numerical calculations. Using MATLAB some of the technical computing problems can run much faster than the traditional programming languages such as C, C++, and FORTRAN. We can input the data or import from different files, other applications into MATLAB for analysis. Once the data is imported into MATLAB, it can be analyzed and visualized using different built-in mathematical and engineering functions, plots, and visualizations available in MATLAB [10]. MATLAB has different add-ons such as signal processing and communication and image processing and optimization tool box which can be used for a range of applications.

For the analysis, Optimization tool box is used, which is an add-on in MATLAB. In Optimization Tool box, constrained non-linear optimization function is used for the
analysis. This function involves the use of both linear and non-linear constraints. All the constraints that are listed above are used in the optimization. Some of the constraints are used as the upper and lower bounds and some of them which involved more than one variable are listed as linear and non linear constraints. Based on the bounds and equations available, a matrix is formed and is used in the optimization tool box. The code written in MATLAB for the optimization is provided in the Appendix at the end of the thesis. Figure 3.5 shows the layout of the optimization tool box.

Figure 3.5: Layout of Optimization Toolbox.
3.5 Introduction to VISSIM

VISSIM is a microscopic simulation program used for multi modal traffic modeling. In VISSIM simulations under different traffic conditions can be run like weather conditions, different volume levels, different speeds, for urban and highway traffic. In simulations, we can also include pedestrian, motorist, and cyclists for different analysis. VISSIM also allows different vehicle inputs like cars, buses, and trams VISSIM uses the car following model and gives the results based on that model.

For the analysis, a traditional and split 2-Phase intersection is built in VISSIM. Since both the intersections are used for comparing the results, the number of lanes in each direction, speed, link length, volume for cars and pedestrians, lane width are maintained same. Figure 3.6 shows the split 2-phase intersection respectively built in VISSIM in analysis.
Figure 3.6: Split 2-phase Intersection in VISSIM.
CHAPTER IV

TESTING AND ANALYSIS

4.1 Preliminary Testing

The goal of the testing is to ensure that the MATLAB programming model can provide physically meaningful output and the results are indeed effective at various arrival rates along each street. The non-linear system is solved numerically by considering all the constraints (discussed in the previous section) to obtain the optimal cycle length and effective green times.

4.2 Effectiveness of Results

The signal timing plans obtained for different levels of traffic flow are implemented in a split 2-PI configuration on the VISSIM simulation platform. The simulation software is able to track the vehicle delay at the intersection based on user input and thus is utilized as a basis for comparison. The results of all cases presented represent the average values found from several trial runs for each simulation.

The results of the optimization are categorized into three cases based on the relationship between arrival rates. The volumes used in the MATLAB programming model and the simulations as well as the cycle times generated from each case are summarized in Table 4.1. Note that the volume levels used in the tests are extremely high, especially for left turns. However, we intentionally selected those values to test the
ability of the new intersection design to handle such traffic conditions.

**TABLE 4.1: Input Values and Results of Preliminary Testing**

<table>
<thead>
<tr>
<th></th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-S Through Volume(veh/hr/ln)</td>
<td>900</td>
<td>1200</td>
<td>900</td>
</tr>
<tr>
<td>E-W Through Volume(veh/hr/ln)</td>
<td>900</td>
<td>900</td>
<td>1400</td>
</tr>
<tr>
<td>Left Turn Volume(veh/hr/ln)</td>
<td>600</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>Right Turn Volume(veh/hr/ln)</td>
<td>600</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>Pedestrian Volume(Ped/hr)</td>
<td>150</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>Cycle Time (sec)</td>
<td>108</td>
<td>112</td>
<td>116</td>
</tr>
</tbody>
</table>

4.2.1 Case 1: Equal Arrival Rates, $q_{NS} = q_{EW}$

The first case specifies that the vehicle arrival rates from both the North-South and East-West directions are equal, as shown in Table 4.1. In addition, the pedestrian arrival rates from either direction are also kept the same. This scenario yielded a cycle time of 108 seconds and an equal green split given to each of the two directions. Figure 4.1 shows the results of numerous simulations when the cycle times and green splits were varied using VISSIM software. This was done to ensure that the model was providing a result that minimized the delay for the given arrival rates.

Figure 4.1 shows that the delay will be minimized at a 50/50 green split, and the delay increases when the split is varied for any cycle length. When the cycle length is decreased below the optimum value, it was observed that the maximum queue sizes given by Equation 3.7 were surpassed during the simulation.
4.2.2 Case 2: Unequal Arrival Rates, $q_{NS} > q_{EW}$

The second case considers that the vehicle arrival rate from the North-South direction exceeds that of the East-West direction by over 33% as given in Table 4.1. For this case, the pedestrian arrival rates and walking speeds were again kept equal in each direction. Figure 4.2 shows that for this scenario, the minimum delay will be experienced at a 60/40 green split for a cycle length of 112 seconds.

The result is generally in keeping with the VISSIM results when cycle time is considered. A cycle time of 112 seconds gives a smaller delay in all cases than a cycle length of 132 seconds. This is because a cycle length longer than the optimized result will provide an excessive amount of green time to either direction, which subsequently means that queued vehicles will have to wait longer to receive green time, resulting in a larger delay.
4.2.3 Case 3: Unequal Arrival Rates, $q_{NS} < q_{EW}$

The final case shows that when the arrival rate from the North-South direction is less than that of the East-West direction by about 36%, the results changed accordingly. This case also considered equal pedestrian arrival rates and walking speeds in each direction. The cycle time obtained was 116 seconds as shown in Table 4.1. Figure 4.3 shows that a cycle length of 116 seconds will yield a smaller delay than 136 seconds. The vehicle arrival rates used were not high enough to warrant a cycle of such length, which causes the vehicles to wait longer for their next green phase.

The results obtained from each of these trials are consistent with what is expected based on engineering judgment. This shows that the optimization model recognizes that the green time must be split between the two directions based on which approaches have the highest demand.
4.3 Comparison with Conventional Intersection Control

Further testing was done to compare the amount of improvement achieved under both normal and congested flow conditions. Several points of generally intermediate volumes were considered to see how the delay was decreased due to implementation of the model. In addition, points of congested volume levels were chosen to show how well the capacity was increased through system optimization.

4.3.1 Delay Minimization

Table 4.2 summarizes the volumes used and the results obtained from each of the trials regarding delay minimization. We reduced the traffic demand in the test in order to avoid excessive queue length in the conventional type of control and obtain meaningful results for comparison. Figure 4.4 provides a comparison of the total vehicle delay encountered at a conventional four-legged intersection and split 2-PI based on VISSIM simulations of each configuration. At each volume level, separate analysis was conducted.
to generate the best signal timing plan for both types of control and results were implemented in VISSIM simulation.

As the figure shows, the delay from the split 2-PI control is noticeably smaller than that experienced at a conventional intersection. At the lowest tested through volume of 350 veh/hr/ln, the delay reduced is over 13%. As the volume is increased to 450 veh/hr/ln, the reduction in delay gradually increases to nearly 26%. This result is obtained because the cycle time is governed by the minimum pedestrian crossing time from Equation 3.9 for the smallest volume. For higher volumes, the green time needed to service the vehicles is already large enough to provide sufficient time for pedestrian crossing. In this case, the cycle time also satisfies all other constraints, resulting in a larger reduction in delay. No further testing was done beyond a volume of 550 veh/hr/ln because the cycle time is to be kept within practical limits, and the maximum of 150 seconds occurs at this flow rate.

**TABLE 4.2: Input Values and Results of Delay Minimization Trials**

<table>
<thead>
<tr>
<th></th>
<th>Typical</th>
<th>Split 2-PI</th>
<th>Typical</th>
<th>Split 2-PI</th>
<th>Typical</th>
<th>Split 2-PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Through Volume (veh/hr/ln)</td>
<td>350</td>
<td>400</td>
<td>450</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Turn Volume (veh/hr/ln)</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right Turn Volume (veh/hr/ln)</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pedestrian Volume (Ped/hr)</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle Time (sec)</td>
<td>82.5</td>
<td>126.38</td>
<td>150</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Throughput (veh/hr)</td>
<td>0.58</td>
<td>0.5</td>
<td>1.45</td>
<td>1.12</td>
<td>2.19</td>
<td>1.63</td>
</tr>
<tr>
<td>Decrease in Delay</td>
<td>13.29%</td>
<td>22.37%</td>
<td>25.71%</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 4.4: Comparison of Results from Delay Minimization Trails

4.3.2 Capacity Maximization

Table 4.3 summarizes the volumes used and the results obtained from each of the trials regarding capacity maximization. Figure 4.5 provides a comparison of the intersection throughput at a conventional four-legged intersection versus split 2-PI. The timing plan used in the simulation for the conventional control was again found through the signal optimization technique. As the figure shows, the intersection throughput measured at each congested volume level for the split 2-PI control is consistently greater than that obtained from the conventional control. We also noticed, however, that as the volume level is increased, the difference in throughput between the two types of control starts to reduce. At a volume of 500 veh/hr/ln, the throughput increase has reached almost 28% while less than a 5% increase results at a volume of 600 veh/hr/ln. This result is found because although the through volumes were increased with each trial, the left and right turn volumes were held constant and the ratio of left turn green time to the through
movement green time was decreased. In conventional control, the green time for the left turns is also assigned according to demand proportionality. Thus, as the through movement volume was increased in the simulation, the advantage of the displaced left turn and the effect on throughput is decreased.

TABLE 4.3: Input Values and Results of Capacity Maximization Trials

<table>
<thead>
<tr>
<th></th>
<th>Typical</th>
<th>Split 2-PI</th>
<th>Typical</th>
<th>Split 2-PI</th>
<th>Typical</th>
<th>Split 2-PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Through Volume(veh/hr/ln)</td>
<td>500</td>
<td>550</td>
<td>600</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Turn Volume(veh/hr/ln)</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right Turn Volume(veh/hr/ln)</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pedestrian Volume(Ped/hr)</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle Time (sec)</td>
<td>82.5</td>
<td>90.91</td>
<td>116.17</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Throughput (veh/hr)</td>
<td>2357</td>
<td>3011</td>
<td>2416</td>
<td>2851</td>
<td>2758</td>
<td>2882</td>
</tr>
<tr>
<td>Increase in Throughput</td>
<td>27.75%</td>
<td>18.00%</td>
<td>4.50%</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.5: Comparison of Results from Capacity Maximization Trials
CHAPTER V

CONCLUSIONS AND FUTURE WORK

In this study, a geometric improvement to a typical 2-PI is proposed in order to enhance pedestrian safety and comfort at this complex intersection. This altered design is referred to as a split 2-PI, and the objective of this paper was to maximize the operational benefits of this unconventional intersection design through system optimization techniques. By developing and applying the MATLAB non-linear optimization model to the proposed split 2-PI configuration, the appropriate signal timing has been found to enhance the benefits of unconventional geometric designs by maximizing intersection throughput and minimizing vehicle delay. Preliminary tests found that the optimization method used is very effective in finding the appropriate cycle time for yielding minimal intersection delay based on arrival rates. When employing optimization results in simulations of both conventional and split 2-PI controls, the unconventional geometry consistently performs better. In both normal traffic and congested flow conditions, the vehicle throughput was found to increase while delay was also reduced.

The next step in the research is to explore the benefits of lane reduction in split 2-phase intersection. In this lane reduction process the total number of lanes in each leg will be reduced 2 lanes thus reducing the number of lanes pedestrians cross and also the green time. This type of geometry would be ideal for locations with lower volumes of 12
turning movements. This involves allowing both right- and left-turning vehicles to utilize the same lanes as those used by through vehicles to exit the intersection.

Future work surrounding the optimization of the split 2-PI network will involve using an approach-based method to analyze the system. With this technique, each approach will be treated separately so that the cycle time may be further split up in the most efficient manner possible based on the demand encountered. Although unconventional geometric intersections require a substantial amount of right of way, the feasible locations of installations have the potential to greatly benefit from its unique design characteristics.
REFERENCES


APPENDIX

MATLAB CODE

function f = DelayFxn(x)
    %%%Input Assumed Values
    %Saturation Flow Rate, veh/sec
    SNT=0.33;
    SNR=0.85*SNT;
    SNL=0.85*SNT;
    SST=0.33;
    SSR=0.85*SST;
    SSL=0.85*SST;
    SET=0.33;
    SER=0.85*SET;
    SEL=0.85*SET;
    SWT=0.33;
    SWR=0.85*SWT;
    SWL=0.85*SWT;
    %Arrival Rate, veh/sec
    qNT=200/3600;
    qNR=0.40*qNT;
    qNL=0.40*qNT;
    qST=200/3600;
    qSR=0.40*qST;
    qSL=0.40*qST;
    qET=200/3600;
    qER=0.40*qET;
    qEL=0.40*qET;
    qWT=200/3600;
    qWR=0.40*qWT;
    qWL=0.40*qWT;
    %Link Length, ft
    LNT=580;
    LNR=270;
    LNL=380;
    LST=580;
    LSR=270;
LSL=380; LET=400; LER=250; LEL=380; LWT=400; LWR=250; LWL=380;

%Approach Speed, ft/sec
UNT=55; UNR=40; UNL=40; UST=55; USR=40; USL=40; UET=55; UER=40; UEL=40; UWT=55; UWR=40; UWL=40;

%%Calculate 13 Constants Associated with 13 Variables
a1=qNT+qNR+qNL+qST+qSR+qSL+qET+qER+qEL+qWT+qWR+qWL;
a2=SNT;
a3=SNR;
a4=SNL;
a5=SST;
a6=SSR;
a7=SSL;
a8=SET;
a9=SER;
a10=SEL;
a11=SWT;
a12=SWR;
a13=SWL;
a14=(LNT*qNT/UNT)+(LNR*qNR/UNR)+(LNL*qNL/UNL)+(LST*qST/UST)+(LSR*

%Objective Function
f = ((a1*(x(1)*x(1)))-(a2*x(1)*x(2))-(a3*x(1)*x(3))-(a4*x(1)*x(4))-(a5*x(1)*x(5))-(a6*

%%Input Assumed Values

return

%%Input Assumed Values
%Saturation Flow Rate, veh/sec
SNT=0.33;
SNR=0.85*SNT;
SNL=0.85*SNT;
SST=0.33;
SSR=0.85*SST;
SSL=0.85*SST;
SET=0.33;
SFR=0.85*SET;
SFL=0.85*SET;
SWT=0.33;
SWR=0.85*SWT;
SWL=0.85*SWT;

%Arrival Rate, veh/sec
qNT=200/3600;
qNR=0.40*qNT;
qNL=0.40*qNT;
qST=200/3600;
qSR=0.40*qST;
qSL=0.40*qST;
qET=200/3600;
qER=0.40*qET;
qEL=0.40*qET;
qWT=200/3600;
qWR=0.40*qWT;
qWL=0.40*qWT;
q = [qNT; qNR; qNL; qST; qSR; qSL; qET; qER; qEL; qWT; qWR; qWL];

%Link Length, ft
LNT=580;
LNR=270;
LNL=380;
LST=580;
LSR=270;
LSL=380;
LET=400;
LER=250;
LEL=380;
LWT=400;
LWR=250;
LWL=380;

%Approach Speed, ft/sec
UNT=55;
UNR=40;
UNL=40;
UST=55;
USR=40;
USL=40;
UET=55;
UER=40;
UEL=40;
UWT=55;
UWR=40;
UWL=40;

% Number of Lanes, unitless
nNT=2;
nNL=1;
nNR=1;
nST=2;
nSL=1;
nSR=1;
nET=2;
nEL=1;
nER=1;
nWT=2;
nWL=1;
nWR=1;

% Minimum Headway for Different Turning Movements, veh/sec
hNT=2.5;
hNR=2.5;
hNL=2.5;
hST=2.5;
hSR=2.5;
hSL=2.5;
hET=2.5;
hER=2.5;
hEL=2.5;
hWT=2.5;
hWR=2.5;
hWL=2.5;

% Miscellaneous
\[ d=4.5; \] % Start-Up and Change-of-Phase Delay, sec
\[ s=35; \] % Average Vehicle Spacing, ft
\[ \text{UPED}=3.5; \] % Pedestrian Walking Speed, ft/sec
\[ w=12; \] % Lane Width, ft

% Lower Bounds
lb=zeros(13,1);
lb(1)=45;
lb(2)=1;
\begin{verbatim}
\text{lb}(3) = 1;
\text{lb}(4) = 130 / \text{UNL};
\text{lb}(5) = 1;
\text{lb}(6) = 1;
\text{lb}(7) = 130 / \text{USL};
\text{lb}(8) = 1;
\text{lb}(9) = 1;
\text{lb}(10) = 130 / \text{UEL};
\text{lb}(11) = 1;
\text{lb}(12) = 1;
\text{lb}(13) = 130 / \text{UWL};

\%\% \text{Upper Bounds}
\text{ub} = \text{Inf}(13, 1);
\text{ub}(1) = 150;

\%\% \text{Linear Inequalities}
\text{A} = \text{zeros}(28, 13);
\text{A}(1, [2, 1]) = [\text{SNT}, -\text{qNT}]; \text{b}(1) = 0; \% \text{Inequalities for minimum queue size}
\text{A}(2, [3, 1]) = [\text{SNR}, -\text{qNR}]; \text{b}(2) = 0;
\text{A}(3, [4, 1]) = [\text{SNL}, -\text{qNL}]; \text{b}(3) = 0;
\text{A}(4, [5, 1]) = [\text{SST}, -\text{qST}]; \text{b}(4) = 0;
\text{A}(5, [6, 1]) = [\text{SSR}, -\text{qSR}]; \text{b}(5) = 0;
\text{A}(6, [7, 1]) = [\text{SSL}, -\text{qSL}]; \text{b}(6) = 0;
\text{A}(7, [8, 1]) = [\text{SET}, -\text{qET}]; \text{b}(7) = 0;
\text{A}(8, [9, 1]) = [\text{SER}, -\text{qER}]; \text{b}(8) = 0;
\text{A}(9, [10, 1]) = [\text{SEL}, -\text{qEL}]; \text{b}(9) = 0;
\text{A}(10, [11, 1]) = [\text{SWT}, -\text{qWT}]; \text{b}(10) = 0;
\text{A}(11, [12, 1]) = [\text{SWR}, -\text{qWR}]; \text{b}(11) = 0;
\text{A}(12, [13, 1]) = [\text{SWL}, -\text{qWL}]; \text{b}(12) = 0;
\text{A}(13, [2, 1]) = [-\text{SNT}, \text{qNT}]; \text{b}(13) = \text{LNT}/s; \% \text{Inequalities for maximum queue size}
\text{A}(14, [3, 1]) = [-\text{SNR}, \text{qNR}]; \text{b}(14) = \text{LNR}/s;
\text{A}(15, [4, 1]) = [-\text{SNL}, \text{qNL}]; \text{b}(15) = \text{LNL}/s;
\text{A}(16, [5, 1]) = [-\text{SST}, \text{qST}]; \text{b}(16) = \text{LST}/s;
\text{A}(17, [6, 1]) = [-\text{SSR}, \text{qSR}]; \text{b}(17) = \text{LSR}/s;
\text{A}(18, [7, 1]) = [-\text{SSL}, \text{qSL}]; \text{b}(18) = \text{LSL}/s;
\text{A}(19, [8, 1]) = [-\text{SET}, \text{qET}]; \text{b}(19) = \text{LET}/s;
\text{A}(20, [9, 1]) = [-\text{SER}, \text{qER}]; \text{b}(20) = \text{LER}/s;
\text{A}(21, [10, 1]) = [-\text{SEL}, \text{qEL}]; \text{b}(21) = \text{LEL}/s;
\text{A}(22, [11, 1]) = [-\text{SWT}, \text{qWT}]; \text{b}(22) = \text{LWT}/s;
\text{A}(23, [12, 1]) = [-\text{SWR}, \text{qWR}]; \text{b}(23) = \text{LWR}/s;
\text{A}(24, [13, 1]) = [-\text{SWL}, \text{qWL}]; \text{b}(24) = \text{LWL}/s;
\text{A}(25, [4, 2]) = [1, -1]; \text{b}(25) = -15; \% \% \text{Inequalities for min pedestrian walking times}
\text{A}(26, [7, 5]) = [1, -1]; \text{b}(26) = -15;
\text{A}(27, [10, 8]) = [1, -1]; \text{b}(27) = -15;
\text{A}(28, [13, 11]) = [1, -1]; \text{b}(28) = -15;
\end{verbatim}
%%Linear Equalities
Aeq=zeros(8,13); beq=zeros(1,8);
Aeq(1,[4,9])=[1,-1];
Aeq(2,[12,7])=[1,-1];
Aeq(3,[10,6])=[1,-1];
Aeq(4,[13,3])=[1,-1];
Aeq(5,[1,2,8])=[1,-1,-1]; beq(5)=2*d+12;
Aeq(6,[1,2,11])=[1,-1,-1]; beq(6)=2*d+12;
Aeq(7,[1,5,8])=[1,-1,-1]; beq(7)=2*d+12;
Aeq(8,[1,5,11])=[1,-1,-1]; beq(8)=2*d+12;
x0=[45; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0];%Start points
options = optimset('Algorithm','active-set');
[x,fval] = fmincon(@DelayFxn,x0,A,b,Aeq,beq,lb,ub,[],options);
%%Calculate Capacity
a1=3600;
a2=nNT/hNT;
a3=nNR/hNR;
a4=nNL/hNL;
a5=nST/hST;
a6=nSR/hSR;
a7=nSL/hSL;
a8=nET/hET;
a9=nER/hER;
a10=nEL/hEL;
a11=nWT/hWT;
a12=nWR/hWR;
a13=nWL/hWL;
c =
(a1/x(1))*(a2*x(2)+a3*x(3)+a4*x(4)+a5*x(5)+a6*x(6)+a7*x(7)+a8*x(8)+a9*x(9)+a10*
x(10)+a11*x(11)+a12*x(12)+a13*x(13));