A NOVEL APPROACH TO DILEMMA ZONE PROBLEM
FOR HIGH SPEED SIGNALIZED INTERSECTIONS

A Thesis
Presented to
The Graduate Faculty of The University of Akron

In Partial Fulfillment
of the Requirements for the Degree
Master of Science

Venkata Suresh Raavi
May, 2010
A NOVEL APPROACH TO DILEMMA ZONE PROBLEM
FOR HIGH SPEED SIGNALIZED INTERSECTIONS

Venkata Suresh Raavi

Thesis

Approved:

Advisor
Dr. Ping Yi

Committee Member
Dr. Anil Patnaik

Committee Member
Dr. William H. Schneider IV

Accepted:

Department Chair
Dr. Wieslaw K. Binienda

Dean of the College
Dr. George K. Haritos

Dean of the Graduate School
Dr. George R. Newkome

Date

ii
ABSTRACT

Safety and efficiency are both major concern at high-speed intersections. It has been a major challenge for Traffic Engineers over the years to increase efficiency without sacrificing delay. An IntelliDrive signal control system was developed in the University of Akron’s Transportation lab based on the concept of IntelliDrive technology that can improve safety significantly while retaining efficiency. The new intelligent signal control system can address the dynamic dilemma zone problem, thus providing maximum safety at intersections. Every vehicle in the system is addressed to its own dilemma boundary rather than addressing dilemma zone for the whole intersection approach.

By obtaining advance speed information, vehicle classification, vehicle location from the stop line, for every vehicle for every second in the system in real time, unneeded extensions can be avoided. Thus the intersection not only operates more safely, but also more efficiently because of the proposed algorithm logic that utilizes IntelliDrive technology. IntelliDrive test bed was created in VISSIM simulation platform and tested for operational efficiency when safety criteria are met.

This thesis also discusses the characteristics of the Peak Flow Factor (PFF) which is the ratio between peak hourly volume and peak flow rate within the hour. Despite the fact that the Highway Capacity Manual (HCM) requires use of this ratio in evaluating the Levels of Service (LOS) of roadway facilities, a very limited amount of prior research has been made on its variability. Subsequently, the HCM established a time interval of 15
minutes for obtaining the peak flow rate without much explanation on why this time interval was chosen.

In order to investigate the PFF’s effectiveness for representing the variation of flow within peak hour, this research focuses on PFF analysis and time series models of the PFF at various time intervals for signalized isolated intersections.

Results showed that the PFF decreased when time interval decreased and that significant differences in the stability were apparent when each time interval were compared. In addition, the developed time series models showed the the PFF can be forecasted with relative accuracy and simulation models within this study demonstrated the test results from data are dependable.
ACKNOWLEDGEMENTS

First and foremost, I would like to express my sincere appreciation to my advisor, Professor Ping Yi, for his valuable guidance, continuous support and encouragement throughout this study and throughout my time so far in the United States.

Acknowledgements are also extended to my committee members, Dr. William H. Schneider, Dr. Anil Patnaik for reviewing my work and helpful recommendations.

Special thanks are given to my fellow graduate students, Chun Shao, Jialei Mao, Cong Feng, Baoji Wang, and Silin Ding for their useful discussions related to the topic of Dilemma Zone and their support in VISSIM software usage and programming. The sincere friendship and support from them always gave me energy and impetus to finish this thesis.

My deepest gratitude goes to my family and friends who provides love and support more than I could ever expect.
TABLE OF CONTENTS

LIST OF TABLES ........................................................................................................ viii
LIST OF FIGURES ....................................................................................................... ix

CHAPTER

I. INTRODUCTION ...................................................................................................... 1
   1.1 Statement of Problem ..................................................................................... 3
   1.2 Motivation and General Description ............................................................. 4
   1.3 Research Objectives and Methodology ......................................................... 5
   1.4 Organization of the Thesis ........................................................................... 7

II. LITERATURE REVIEW ........................................................................................... 8
   2.1 Dilemma Zone ............................................................................................... 8
   2.2 Traffic Actuated Control at High Speed Intersections ................................. 10
   2.3 Existing Dilemma Zone Protection Strategies ............................................. 16
   2.4 Concluding Remarks ................................................................................... 19

III. PROPOSED INTELLIDRIVE SIGNAL CONTROL SYSTEM ............................. 21
   3.1 Introduction to IntelliDrive Technology ...................................................... 21
   3.2 Proposed Algorithm ..................................................................................... 22
   3.3 Summary of the New IntelliDrive Signal Control System .......................... 27

IV. COMPUTER MODELING AND IMPLEMENTATION ........................................ 29
4.1 History and Background of Traffic Simulation.................................29
4.2 Introduction of VISSIM (Urban Traffic Simulator).........................30
4.3 VISSIM-Based Microscopic Traffic Simulation Model.......................33
4.4 Data Analysis.......................................................................................39
4.5 Concluding Remarks............................................................................43

V. PEAK FLOW FACTOR CHARACTERISTICS AT INTERSECTIONS.......47
5.1 Introduction.........................................................................................47
5.2 Literature Review................................................................................49
5.3 Data Collection....................................................................................51
5.4 PFF Variations....................................................................................52
5.5 Time Series Models............................................................................57

VI. CONCLUSIONS.....................................................................................60

BIBLIOGRAPHY..........................................................................................63
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Features of the Different Dilemma Zone Protection Strategies</td>
</tr>
<tr>
<td>4.1</td>
<td>Average Percentage of Max-outs for 10 Simulation Runs</td>
</tr>
<tr>
<td>4.2</td>
<td>Average Total Vehicle Delays for 10 Simulation Runs</td>
</tr>
<tr>
<td>5.1</td>
<td>Illustrating Method of Time Lagging for the Last 5 minutes for E Exchange and S Broadway Intersection</td>
</tr>
<tr>
<td>5.2</td>
<td>Mean PFFs at a 10-second Time Lag Interval</td>
</tr>
<tr>
<td>5.3</td>
<td>Standard Deviation of PFFs at a 10-second Time Lag Interval</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>1.1</td>
<td>Illustration of dilemma zone at an intersection</td>
</tr>
<tr>
<td>2.1</td>
<td>Dilemma zone showing stop zone and go zone boundaries</td>
</tr>
<tr>
<td>2.2</td>
<td>Traffic signalization hardware</td>
</tr>
<tr>
<td>2.3</td>
<td>Bierele-recommended detector configuration for 50 mph design speed</td>
</tr>
<tr>
<td>2.4</td>
<td>SDITE-recommended detector configuration for 50 mph design speed</td>
</tr>
<tr>
<td>2.5</td>
<td>Bonneson-recommended detector configuration for rural intersections with</td>
</tr>
<tr>
<td></td>
<td>50 mph design speed</td>
</tr>
<tr>
<td>3.1</td>
<td>Algorithm showing the proposed IntelliDrive Signal Control System</td>
</tr>
<tr>
<td>3.2</td>
<td>Overview of the UA Intelligent Signal Control System</td>
</tr>
<tr>
<td>4.1</td>
<td>Traffic composition</td>
</tr>
<tr>
<td>4.2</td>
<td>Study area of the IntelliDrive Signal Control System</td>
</tr>
<tr>
<td>4.3</td>
<td>Desired speed distributions</td>
</tr>
<tr>
<td>4.4</td>
<td>Driver behavior parameters: Car following</td>
</tr>
<tr>
<td>4.5</td>
<td>Driver behavior parameters: Lane change</td>
</tr>
<tr>
<td>4.6</td>
<td>Driver behavior parameters: Lateral distance</td>
</tr>
<tr>
<td>4.7</td>
<td>Driver behavior parameters: Signal control</td>
</tr>
<tr>
<td>4.8</td>
<td>Link types</td>
</tr>
<tr>
<td>4.9</td>
<td>Simulation parameters</td>
</tr>
<tr>
<td>Topic</td>
<td>Page</td>
</tr>
<tr>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Performance evaluation</td>
<td>39</td>
</tr>
<tr>
<td>Average vehicle delay for minor road volume of 50 vphpl</td>
<td>43</td>
</tr>
<tr>
<td>Average vehicle delay for minor road volume of 100 vphpl</td>
<td>44</td>
</tr>
<tr>
<td>Average vehicle delay for minor road volume of 150 vphpl</td>
<td>44</td>
</tr>
<tr>
<td>Percentage of max-out occurrence for minor road volume of 50 vphpl</td>
<td>45</td>
</tr>
<tr>
<td>Percentage of max-out occurrence for minor road volume of 100 vphpl</td>
<td>45</td>
</tr>
<tr>
<td>Percentage of max-out occurrence for minor road volume of 150 vphpl</td>
<td>46</td>
</tr>
<tr>
<td>Variations in traffic counts at E Exchange St and S Broadway St</td>
<td>53</td>
</tr>
<tr>
<td>Variations in traffic counts at E Exchange St and Spicer St</td>
<td>53</td>
</tr>
<tr>
<td>Variations in traffic counts at Lovers Ln and Hummel Ave</td>
<td>54</td>
</tr>
<tr>
<td>Variations in traffic counts at Union St and Perkin St</td>
<td>54</td>
</tr>
<tr>
<td>Charts illustrating comparison between original PFF and forecast PFF</td>
<td>58</td>
</tr>
</tbody>
</table>
CHAPTER I

INTRODUCTION

As vehicles approach an intersection and the signal enters its change interval and displays the yellow a decision must be made whether to proceed through the intersection or whether to stop. This decision has serious safety consequences as an abrupt stop may cause a rear-end accident while proceeding through the intersection may lead to a right-angle collision. At high speed intersections, defined as intersections where the approach speeds exceed 40 mph, this problem is of great concern.

At a certain range of distances from the stop line, which depend upon the speed of the approaching vehicles, the driver may be undecided whether to accelerate and cross the intersection or whether to brake sharply and stop the vehicle. This area of indecision is known as the ‘dilemma zone’. Some researchers, using empirical data (1) have arbitrarily defined the dilemma zone as the area in which at one end 90% of the drivers will stop their vehicles and at the other end 90% of the drivers will accelerate their vehicles.
A dilemma zone occurs for the situation in which $X_s > X_c$ and the areas overlap as illustrated in Figure 1.1. Several strategies have been developed to alleviate the dilemma zone problem although the more complex methods involve large area detection and will be perused here. The simplest method involves the placement of the vehicle detectors in advance of the stop line. By placing the detector at the far end of the dilemma zone (distance $X_s$) and setting the vehicle interval extension equal to the passage time required for the for the vehicle to travel from the detector to the stop line, the driver will have assured green time and the signal will change to yellow as the stop line is crossed. Vehicles traveling faster than the average speed will enter the intersection sooner than required and pose no problem. Vehicle traveling slower than the average speed will experience the yellow signal before they reach the stop line, however, it is quite likely that they will have reached the end of the dilemma zone (distance $X_c$) and will be able to safely accelerate into the intersection.
For higher speed intersections and at intersections using advanced volume-density controllers with gap reduction capabilities a series of detectors are placed at increasing distances from the stop line. Several methods for calculating the proper detector spacing are detailed in the Traffic Detector Handbook.

1.1 Statement of Problem

Traffic signals at high-speed intersections, when designed appropriately, can increase safety of the intersection by reducing the size and frequency of the dilemma zone. For this reason, a great deal of effort has been made by the researchers and implementers to study the characteristics of dilemma zone. In 1998, nearly 43 percent of all crashes were at intersections or intersection-related. In addition, roughly 73 percent of fatal crashes occurred where posted speed limits were 45 mph or greater (2).

According to Federal Highway Administration (FHWA), in the year 2000:

- 2.8 million crashes occurred at intersections (44% of all reported crashes)
- 8,500 fatalities were at intersections (23% of total fatalities)
- One million crashes with injuries were reported at intersections

Red-light running, which results in roughly 950 deaths and 90,000 injuries a year, is estimated to be the cause in 92,000 annual crashes. These statistics indicate that research on this subject must be continued.

Signalized intersections with approach speeds greater than 45 mph are a major concern because of the higher speeds. At these intersections, drivers are more likely to face a dilemma zone at the onset of the yellow light, where a decision to stop may result
in a rear-end collision while continuing through the intersection may produce a right-angle accident. (3), (4)

The dilemma zone, poses a high accident potential for the driver in stopping safely during the yellow interval or in proceeding through the intersection before the beginning of red. Generally, the location of the driver on the intersection approach and the speed of the vehicle influence the driver’s decision to stop or proceed when he sees the green signal changing to yellow. The clearing distance is the distance the vehicle travels between the times the signal changes to yellow to the time the signal changes to red. The stopping distance is the distance traveled by the vehicle between the times the signal changes to yellow to the time when the vehicle actually comes to rest. If the stopping distance is greater than the clearing distance, and the vehicle is placed in between them, a dilemma zone is formed. In this situation, neither the distance to the intersection is adequate for stopping nor is the signal interval adequate for clearing the intersection. The driver is in a potentially hazardous situation whereby, if he tries to cross the intersection at the onset of red interval, he may end up in an angle accident with the cross street traffic or if he accelerates through yellow, he may end up in a rear-end collision. The uncertain situation in a dilemma zone can potentially lead to rear-end or right angle collisions.

1.2 Motivation and General Description

The purpose of traffic control is to assign the right of way to drivers, and thereby facilitate roadway safety by ensuring the orderly movement of traffic. If not designed correctly, intersections that require signal control can adversely impact the safety of the
public and the efficiency of the intersection. Traffic signals, when properly designed and effectively operated, can increase safety by reducing the frequency of certain types of accidents and improve efficiency by minimizing system delays and increasing capacity.

At high speed intersections, drivers are more likely to face a dilemma zone where the yellow light is displayed. In order to minimize dilemma zone incidents at high-speed, isolated intersections, current signals make use of advance detectors and traffic actuated controllers. These dynamically respond to varying traffic demands and improve the safety of the intersection by detecting an approaching vehicle and extending the green until the driver safely clears the intersection.

However, some drivers can still face a dilemma zone situation since the green cannot prolonged endlessly. If the green is extended by heavy traffic to the maximum interval, dilemma zone protection is no longer provided. Furthermore, more detectors in use today consist of wire loops permanently placed under the surface of the pavement with their location having been determined by the design speed. Since speeds vary depending on motorists and driving conditions, the location and size of the dilemma zone is impacted. A driver whose speed deviates from the design speed may not be detected in a timely manner. Therefore, dilemma zone protection is not ensured. Hence this research tries to provide a solution to the possible extent to minimize dilemma zone hazard at signalized intersections.

1.3 Research Objectives and Methodology

In order to minimize dilemma zone at high-speed intersections, current signal designs make use of advance detectors and traffic-actuated controllers. Actuated control
dynamically responds to varying traffic demands and improves the safety of the intersection by extending the green until driver is clear of dilemma zone.

Despite the use of green extensions, dilemma zone still exists due to different approaching speeds selected by drivers who are limited in terms of experience, attentiveness, physical and mental skills, etc. In addition, dilemma zones are affected by varying driving conditions. For example, if the average approaching speed to an intersection increases or if the roadway friction decreases due to weather or other environmental factors, the dilemma zone tends to move further upstream. On the other hand, the dilemma zone generally moves closer to the intersection if vehicles are arriving at a slower speed. In either case, the length of the dilemma zone will be changed from the initial design. Currently existing dilemma zone protection strategies cannot address this dynamic dilemma zone issue. This research include the dynamic dilemma zone by considering each and every vehicle’s dilemma zone separately, thus controlling the signals and minimizing number of vehicles that can be caught in a dilemma on the onset of the yellow light. The main objectives of this research are:

1. To minimize the accident potential due to dilemma zone at high speed intersections, thus to improve the safety and

2. To increase efficiency of the same

The methodology used in this research is based on IntelliDrive technology, which can provide the speed of the vehicle, position of the vehicle and length of the vehicle at any point in the network for every second. Based on this information, an algorithm was developed and tested which drastically reduced the effect of dilemma zone, thus providing safety without sacrificing efficiency or sometimes even increasing efficiency.
1.4 Organization of the Thesis

In Chapter II, a literature review was conducted to explore different dilemma zone protection strategies which are in use so far. Prior to that, concepts of dilemma zone are discussed to understand the problem involved. The functioning of an actuated signal control is explained to understand the later part of the thesis. Also, a literature review on these various detector configurations was also conducted and a comparison is made between them.

In chapter III, the IntelliDrive technology was introduced and the newly proposed IntelliDrive signal control system was discussed with its features. Summary of the IntelliDrive signal control system comparing with dilemma zone protection strategies was given to clearly understand the concept of the newly introduced system.

Computer modeling and implementation was conducted in Chapter IV. In the first section, history and back ground of traffic simulation was presented to better support this research. VISSIM microscopic simulation model was used to build a model to implement and test the developed algorithm, a discussion was made on VISSIM traffic simulation software and the methodology adopted in building the VISSIM model for this research. Results were discussed and conclusions were drawn.

Finally, Chapter V presents the conclusion of the thesis and recommendations for further research.
2.1 Dilemma Zone

A driver approaching a signalized intersection as the light turns yellow will either have to stop or proceed through the intersection. If the driver decided to stop, the distance required to safely stop before entering the intersection is known as the stop zone and is defined as

$$\text{StopZone} = vt + \frac{v^2}{2g(f \pm G)}$$

(2.1)

Where: $v =$ vehicle approach speed (ft/sec)

t = driver perception reaction time (sec)

g = acceleration of gravity (ft/sec$^2$)

$f =$ coefficient of friction

$G =$ roadway gradient ($\%$/100)

As can be seen from equation 2.1, a higher approach speed will result in a larger stop zone.
Figure 2.1. Dilemma zone showing stop zone and go zone boundaries.

From the onset of the yellow interval, the distance a vehicle must travel with no change in speed and be able to safely proceed through the intersection is known as the go zone and is formulated as follows:

\[
\text{Go Zone} = yv - (W + L) \tag{2.2}
\]

Where:  
\( y \) = yellow interval (sec)  
\( V \) = vehicle approach speed (ft/sec)  
\( W \) = intersection width (ft)  
\( L \) = length of vehicle (ft)

The intersection width should be measured from the near side of the stop line to the far side edge of the conflicting approach (Figure 2.1). The dilemma zone, which is the segment of road on approach to an intersection where a vehicle’s location within it creates indecision for drivers with regard to stopping or proceeding, is given by:

\[
\text{Dilemma Zone} = \text{Stop Zone} - \text{Go Zone} \tag{2.3}
\]
Therefore, if the stop zone is greater than the go zone at the onset of the yellow interval, a dilemma zone situation exists as shown in figure 2.1 and the driver can neither safely stop nor clear the intersection. However, if the stop zone is less than go zone, no dilemma zone is encountered and the driver has the option to stop or proceed through intersection.

The objective is to eliminate or minimize dilemma zone. Current practice at all signalized intersections involves the timing of the phase change interval to minimize the dilemma zone. By equating the stop zone to the go zone, the required phase change interval can be obtained as:

$$Y + AR = t + \frac{v}{30(f + G)} + \frac{W + L}{v}$$

(2.4)

In many cases, equation 2.4 cannot be satisfied since driver perception reaction time, vehicle approach speed, coefficient of friction and vehicle length are all treated as constants when phase change interval is defined during design. Consequently, dilemma zone will be encountered at many intersections, especially high speed ones.

2.2 Traffic Actuated Control at High Speed Intersections

Depending on location, the type of traffic which they are designed to serve, and the type of controller that is their operation, traffic signals can be pre-timed, semi-actuated or fully actuated. For isolated intersections at which large fluctuations of traffic volumes occur on two or more conflicting approaches, full-traffic-actuated signals are best suited. This type of control, which uses vehicle detectors on all approaches and an actuated control unit, assigns has the following advantages (5):
• Generally reduced delay, if properly timed
• Adaptable to short-term fluctuations in traffic flow
• Generally increased capacity by reapportioning green time
• Particularly effective at intersections with three or more phases

Alternatively, the disadvantages of traffic actuated control include:

• Added equipment cost (i.e. detectors, lead-in cables, controllers)
• Added installation and maintenance costs
• More complex than pretimed controllers

2.2.1 Traffic Signalization Hardware

A functioning traffic signal installation at a typical vehicle-actuated intersection consists of several components as shown in figure 2.2, including a controller assembly, signal heads, detection systems and interconnecting cables (3). The controller assembly is the complete electrical mechanism mounted in a cabinet for controlling assembly that changes the colors of the signal headlamps according to the timing plan. It assigns the correct and safe right of way to different approaches at appropriate times.

Vehicle detection systems, which include detectors and detector amplifiers, register the presence or passage of a vehicle at approach to an intersection and provide input for traffic-actuated signal control (3). There are variety of detector types but the most commonly used one for traffic signal applications is the under pavement loop detector. This detector senses a change in inductance caused by presence or passage of a vehicle near the sensor and sends the signal to the detector amplifier.
Figure 2.2. Traffic signalization hardware.

The detector amplifier, housed in the controller assembly cabinet, amplifies the detector signal prior to sending it to the controller. A detector amplifier can operate in one of two modes: pulse-mode and presence-mode. In pulse-mode, each time a vehicle enters the loop area the detector senses the passage of the vehicle and generates a short pulse that is passed on to the controller as a vehicle call. The controller does not drop this vehicle call when the vehicle leaves and holds the call for the period of time that the vehicle is within the field of detection. Detector amplifiers that operate in presence-mode may have two features, call-delay and call-extension, that can be used to modify the time the call is placed to the controller and/or its duration. Call-delay waits to send the call from the detector to the controller by a predetermined amount of time. If the vehicle leaves the detector before the delay time has expired, the call is not sent to the controller. This feature is typically used for right-turn-on-red or for protected/permitted left turn...
movements. The call-extension feature extends the duration of the “on” output from the
detector amplifier and is commonly used at high-speed approaches.

Interconnecting cables consist of lead-in wires and lead-in cables. The lead-in
wire runs from the edge of the loop to the pull box while the lead-in cable connects the
lead-in wire to the input of the detector amplifier. The pull box is an underground
receptacle that contains the splices between the lead-in cable and loop-lead in wire.

2.2.2 Traffic Signal Timing Concepts

The objective of signal timing is to alternate the right of way in such a way as to
provide for the orderly movement of traffic, minimize average delay to vehicles and
pedestrians, reduce the potential for accident-producing conflicts and maximize the
capacity of each intersection approach (6). Timing parameters used in the design of signal
timing include:

- **Cycle** – The total time to complete one sequence of signalization around an
  intersection.
- **Phase** – The part of a signal cycle allocated to any combination of traffic
  movements having the right of way simultaneously.
- **Phase change interval** – The time allocated for vehicles to clear the intersection
  after the green interval and before conflicting movements are released. The
  phase change interval consists of the yellow change interval and, if defined, the
  all red clearance interval.
- **All red clearance interval** – Time added to the phase change interval to satisfy
  legal requirements to clear the intersection or because of safety considerations.
at wide intersections. During this interval, a red indication is displayed for all approaches

Additional timing parameters required for vehicle-actuated operation include:

- **Minimum green interval** – The main purpose of the minimum green interval is to ensure that pedestrians have adequate time to safely cross the street. For those locations where pedestrians are few, the interval can be used to artificially ensure adequate pedestrians crossing time. In addition, the minimum green interval is also utilized to provide sufficient time for all vehicles potentially stored between the detector and the stop line to ensure the intersection (7).

- **Maximum green interval** – This is the maximum length of time that a phase can be held green in the presence of a call from a conflicting phase. Ordinarily, the maximum green interval is set between 30 and 60 seconds (8).

- **Passage time** – This is the amount of time that the green interval is extended for each actuation of the detector after the time gap between vehicles is not greater than the passage time. The total number of times the green interval can be extended is limited by the duration of the maximum green interval.

Upon approach to a signalized intersection some drivers may be faced with a dilemma zone situation. This occurs when the yellow interval is presented to the drivers and they can neither stop safely behind the stop line nor continue through the intersection before the start of the cross street green interval. This poses a potentially hazardous situation. If drivers attempt to stop, there is a chance for rear end collisions. If they decide to continue through the intersection, they may expose themselves to right angle
collisions. The objective is to minimize the dilemma zone. Various methods currently utilized to protect drivers from dilemma zone situation include phase change interval timing and the use of passage detectors to extend the green interval. The timing of the phase change interval is critical at all signals installations while passage detectors are employed at high-speed, isolated intersections.

Due to the fact that the timing of the phase change interval is a controversial topic, much has been written about it. Normally, the range for the yellow change interval is within 3 to 6 seconds (9). For safety reasons, yellow intervals are seldom less than three seconds (10). A longer yellow interval allows drivers more time to proceed through the intersection, but it should not be so long as to cause a loss in efficiency and capacity at the intersection or encourage driver disrespect. Taking this into account, many installations limit the yellow interval to a maximum of five seconds and if a longer phase change interval is required, an all red clearance interval can be inserted (11). However, excessive use of all red leads to a reduction of intersection capacity as no vehicle can move during this time.

Various formulae have been published to assist with the timing of the phase change interval, which is dependent on the approach speed and other factors related specifically to an intersection. While the 85th percentile speed (the free low speed at which 85 percent of the motorists drive at or below) is typically used when calculating the yellow interval, attention should also be paid to slower vehicles traveling at the 15th percentile speed. These vehicles may require a longer clearance time, especially at wide intersections, and therefore, encounter dilemma zone situations.
When passage detectors are used to provide dilemma zone protection, the detectors are placed upstream from the intersection, so that vehicles are detected upon approach to the intersection and the green interval is extended to ensure safe passage through the intersection. Since a maximum green interval is defined to ensure conflicting movements are serviced, dilemma zone protection is not provided for those drivers who approach the intersection after the maximum green time has expired.

2.3 Existing Dilemma Zone Protection Strategies

The dilemma zone problem has been addressed by several different configurations for high-speed intersections (12). Four alternative configurations were evaluated including the newly developed IntelliDrive signal control system. The existing three dilemma zone protection detector configurations are Bielere detector configuration, South District Institute of Transportation Engineers (SDITE) detector configuration and the most famous Bonneson detector configuration.

2.3.1 Beirele Configuration

The type of the system Bierele configuration is based on is the Multiple-point detection system for basic controllers system which uses a locking memory with full-actuated mode. This configuration uses a 1 second passage time throughout the detection zone (12). The controller operates in a fully actuated locking mode. It utilizes 6 foot x 6 foot presence mode loop detectors. The detector layout is based on a safe stopping distance for vehicles with different speeds. The outermost detector is placed where a vehicle with design speed can stop safely. The second detector is located at the safe stopping distance for a speed 10 mph less than the design speed. Other detectors closer to
the intersection follow the same procedure, with 10 mph less each time, until the last one is within 75 ft of the stop bar.

Figure 2.3. Bierele-recommended detector configuration for 50 mph design speed.

The Texas Department of Transportation (TxDOT) modified this configuration with AASHTO stopping distance criteria. For speeds greater than 40 mph, the first 3 detectors from the stop line are based on 40 mph speeds and operate on 1 amplifier (speed detector). Two additional, detectors are placed at 1 second intervals for 40 mph until a distance of 349 ft (106.4 m) is reached, and tied to the second amplifier (standard detector). For speeds greater than 40 mph the special detectors are disabled, and only the standard detectors extend the green to maintain a tolerable gap. The detector placement in the Bierele configuration for 50-mph design is shown in the figure (2.3)

2.3.2 SDITE Detector Configuration

The South District Institute of Transportation Engineers (SDITE) detector configuration (13, 14) uses a basic actuated detection without any volume density controllers. This configuration uses multiple 6 foot x 6 foot detectors which operate in nonlocking mode. The passage time adapted by SDITE configuration is 2 seconds.
This configuration utilizes primarily engineering reasoning to determine the position of the detectors from stop line. The outermost detector is positioned at approximately 5 seconds of travel design speed to give protection to dilemma zone. The second detector should be located to allow the 50-mph vehicle to hold the green till it clears the intersection. The remaining detectors are placed to accommodate vehicles with reduced speed than the design speed of the road. This configuration uses a stop line detector and this prevents premature gap-out during queue discharge. Figure 2.4 represents the SDITE recommended detector configuration for 50 mph design speed road.

2.3.3 Bonneson Detector Configuration

The most famous dilemma zone protection detector configuration Bonneson detector configuration was developed by Bonneson and McCoy (15) in 1994. They developed two different recommendations for rural and urban intersections. We will be considering the recommended detection design for rural intersections in this research. This configuration uses locking memory with pulse-mode detection. The passage time is set as 2 seconds and this configuration do not use the stop line detector.
Figure 2.5. Bonneson-recommended detector configuration for rural intersections with 50 mph design speed.

According to the design speed of the road, advanced detectors are located at the starting of the dilemma zone. The outermost detector will be having the same design speed as the road, and every subsequent detector has a design speed 10mph less than the one before it. The intention is to carry the last vehicle through its dilemma zone before the signal turns to yellow or to carry the last vehicle to the stop bar before the onset of yellow light. Figure 2.5 represents the Bonneson detector configuration for a road design speed of 50 mph.

2.4 Concluding Remarks

All of the above described detection configurations are intended to provide dilemma zone protection. Compared to fixed time control and single detector actuated signal control, these detector configurations improved safety to a significant amount. Still due to the limitations existing with the loop detectors, the dilemma zone is not always completely protected. Out of the above three described detector configurations, Bonneson proved to be the much better in terms of both safety and efficiency (16). There is no such thing as complete dilemma zone protection, but there still exists a chance to minimize dilemma hazard at the high speed intersections utilizing the modern day technology.
Based on this, applying the concept of IntelliDrive, a new system was developed at the University of Akron’s Transportation Lab which can significantly improve the safety without losing the efficiency.
3.1 Introduction to IntelliDrive Technology

IntelliDrive formerly known as Vehicle-Infrastructure-Integration is an initiative fostering research and applications development for a series of technologies directly linking road vehicles to their physical surroundings, first and foremost in order to improve road safety. To save lives and prevent injuries on roadways, communication among vehicles and between vehicles and the roadside is required. Such advanced, wireless communication is supported by dedicated short-range communications (DSRC). Data transmitted from the roadside to the vehicle could warn a driver that it is not safe to enter an intersection. Vehicles could serve as data collectors and anonymously transmit traffic and road condition information from every major road within the transportation network. Such data would provide transportation agencies with the information needed to implement active strategies to relieve traffic congestion.

This capability to identify, collect, process, exchange, and transmit real-time data provides drivers with a greater situational awareness of the events, potential, threats, and imminent hazards within the vehicle’s environment. When combined with technologies that intuitively and clearly present alerts, advice, and warnings—drivers can make better and safer decisions while driving. Additionally, when further combined with automated vehicle-safety applications, IntelliDrive provides the vehicle with the ability to respond...
and react when the driver can’t or doesn't in time, significantly increasing the effectiveness of crash prevention and mitigation applications. Some potential applications of IntelliDrive are described below:

- Vehicle-to-vehicle (V2V). When a vehicle breaks suddenly, it can transmit a notice to vehicles behind that enable those vehicles to warn drivers to stop, or automatically apply brakes if a crash is imminent.
- Vehicle-to-infrastructure (V2I). A vehicle in an accident could transmit incident data—time of incident, type of crash, severity—through a roadside infrastructure device to system operators who then broadcast regional warnings. Simultaneously, incident data could be transmitted directly to emergency dispatchers for emergency response.
- Vehicle-to-others (V2D). A car turning right may be able to send an alert to a bicyclist’s cell phone or device on the bike and avoid a potential collision.

3.2 Proposed Algorithm

The proposed algorithm is based on the IntelliDrive technology which utilizes information regarding speed of the vehicle, position of the vehicle in the network, and the length of the vehicle along with road gradient and road friction information. Present research presented in this thesis deployed only speed of the vehicle, position of the vehicle and the length of the vehicle, leaving a room for road gradient and road friction information which is to be deployed in the future improvements. The algorithm logic is based on the stopping sight distance and passing sight distance criteria of the Highway Capacity Manual (HCM) (17).
Figure 3.1. Algorithm showing the proposed IntelliDrive Signal Control System logic.
The proposed method controls the signals as shown in the above figure 3.1. First when the program starts, vehicles waiting in the queue on the in the red light approach were counted and a minimum green (MinG1) for that direction of traffic is calculated such that the calculated minimum green can clear the queue from the intersection when green is served for that approach. If it is a two-way flow, the direction having maximum number of vehicles is considered. Vehicles below 5 mph speed are treated in queue so as to consider the slowing moving vehicles, which are not completely stopped. It is assumed here that vehicles drop their speeds below 5 mph only, if there is an interruption to the traffic flow in the form of traffic signal without any other obstructions. Hence efficiency can be maintained by providing sufficient minimum green time. Also a second minimum green (MinG2) time is calculated based on the pedestrian criteria, so as to ensure enough time for the pedestrians to cross the intersection. Out of these two minimum greens, the one with less value is finally adopted as the final minimum green for that approach of traffic flow. Thus minimum green is dynamic, based on the demand from the queue length and pedestrian walk time. Adapting dynamic minimum green can avoid unnecessary extensions and reduce the delay in opposite directions, thus increasing the system efficiency.

\[ \text{MinG1} = l \times t_1 + t_2 \]  

Where, MinG1 = Minimum green time 1  
\( l \) = maximum queue size  
\( t_1 \) = Average headway in seconds  
\( t_2 \) = Startup delay time in seconds
\[ \text{MinG}_2 = \frac{w}{v_p} \]  

(3.2)  

Where, MinG2 = Minimum green time 2  
\( w \) = width of the intersection to cross in feet  
\( v_p \) = pedestrian walking speed in fps  

Thus, from equations 3.1 and 3.2 the minimum green timings are calculated and the minimum of the two is finally used in the signal control by the algorithm.  

After obtaining the minimum green, if there is a conflicting call from the other approach then IntelliDrive environment is activated, thus for each and every vehicle its speed, position and length information is send to the API controller from the COM interface as shown in the figure 3.2. The overview of the system is presented in the figure 3.2.  

![Figure 3.2. Overview of the UA Intelligent Signal Control System.](image)

API after receiving the IntelliDrive information, checks pass zone condition (defined in the next section) on the onset of yellow light. If a vehicle passes the pass zone condition, then there is a probability that the vehicle can be in dilemma zone, if not it is
safe. To confirm if the vehicle will be in dilemma zone or not, stop zone condition
(defined in the next section) is checked once if it passes stop zone condition then the
vehicle soon will face a dilemma zone situation in very near future. Thus this information
is processed and a decision to extend or terminate green time is made in the API
ccontroller and is send to the signal changes in the VISSIM model.

On the onset of yellow light, if the driver decided to stop, the distance required to
safely stop before entering the intersection is known as the stop zone and is defined as

\[ StopZone = vt + \frac{v^2}{2g(f \pm G)} \]  

(3.3)

Rearranging equation 3.3 on the time line we get the stop zone condition as,

\[ \left(\frac{d}{v} - t - \frac{v}{2g(f \pm G)}\right) > 0 \]  

(3.4)

Where d = distance of the vehicle from stop line when tracked (ft)

v = speed of the vehicle when tracked (fps)

t = driver perception reaction time (sec)

g = acceleration of gravity (ft/sec²)

f = coefficient of friction

G = roadway gradient (%/100)

From the onset of the yellow interval, the distance a vehicle must travel with no
change in speed and be able to safely proceed through the intersection is the pass zone/go
zone and is formulated as follows:

\[ GoZone = yv - (W + L) \]  

(3.5)
Rearranging equation 3.5 on the time line we get pass zone condition as,

\[
\left(\frac{d}{v} - y + \frac{(W + L)}{v}\right) > 0
\]

(3.6)

Where: 
- \(d\) = distance of the vehicle from the stop line (ft)
- \(y\) = yellow interval (sec)
- \(v\) = vehicle approach speed (ft/sec)
- \(W\) = intersection width (ft)
- \(L\) = length of vehicle (ft)

As can be seen from equation 3.4 and equation 3.6, the major factors deciding if a vehicle can be caught in dilemma zone are the speed of the vehicle, length of the vehicle and position of the vehicle at the onset of yellow light. Also, these factors decide the size and position of the dilemma zone; hence dilemma zone is unique for every vehicle. Thus supporting the fact that dilemma zone is dynamic in nature in real (18). The other minor factors effecting dilemma zone are road friction and road gradient. In this research so far, they are assumed to be constants at an intersection, since their effect is very minimum.

IntelliDrive environment built in COM interface can acquire this information and send to the API server, thus signal control is done by API server and information is send to the VISSIM model for signal changes.

3.3 Summary of the New IntelliDrive Signal Control System

The features of this new IntelliDrive signal control system can be summarized as follows:

- Adopts dynamic minimum green to minimize the delay, thus improving efficiency of the system
• System checks for green termination only when there is a conflicting call from the other approaches

• Passage time or the green extension time is one second and checks for dilemma zone safety each second when receives a conflicting call

• Second-by-second tracking of the vehicles in both approaches

• Terminates green only if, there is no vehicle caught in dilemma zone or when a max-out occurs.

To better understand the newly developed system, a comparison is drawn with the already existing systems for various parameters used. Table 3.1 provides various features of the other three detector configurations along with newly developed algorithm.

Table 3.1 Features of the Different Dilemma Zone Protection Strategies

<table>
<thead>
<tr>
<th></th>
<th>Beirele</th>
<th>SDITE</th>
<th>Bonneson</th>
<th>Algorithm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min G</td>
<td>15/10</td>
<td>15/10</td>
<td>15/10</td>
<td>Queue size</td>
</tr>
<tr>
<td>Max G</td>
<td>60/20</td>
<td>60/20</td>
<td>60/20</td>
<td>60/20</td>
</tr>
<tr>
<td>Passage time(sec)</td>
<td>1sec</td>
<td>2sec</td>
<td>2sec</td>
<td>1sec</td>
</tr>
<tr>
<td>Detector Memory</td>
<td>Locking</td>
<td>Non locking</td>
<td>Locking</td>
<td>NA</td>
</tr>
<tr>
<td>Detector mode</td>
<td>Presence</td>
<td>Presence</td>
<td>Pulse</td>
<td>NA</td>
</tr>
<tr>
<td>Stop-Line Detector</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>NA</td>
</tr>
</tbody>
</table>
CHAPTER IV

COMPUTER MODELING AND IMPLEMENTATION

4.1 History and Background of Traffic Simulation

Traffic simulation is commonly used in transportation engineering today. In an effort to manage traffic as efficiently as possible, traffic simulation has gained recognition as a very useful tool for the design of improvements to urban freeway systems. Traffic simulation enables the engineer to predict the outcome of a proposed change to the traffic system before it is implemented, evaluate the merits of competing designs, quantify traffic operations and assess traffic conditions for various design alternatives of transportation facilities.

Traffic simulation models are generally classified into macroscopic, mesoscopic and microscopic models, depending on their level of modeling detail. [19] Macroscopic models (TRANSIT-7F, HCS) describe the traffic process with aggregate quantities, such as flow and density. They cannot model the interactions of vehicles on design configurations. Microscopic models (CORSIM, SYNCHRO, VISSIM, PARAMICS, and INTEGRATION) describe the behavior of the individual drivers as they react to their perceived environments. They essentially produce trajectories of vehicles as they move through the network. They have embedded algorithms and rules describing how vehicles move and interact, including acceleration, deceleration, lane changing and different
driving behaviors. Mesoscopic models combine the properties of both microscopic and macroscopic simulation models. Mesoscopic models are somewhat less consistent than microscopic tools, but are superior to some other traffic analysis techniques. These models simulate individual vehicles, but describe their activities and interactions based on aggregate (macroscopic) relationships.

A macroscopic model is often sufficient for the purpose of evaluating a proposed modification. These models tend to be easier to calibrate, since their parameters can be directly related to field data available from the existing sensor infrastructure. However since microscopic traffic simulation depends on a large number of inputs and parameters, the calibration process may become more difficult.

Nowadays, microscopic traffic simulation tools have wider applications than macroscopic models because of their flexibility to represent spatial and temporal demand patterns, different driving behavior habits, OD matrix and detailed traffic control functions, and strategies. Among all the different kinds of microscopic traffic simulation models, VISSIM is more commonly and widely used than others. This thesis includes a case study with VISSIM simulation; therefore, an introduction of this software will be given in the following section

4.2 Introduction of VISSIM (Urban Traffic Simulator)

German for Traffic in Towns – Simulation (VISSIM) has the ability of model transit and traffic flow in urban areas as well as interurban motorways on a microscopic level. It is a product with continuous add-ons provided by research institutions. VISSIM
is a micro simulation program developed at the University of Karlsruhe, Germany during the early 1970s.

VISSIM is a microscopic, behavior-based, multi-purpose traffic simulation program and has become increasingly popular throughout the world. It offers a wide variety of urban and highway applications, integrating public and private transportation. Even complex traffic conditions are visualized in an unprecedented level of detail, providing realistic traffic models.

The traffic flow model of VISSIM is a discrete, stochastic, time step based microscopic model, with driver-vehicle-units as single entities. The model is based on the continuous work of Wiedemann at the University of Karlsruhe, and further calibrated and validated by PTV AG. Car-following and lane changing together form the traffic flow model, being the kernel of VISSIM.

VISSIM can be applied as a useful tool in a variety of transportation problem settings. The following list provides some outstanding features of VISSIM (20):

- Easy network editing

  VISSIM provides the users with a convenient and intuitive network editor. It creates and edits traffic networks based on background images supporting many different formats. In VISSIM, users can define highly exact positioning of numerous network elements and import the network topology and fixed time signal control from SYNCHRO.

- Vehicle behavior modeling

  VISSIM utilizes the Wiedemann 99 car-following model for freeway travel based on the work of R. Wiedemann. This model contains ten modifiable car-following
parameters that classify the reactions of drivers in one of four driving modes: free
driving, approaching, following or decelerating. A Lane-changing behavior model has
also been built in.

- Urban and regional traffic operations and control

It has a wide range of applications such as non-signalized intersections,
conflicting movements and coordinated and actuated traffic signals. There are built in
fixed-time signal control and NEMA controller, and users can develop any other type of
vehicle-actuated signal controller that can be coded with VAP, the C-like traffic control
language.

- Numerous analysis options

VISSIM provides a wide range of customizable evaluation such as number of
vehicles, average speed, travel time, delay time and length of traffic queue spillback. The
most recent patch also includes the Node analysis for users to evaluate the LOS and
delays for different intersections.

- A variety of animation capabilities

In VISSIM, users can visualize the vehicle movements either in 2D or 3D which
will offer clearer and more descriptive presentations.

VISSIM results are used to define optimal vehicle actuated signal control
strategies, test various layouts and lane allocations of complex intersections, test the
location of bus bays, test the feasibility of complex transit stops, test the feasibility of toll
plazas, and find appropriate lane allocations of weaving sections on motorways. VISSIM
is coupled with micro-scale decentralized controllers of various signal control
manufacturers to test their control strategies in detail before they are implemented.
VISSIM is a multipurpose simulator aimed for technical staff at cities responsible for signal control, transit operators, city planners and researchers to evaluate the influence of new control and vehicle technologies.

The traffic flow model of VISSIM is a discrete, stochastic, time step based microscopic model, with driver-vehicle-units as single entities. The model contains a psycho-physical car-following model for longitudinal vehicle movement, and a rule-based algorithm for lane changing (lateral movements). Vehicles follow each other in an oscillating process. As faster a vehicle approaches a slower vehicle on a single lane, it has to decelerate. The action point of conscious reaction depends on the speed difference, distance and driver dependent behavior. On multi-lane links, vehicles check whether they improve their speed by changing lanes. If so, they seek acceptable gaps on neighboring lanes.

Based on the discussion above, VISSIM convincingly shows efficiency in performing traffic operation analysis. In this thesis, VISSIM has been chosen as the simulation platform of IntelliDrive signal control system study.

4.3 VISSIM-Based Microscopic Traffic Simulation Model

This model was developed to study the performance of various signal control and dilemma zone protection strategies presently in use along with the IntelliDrive signal control system developed at the University of Akron’s Transportation Lab to improve the safety at a high speed signalized intersection without sacrificing efficiency. The purpose of this model is to examine how delay and percentage of max-outs vary for each of the Bierele, SDITE, Bonneson detector configurations and IntelliDrive signal control system
for various minor road and major road volumes. Simulation run length is 1 hour (3600 seconds). VISSIM 4.30-05 version is used to develop and simulate the model. Visual representation of the model elements are presented in the following part.

4.3.1 Detailed model Documentation

Below are screenshots of the model showing lane configuration, traffic composition used and driving behavior parameters for car following and lane changing. Simulation parameters and various performance evaluations used for post processing the data are shown.

Figure 4.1. Traffic composition.
Figure 4.2. Study area of the IntelliDrive Signal Control System.

Figure 4.3. Desired speed distributions.
Figure 4.4. Driver behavior parameters: Car following.

Figure 4.5. Driver behavior parameters: Lane change.
Figure 4.6. Driver behavior parameters: Lateral distance.

Figure 4.7. Driver behavior parameters: Signal control.
Figure 4.8. Link types.

Figure 4.9. Simulation parameters.
4.4 Data Analysis

In this study, after setting up the VISSIM simulation platform which can communicate with COM server and the API interface different traffic scenarios were tested for the developed algorithm and also for the existing dilemma zone protection strategies and found convincing results.

Each configuration was implemented on the basis of its specific characteristics of the detector location and traffic-controller timings by using NEMA controller included in VISSIM. Lane-by-lane detection was achieved by using a separate phase for each lane. The detector layouts for all alternatives were designed according to an arterial design speed of 50 mph. The minimum and maximum green times were 15 and 60 seconds for arterial road and 10 and 20 seconds for minor road. Four levels of traffic volume 200
vphpl, 300 vphpl, 400 vphpl, 500 vphpl on major road for each case of a minor road volume – 50 vphpl, 100 vphpl, 150 vphpl were also used to investigate the performance of the existing detector configurations and newly developed algorithm under different major and minor road volume conditions. A relatively low desired speed (30 mph) was used on the cross street, with a configuration of 6-ft long stop-bar presence detectors on both minor-road directions. Each alternative was run 10 times for 3600 seconds under each major street volume and minor street volume for different random seeds. After the simulation process, the outputs from a total of 480 independent 3600-s runs were analyzed and compared.

The three detector configurations Bierele, SDITE, and Bonneson were compared with the newly developed IntelliDrive signal control system on the basis of a safety surrogate and efficiency. The measure of effectiveness for safety performance is the number of instances of the major road green indication being forced off because of the maximum timer, i.e., percentage of max-out occurrence. Average vehicle delay is used as an efficiency measure of effectiveness.

4.4.1 Max-out Occurrences

The term “max-out” refers to the immediate termination of the green when it has been extended to its maximum allowable time even if there is a continues call for the green extension. If the green is terminated by max-out instead of gap-out, the signal controller will not provide any dilemma zone protection to the vehicles once after forced green termination. To be precise, a higher occurrence of max-out indicates less effective dilemma zone protection and hence more chances of vehicles being caught in the
dilemma zone. As we have discussed earlier, the major part of this research is to reduce the percentage of vehicles being caught in the dilemma zone, by reducing the percentage of max-outs.

Table 4.1 Average Percentage of Max-outs for 10 Simulation Runs

<table>
<thead>
<tr>
<th>Minor Rd Vol</th>
<th>Major Rd Vol</th>
<th>Bierele</th>
<th>SDITE</th>
<th>Bonneson</th>
<th>UA Algorithm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vphpl</td>
<td>vphpl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>200</td>
<td>1.326</td>
<td>6.99</td>
<td>1.9</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>2.8657</td>
<td>23.4</td>
<td>5.125</td>
<td>0.217</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>8.324</td>
<td>26.24</td>
<td>17.711</td>
<td>0.248</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>20.43</td>
<td>75.32</td>
<td>32.66</td>
<td>3.616</td>
</tr>
<tr>
<td>100</td>
<td>200</td>
<td>0.111</td>
<td>4.41</td>
<td>6.117</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>0.487</td>
<td>19.96</td>
<td>2.28</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>6.597</td>
<td>48.78</td>
<td>13.53</td>
<td>1.6313</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>14.845</td>
<td>72.28</td>
<td>29.443</td>
<td>4.32</td>
</tr>
<tr>
<td>150</td>
<td>200</td>
<td>0</td>
<td>3.57</td>
<td>0.338</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>0.249</td>
<td>19.93</td>
<td>1.979</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>5.073</td>
<td>48.43</td>
<td>11.32</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>15.2</td>
<td>71.03</td>
<td>30.74</td>
<td>4.972</td>
</tr>
</tbody>
</table>

The results of the max-out occurrences on the major road for each of the detector configurations Bierele, SDITE, Bonneson and the proposed UA algorithm are shown in Table (4.1). As we can observe from the table, the SDITE configuration generated largest max-out occurrences compared to the rest three. This may be because SDITE uses more number of detectors than any other detector configuration which makes it hard to find out a gap to terminate green phase before max-out. UA algorithm, which is based on the intellidrive technology, outperforms all the detector configurations in minimizing the percentage of max-outs, thus reducing the probability of vehicles being caught in the dilemma zone.
4.4.2 Average Vehicle Delay

The average total delay time of all the vehicles traveling on both the major road and minor roads is taken as an effective measure to compare the operational efficiency of the intersection for different major road volumes and minor road volumes. The average vehicle delay is calculated as the difference between the actual travel times of the vehicles to free flow travel times. The free flow travel time is obtained when there are no other vehicles or interruptions to the flow. The level of service (LOS) of an intersection depends on the delay; hence LOS can be improved by reducing the overall delay of the intersection. The average total delay of every vehicle is calculated by dividing the total time by the number of vehicles in the simulation network. Thus minimizing delay improves the operational efficiency of a network.

Table 4.2 Average Total Vehicle Delays for 10 Simulation Runs

<table>
<thead>
<tr>
<th>Minor Rd Vol vphpl</th>
<th>Major Rd Vol vphpl</th>
<th>Bierele</th>
<th>SDITE</th>
<th>Bonneson</th>
<th>UA Algorithm</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>200</td>
<td>6.3406</td>
<td>7.0077</td>
<td>6.6249</td>
<td>5.3671</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>6.3463</td>
<td>6.9319</td>
<td>6.9176</td>
<td>5.6712</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>6.7701</td>
<td>7.031</td>
<td>7.194</td>
<td>6.5746</td>
</tr>
<tr>
<td>100</td>
<td>200</td>
<td>7.9508</td>
<td>9.1193</td>
<td>8.2889</td>
<td>6.972</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>8.0018</td>
<td>9.281</td>
<td>8.3931</td>
<td>7.353</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>8.2648</td>
<td>9.3764</td>
<td>8.5621</td>
<td>8.1353</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>8.66</td>
<td>9.3415</td>
<td>8.83167</td>
<td>8.6126</td>
</tr>
<tr>
<td>150</td>
<td>200</td>
<td>8.923</td>
<td>10.6694</td>
<td>9.1494</td>
<td>7.8503</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>9.4587</td>
<td>11.3156</td>
<td>9.4155</td>
<td>8.42</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>10.047</td>
<td>11.7372</td>
<td>9.9987</td>
<td>9.274</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>10.5592</td>
<td>11.8609</td>
<td>10.479</td>
<td>10.264</td>
</tr>
</tbody>
</table>

From the table above, SDITE detector configuration produces more delay relatively compared with Bonneson, Bierele and UA algorithm. Delays from Bonneson...
detector configuration and Bierele detector configuration are minimum compared to SDITE, while UA algorithm performs better than all the three detector configurations, thus maintaining efficiency while increasing the safety significantly which fulfills the purpose.

4.5 Concluding Remarks

In this chapter, a simulation model was built to test the safety and efficiency of the proposed algorithm comparing with the presently existing dilemma zone protection strategies. Clearly, safety in significantly increased in the proposed algorithm compared with the rest detector configurations.

![Figure 4.11. Average vehicle delay for minor road volume of 50 vphpl.](image)
Figure 4.11, figure 4.12, and figure 4.13 represents the graphical view of average vehicle delays for different major road and minor road volumes. As seen from the figures, operational efficiency was maintained a little higher than the Bierele, SDITE and Bonneson detector configurations.
Figure 4.14. Percentage of max-out occurrence for minor road volume of 50 vphpl.

Figure 4.15. Percentage of max-out occurrence for minor road volume of 100 vphpl.
Figure 4.16. Percentage of max-out occurrence for minor road volume of 150 vphpl.

Figure 4.14, figure 4.15, and figure 4.16 represents the graphical view of percentage of max-out occurrences for different major road and minor road volumes. The percentage of max-outs decreased a lot for the IntelliDrive signal control system, compared to the rest, thus fulfilling the major goal of this research objective.
CHAPTER V

PEAK FLOW FACTOR CHARACTERISTICS AT INTERSECTIONS

5.1 Introduction

One of the most important traffic parameters used to determine the Level of Service of an isolated intersection or highway is traffic flow. Because operational design of a roadway facility is dependent upon the worst operating conditions during the day, the expected daily peak traffic flow must be determined. In order to define the peak traffic flow as accurately as possible, it is particularly important to observe traffic flow variability during the peak hour.

The measure of flow variability during peak hour is expressed by the peak hour factor (PHF). In the highway capacity manual (HCM) (21), the PHF is defined as the ratio between the average peak hourly traffic flow rate to the peak 15-minute traffic flow rate within the hour. It is calculated using the following equation:

\[ PHF = \frac{V_{\text{HOUR}}}{V_T \left( \frac{60}{T} \right)} \]  

(5.1)

Where \( V_{\text{HOUR}} \) is the volume of traffic during the peak hour, \( V_T \) is the volume of traffic during the peak time interval, and \( t \) is a peak time interval of 15 minutes.

Although, a PHF value in the range of 0.85 to 0.95 is used in most field applications, theoretically, the PHF can range anywhere from 0.25 to 1.00. If there were
very little fluctuations within the hour, the PHF would be close to 1.00. On the other hand, a PHF of below 0.80 would be indicative of high volume fluctuation.

In operational analysis, determination of the PHF is essential for computing the peak traffic flow of an intersection or highway and the subsequent level of service of that roadway. This is accomplished by dividing the PHF into the peak hourly flow rate to obtain the adjusted peak period flow rate. Therefore, as the PHF decreases, the calculated peak period flow rate increases. Thus, if the PHF used in the level of service analysis is much lower than field conditions show, it could lead to oversaturation of the worst operating conditions of a roadway whereas use of an inappropriately high PHF could lead to underestimation of the worst operating conditions.

Although some prior studies acknowledge that the importance of the PHF is often overlooked, not much emphasis has been placed on careful evaluation of it in the past. For example, many traffic engineers do not practically determine PHF using field data. Quite often in practice, a PHF value of 0.9 is assumed to complete the level of service of a roadway facility. Yet, field evidence often shows PHF values that differ significantly from 0.9. Also, since the HCM offers little guidance on computing the PHF for intersections, participants in a technical assistance program (May and Skabardonis) (22) for the HCM questioned whether the PHF should be computed for an entire intersection, each approach, or each movement.

Additionally, it is uncertain as to how the time interval for the PFF was chosen. Although all procedures in the HCM are currently based on a 15-minute time interval for computation of the PHF, the HCM offers little explanation on how 15 minutes became the standard for representing peak flow. The HCM does state that flow rates at shorter
time intervals are unstable. However, the actual stability of the PHF for each of the different time intervals has not been discussed.

Furthermore, field data in recent years indicates that traffic congestion could occur during a time period much shorter than 15 minutes. Therefore, it is logical to question if use of a 15-minute based peak hour factor is even adequate for representing changes in traffic dynamics.

This study analyzed PHF characteristics at various time intervals for a variety of traffic conditions. In place of the PHF, the peak flow factor (PFF) was used to define the relationship between the peak hourly volume and the peak flow rate at different time intervals. The PFF is calculated in the same manner as the PHF with the exception that the time interval defined in equation 1 may differ from 15 minutes. Furthermore, a method of continuous counting was used to evaluate the PFF at different time lags to analyze the stability of the PFF.

This study involved different types of analysis of the PFF at signalized isolated intersections; it will provide a better understanding of PFF characteristics and contribute to the existing literature on peak flow analysis.

5.2 Literature Review

Up to date, a very limited amount of research has been conducted on peak flow characteristics. The HCM has been using 15-minute based PHFs for more than thirty years, but it provides very little explanation on how fifteen minutes became the standard for representing peak flow (TRB 1997). However, there has been some discussion on the stability of traffic flow and the importance of close evaluation of the PHF at intersections.
The HCM offers some discussion on how a time interval of 15 minutes was chosen for computing the PHF. Chapter 2 of the HCM states that 5-minute flow rates have been proven to be statistically unstable through prior research, and that a design based on a 5-minute flow rate could result in substantial excess capacity during most of the peak hour. On the other hand, roadway design based upon an hourly volume could cause a substantial amount of congestion during the peak hour. Therefore, a compromising time interval of 15 minutes was chosen as the most acceptable time period.

In an article by Roess and McShane (23), it was also stated that statistically stable relationships can be expected at the 15-minute interval but not at intervals of five minutes or less. However, no further explanation was given or is in the HCM on how the stability of traffic flow at a certain time interval was determined.

The most relevant study was performed in 1990 at Delaware Transportation Center by Polus and Kikuchi 1990 (24). In this study, traffic counts were collected during afternoon peak hours for six four-lane arterial highways with varying traffic demands. Then, by using these traffic counts, the PHFs were calculated for certain time intervals that ranged from 1 minute to 20 minutes.

Results from Delaware study showed that the PHF generally increased with time interval. The variation of flow rates between the corresponding time intervals was also compared, and this indicated that the flow rates were much more stable for the longer time intervals than for the shorter ones. Therefore, in order to avoid potential over-design, it was concluded that 15 minutes would be the optimum time interval for the observation of peak flow traffic on four-lane arterial roads.
Although prior studies have been discussed traffic flow variations and the effect of the PHF on vehicle delays at signalized intersections, a focused effort on stability analysis of the actual PHF or PFF appears to be lacking. When comparing high traffic volume flows against low flows within the hour, it is logical that flow rates become more unstable at a shorter time interval. On the other hand, the claim that a 5-minute based PFF is unreliable is insufficiently supported. Separate analysis must be performed using both the peak hourly volume and the peak flow rate to analyze PFF characteristics and determine the stability of the PFF at different time interval durations.

5.3 Data Collection

Traffic counts were collected from four sites; all are isolated signalized intersections to analyze the PFF characteristics. All of these sites are located in an urban environment within the Akron, Ohio area. Traffic flow data for one direction was recorded at each of the arterial streets. At all sites, the traffic became congested at some point for a time period of less than 15 minutes although it remained relatively uncongested during most of the analysis period. Such a change in traffic dynamics shows what effect a shorter congestion period has on PFF calculations. This demonstrates that use of a smaller time interval for PFF calculations could represent short-term changes in traffic dynamics more accurately than 15-minute time interval.

The sites were chosen in order reflect ideal conditions as closely as possible. First of all, the roadway sections studied were fairly level, and all sites carried less than 3% heavy vehicles. Because the vast majority of the drivers were assumed to be commuters. Also, the traffic at all sites was recorded during good weather when the pavement was
dry. Finally, traffic flows at all four sites represented normal peak-hour traffic conditions which were uninhibited by unusual factors such as roadway blockage due to construction or traffic accidents.

To analyze the traffic flows in detail, traffic counts were recorded for every 10-second time interval within analysis period at each site. For all of the sites, manual traffic counts were recorded from videotape observation. The four intersections chosen are St. union st - Perkin st; E Exchange st - Spicer st; Hummel st - Lovers ln; E Exchange st - S Broadway st. Traffic counts were recorded between 4:00 P.M to 5:30 P.M using videotape observation at all the sites.

5.4 PFF Variations

After 10-second traffic counts were recorded for each site, the next step was to compute the PFFs for all four sites. Using time interval durations of 3 minutes, 5 minutes, 10 minutes, 15 minutes and 20 minutes, the PFF were evaluated for 10-second time lag within a total analysis period of one and half hour. This method illustrates fluctuations from one time lag to the next, there by revealing dynamic traffic characteristics within the analysis period. Variations in traffic counts can be observed from the following figures.
Figure 5.1. Variations in traffic counts at E Exchange St and S Broadway St intersection.

Figure 5.2. Variations in traffic counts at E Exchange St and Spicer St intersection.
Figure 5.3. Variations in traffic counts at Lovers Ln and Hummel Ave intersection.

Figure 5.4. Variations in traffic counts at Union St and Perkin St intersection.

The method of time lagging is illustrated in the table 5.1 which shows 3-minute interval PFF per time lag for EExchange St/SBroadway St Intersection for the last 5 minutes. First, the 10-sec traffic volume counts were added together to result in total volume count for each time lag. Then, the maximum volume count and average hourly
flow volume were identified for each time lag and used in equation 1 to compute the PFF for that step in time.

Table 5.1 Illustrating Method of Time Lagging for the Last 5 minutes for E Exchange and S Broadway Intersection

<table>
<thead>
<tr>
<th>Time</th>
<th>Flow</th>
<th>Volume</th>
<th>PFF</th>
</tr>
</thead>
<tbody>
<tr>
<td>5:25:00</td>
<td>0</td>
<td>43</td>
<td>1109</td>
</tr>
<tr>
<td>5:25:10</td>
<td>8</td>
<td>51</td>
<td>1112</td>
</tr>
<tr>
<td>5:25:20</td>
<td>3</td>
<td>54</td>
<td>1112</td>
</tr>
<tr>
<td>5:25:30</td>
<td>0</td>
<td>42</td>
<td>1112</td>
</tr>
<tr>
<td>5:25:40</td>
<td>0</td>
<td>40</td>
<td>1105</td>
</tr>
<tr>
<td>5:25:50</td>
<td>5</td>
<td>45</td>
<td>1103</td>
</tr>
<tr>
<td>5:26:00</td>
<td>4</td>
<td>49</td>
<td>1106</td>
</tr>
<tr>
<td>5:26:10</td>
<td>1</td>
<td>40</td>
<td>1107</td>
</tr>
<tr>
<td>5:26:20</td>
<td>1</td>
<td>38</td>
<td>1108</td>
</tr>
<tr>
<td>5:26:30</td>
<td>7</td>
<td>45</td>
<td>1108</td>
</tr>
<tr>
<td>5:26:40</td>
<td>4</td>
<td>49</td>
<td>1111</td>
</tr>
<tr>
<td>5:26:50</td>
<td>2</td>
<td>43</td>
<td>1113</td>
</tr>
<tr>
<td>5:27:00</td>
<td>0</td>
<td>42</td>
<td>1107</td>
</tr>
<tr>
<td>5:27:10</td>
<td>6</td>
<td>48</td>
<td>1110</td>
</tr>
<tr>
<td>5:27:20</td>
<td>3</td>
<td>51</td>
<td>1112</td>
</tr>
<tr>
<td>5:27:30</td>
<td>0</td>
<td>47</td>
<td>1111</td>
</tr>
<tr>
<td>5:27:40</td>
<td>1</td>
<td>45</td>
<td>1108</td>
</tr>
<tr>
<td>5:27:50</td>
<td>8</td>
<td>53</td>
<td>1107</td>
</tr>
<tr>
<td>5:28:00</td>
<td>0</td>
<td>46</td>
<td>1107</td>
</tr>
<tr>
<td>5:28:10</td>
<td>0</td>
<td>43</td>
<td>1102</td>
</tr>
<tr>
<td>5:28:20</td>
<td>4</td>
<td>47</td>
<td>1100</td>
</tr>
<tr>
<td>5:28:30</td>
<td>8</td>
<td>55</td>
<td>1107</td>
</tr>
<tr>
<td>5:28:40</td>
<td>0</td>
<td>50</td>
<td>1107</td>
</tr>
<tr>
<td>5:28:50</td>
<td>6</td>
<td>51</td>
<td>1100</td>
</tr>
<tr>
<td>5:29:00</td>
<td>3</td>
<td>53</td>
<td>1099</td>
</tr>
<tr>
<td>5:29:10</td>
<td>0</td>
<td>46</td>
<td>1099</td>
</tr>
<tr>
<td>5:29:20</td>
<td>7</td>
<td>47</td>
<td>1092</td>
</tr>
<tr>
<td>5:29:30</td>
<td>4</td>
<td>51</td>
<td>1096</td>
</tr>
</tbody>
</table>

For each site, the resultant average PFF for the different time intervals can be found in Table 5.2. As shown, the PFF decreases as the time interval decreases at all the sites. In most cases, the largest difference in the PFF value occurs between 3-minute and
5-minute time intervals. However, even when comparing the PFFs for 10-minute and 15-minute time intervals, the difference is significant. As there is no stability in PFF after a certain time interval, the question naturally arises which time interval has to be chosen to evaluate the Peak Hour Factor.

Table 5.2 Mean PFFs at a 10-second Time Lag Interval

<table>
<thead>
<tr>
<th>Site</th>
<th>3Min PFF</th>
<th>5 Min PFF</th>
<th>10 Min PFF</th>
<th>15 Min PFF</th>
<th>20Min PFF</th>
</tr>
</thead>
<tbody>
<tr>
<td>EExchange-SBroadway</td>
<td>0.64</td>
<td>0.71</td>
<td>0.75</td>
<td>0.80</td>
<td>0.86</td>
</tr>
<tr>
<td>EExchange-Spicer</td>
<td>0.52</td>
<td>0.65</td>
<td>0.70</td>
<td>0.76</td>
<td>0.84</td>
</tr>
<tr>
<td>Lovers-Hummel</td>
<td>0.56</td>
<td>0.71</td>
<td>0.83</td>
<td>0.87</td>
<td>0.89</td>
</tr>
<tr>
<td>Perkin-StUnion</td>
<td>0.59</td>
<td>0.65</td>
<td>0.80</td>
<td>0.85</td>
<td>0.88</td>
</tr>
</tbody>
</table>

After computing the mean PFFs, the standard deviations were computed for each time interval and are shown in Table 5.3. As shown from this table, a discernable relationship cannot be identified for the change in standard deviation as for the change in the mean PFF between time intervals. This would imply that although traffic flow might be expected to become more unstable as the time interval decreases, the PFF does not necessarily become more unstable as well.

Table 5.3 Standard Deviation of PFFs at a 10-second Time Lag Interval

<table>
<thead>
<tr>
<th>Site</th>
<th>3Min PFF</th>
<th>5 Min PFF</th>
<th>10 Min PFF</th>
<th>15 Min PFF</th>
<th>20Min PFF</th>
</tr>
</thead>
<tbody>
<tr>
<td>EExchange-SBroadway</td>
<td>0.008</td>
<td>0.008</td>
<td>0.008</td>
<td>0.011</td>
<td>0.007</td>
</tr>
<tr>
<td>EExchange-Spicer</td>
<td>0.015</td>
<td>0.023</td>
<td>0.028</td>
<td>0.041</td>
<td>0.036</td>
</tr>
<tr>
<td>Lovers-Hummel</td>
<td>0.017</td>
<td>0.022</td>
<td>0.032</td>
<td>0.022</td>
<td>0.026</td>
</tr>
<tr>
<td>Perkin-StUnion</td>
<td>0.007</td>
<td>0.007</td>
<td>0.009</td>
<td>0.010</td>
<td>0.010</td>
</tr>
</tbody>
</table>
5.5 Time Series Models

Additional analysis of the PFF for the four sites included developing a time series model to observe changes in the PFF from one 10-second time lag to the next. As part of this time model, Holt’s Method (25) was used to forecast the PFF after the fifth time lag to judge whether or not changes in the PFF values are predictable.

The Holt’s method is based upon a process of exponential smoothing which represents a linear combination of all values in a time series up to the time in question. The smoothing estimate and forecast estimate at each time lag were computed using the following equations

\[ h_t = b (y_t - y_{t-1}) + (1 - b) h_{t-1} \]  
\[ y_t = ax_t + (1 - a) (y_{t-1} + h_{t-1}) \]  
\[ y_{EST} = y_t + h_t \]

Where \( x_t \) represents the original estimate, \( h_t \) is an exponentially smoothed estimate of the trend at time \( t \), and \( y_t \) is the actual smoothed estimate of the PFF. The smoothed estimates were computed by using two smoothing constants designated as \( a \) and \( b \) which can both range anywhere from 0 to 1.

For the first two time lags of the models in this study, the starting estimates of \( h_t \) and \( y_t \) were assumed. Next, a value of 0.5 was chosen to produce a moderate amount of smoothing for the rest of the series. In conjunction with this smoothing constant, the second soothing constant that maximizes the forecast accuracy was also applied to each model. The resultant time models are displayed in the figures below.
Figure 5.5. Charts illustrating comparison between original PFF and forecast PFF at various intersections for different time intervals.
As shown by these time series models, the amount of fluctuation varies from one site to the other, but in almost all the sites for every time interval the forecast values are close to the original PFF values, which implies that the traffic pattern can be estimated after a particular time interval.
CHAPTER VI

CONCLUSIONS

Safety and efficiency are both major concern for high-speed intersections. It has been a major problem for over the years to increase safety without sacrificing the efficiency. An intelligent signal control system was developed in the University of Akron’s Transportation lab based on the concept of IntelliDrive technology which can improve safety significantly while retaining efficiency. The currently existing various dilemma zone protection strategies can only reduce the effect of dilemma zone to a limited amount with the presently existing technology. And dynamic dilemma zone problem is not addressed in any of the existing ones.

Till today the designs are based on assuming the dilemma zone to a static one. But in real dilemma zone is dynamic and varies for every vehicle according to their speeds and length. The new intelligent signal control system can address the dynamic dilemma zone problem, thus providing maximum safety at intersections. Every vehicle in the system is addressed to its own dilemma boundary rather than addressing dilemma zone for the whole intersection approach.

IntelliDrive environment is created in the VISSIM model by using COM interface. Thus the information can be obtained to micro level. Hence obtaining advance speed information, vehicle classification, vehicle location from stop line of every vehicle for every second in the system in real time, unneeded extensions can be avoided. Thus
the intersection not only operates more safely, meanwhile increasing the efficiency. VISSIM model is tested for operational efficiency when safety criteria are met and the results from the simulation study proved to have increased safety significantly compared to the existing systems.

Also, this thesis studies PFF analysis at different time intervals other than the conventional 15 minutes. Because traffic flow patterns can change considerably within a short time frame, the value of defining the PFF using smaller time intervals was investigated. The time intervals that were used for comparison against 15 minutes were 3 minutes, 5 minutes, 10 minutes and 20 minutes. To define these PFF characteristics, a method of time lagging was introduced for computing the PFF and Time series models were developed to observe trends over 10-second steps in time.

Important conclusions have been drawn about the PFF based on four intersections analyzed in this study. First of all, the range of PFFs for different time intervals at all sites varies greatly from 0.5 to 0.9. Secondly, the mean PFF decreased when the time interval was reduced at each site. Thirdly, the difference in the variance of the PFFs between each time interval is significant. Finally, a question can be raised why a conventional time interval of 15-minutes was chosen by HCM over the rest.

Through observation and analysis of the collected traffic data, this study will be useful to traffic engineers in providing a better understanding of PFF characteristics. Due to the limited amount of past research on the PFF, and the limited effort in this study, more field data must be collected in order to draw more generalized conclusions on the characteristics of the PFF. With further research, Highway Capacity Manual may incorporate more detailed guidelines as to how PFF should be computed. Because of the
PFF’s importance in determining the peak flow and subsequent Level of Service of a roadway for operational design purposes, the method of PFF computation should be chosen carefully.


4. York, I. and Al-Katib, M., Methods of Traffic Signal Control and Signal Timings at High Speed Sites, Road Transport Information and Control, Conference Publication, 472, Road Transport Division of The Institute of Electrical Engineers, Great Britain, 2000


16. Si, Urbanik, and Han, “Effectiveness of Alternative Detector Configuration for Option Zone Protection on High-Speed Approaches to Traffic Signals,” 2007 Annual Meetings CD-ROM.


