

MISSISSIPPI TRANSPORTATION RESEARCH CENTER

I-55 Integrated Diversion Traffic Management Benefit Study

STATE STUDY NO. 223

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16. Abstract <p>Traffic congestion, recurrent and non-recurrent, creates significant economic losses and environmental impacts. Integrated Corridor Management (ICM) is a U.S.DOT research initiative that has been proven to effectively relieve recurrent congestion and reduce non-recurrent congestion in a transportation network. Traffic signal coordination is an effective approach to improve travel speed and to decrease delays resulting in better travel times. Incorporating traffic signal coordination into ICM can further improve the benefits of ICM strategies. In this project, we propose an ICM optimization system. The optimization approach and the expert system are major components for the ICM optimization system. With respect to the expert system, it serves as a decision support system and a backup system for the optimization approach. For the optimization approach, an ICM optimization model is presented first. The effectiveness of the ICM optimization model is verified by a case study.</p> <p>To maximize the support of implementing ICM strategies from local transportation agencies and residents, we propose a real time and proactive offset tuning algorithm to reduce delays of detour traffic without disrupting the existing traffic. The presented algorithm explicitly incorporates diverted traffic and the diversion traffic patterns into existing traffic. The offsets of each coordinated intersection are fine-tuned and updated every ten cycles to provide smooth traffic progression for upcoming traffic within the next ten cycles. Case studies were conducted to prove the effectiveness of the proposed algorithm. The benchmark cases were established as the TRANSYT-7F coordination traffic signal timing plans. Based on the results, the proactive real time offset tuning algorithm can outperform TRANSYT-7F when the accurate diversion rates are not known in advance which conforms to reality. While, when the diversion rates are predetermined and the average additional diversion volume is taken into consideration for optimization by TRANSYT-7F, the proposed algorithm significantly reduces the number of stops and has comparable performances with TRANSYT-7F. The benefits and costs of implementing the proposed algorithm are analyzed. The net benefit and benefit to cost ratio over 10 years are nearly \$2.5 million and 1.6, respectively, which shows that the proposed algorithm is worth for application. The upcoming ICM simulation test bed, ETFOMM, is introduced. It has two major advantages which are that it has convenient ICM built-in functions and its computation time is low which makes it have the ability to become an online decision support system for ICM strategies. In the last section, conclusions and recommendations of this study are presented.</p>					
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I-55 Integrated Diversion Traffic Management Benefit Study

ABSTRACT

Traffic congestion, recurrent and non-recurrent, creates significant economic losses and environmental impacts. Integrated Corridor Management (ICM) is a U.S.DOT research initiative that has been proven to effectively relieve recurrent congestion and reduce non-recurrent congestion in a transportation network. Traffic signal coordination is an effective approach to improve travel speed and to decrease delays resulting in better travel times. Incorporating traffic signal coordination into ICM can further improve the benefits of ICM strategies. In this project, we propose an ICM optimization system. The optimization approach and the expert system are major components for the ICM optimization system. With respect to the expert system, it serves as a decision support system and a backup system for the optimization approach. For the optimization approach, an ICM optimization model is presented first. The effectiveness of the ICM optimization model is verified by a case study.

To maximize the support of implementing ICM strategies from local transportation agencies and residents, we propose a real time and proactive offset tuning algorithm to reduce delays of detour traffic without disrupting the existing traffic. The presented algorithm explicitly incorporates diverted traffic and the diversion traffic patterns into existing traffic. The offsets of each coordinated intersection are fine-tuned and updated every ten cycles to provide smooth traffic progression for upcoming traffic within the next ten cycles. Case studies were conducted to prove the effectiveness of the proposed algorithm. The benchmark cases were established as the TRANSYT-7F coordination traffic signal timing plans. Based on the results, the proactive real time offset tuning algorithm can outperform TRANSYT-7F when the accurate diversion rates are not known in advance which conforms to reality. While, when the diversion rates are predetermined and the average additional diversion volume is taken into consideration for optimization by TRANSYT-7F, the proposed algorithm significantly reduces the number of stops and has comparable performances with TRANSYT-7F. The benefits and costs of implementing the proposed algorithm are analyzed. The net benefit and benefit to cost ratio over 10 years are nearly \$2.5 million and 1.6, respectively, which shows that the proposed algorithm is worth for application. The upcoming ICM simulation test bed, ETFOMM, is introduced. It has two major advantages which are that it has convenient ICM built-in functions and its computation time is low which makes it have the ability to become an online decision support system for ICM strategies. In the last section, conclusions and recommendations of this study are presented.

Keywords: Integrated Corridor Management, Traffic Signal Coordination, Offset Tuning, Diversion, Mississippi, ETFOMM.

I-55 Integrated Diversion Traffic Management Benefit Study

1. INTRODUCTION

Traffic congestion creates significant economy losses and environmental impacts. According to the Texas A&M Transportation Institute (TTI)'s *2012 Urban Mobility Report*[1], in 2011, urban congestion in the United States (U.S.) resulted in 5.5 billion hours of extra travel time, an additional 2.9 billion gallons of fuel usage, 56 billion pounds of CO₂ generation, and \$121 billion in congestion costs. In general, traffic congestion is categorized into two types: non-recurrent and recurrent congestion.

Non-recurrent congestion does not happen frequently and is usually caused by special events, natural and human-made disasters and work zones. Recurrent congestion is the most common type of congestion people experiences on a day to day basis such as during morning and evening rush hours. Even though recurrent congestion has been well studied in the past, recent studies by several states have shown that a significant portion of congestion is nonrecurring on freeways.

Traffic management systems alleviate congestion and are attracting attention from state and local transportation agencies. Integrated Corridor Management (ICM) is a U.S.DOT research initiative that has been proven to effectively relieve recurrent congestion and reduce non-recurrent congestion in a transportation network. ICM works by improving travel time reliability as well as predictability, manage congestions, and provide travelers with rich traveler information [2]. ICM aims to optimize the utilization of existing infrastructure assets and implement unused capacity within urban corridors which is a promising approach for congestion management. By ICM, transportation practitioners operate transportation corridors as a multimodal system opposed to managing individual modes as in the traditional approach [3].

The most traditional, yet effective, way that state and local transportation agencies manage traffic is through traffic signal coordination. Traffic signal coordination improves travel speed and decreases delays resulting in better travel times. However, traffic coordination is based on the normal traffic conditions and detour traffic is not considered. With newly adaptive traffic control systems (ATCS), such as ACS Lite, SCOOT, and SCATS, the traffic signal timing plan could be adjusted to account for changes in the traffic demand. However, ATCS is not designed to accommodate the sudden surge of traffic from freeways and its effectiveness needs to be improved for ICM strategies.

In this study, the research team proposes an ICM optimization system which includes two major components: the optimization approach and the expert system. The expert system serves as an offline decision support system and it also works as a backup system for the optimization approach. With respect to the optimization approach, an ICM optimization model and a proactive real-time offset tuning algorithm are proposed. The proposed real time and proactive traffic signal coordination algorithm explicitly incorporate diversion traffic and diversion traffic patterns into ICM. The adaptabilities of traffic signal coordination for detour traffic variations is improved, since offsets of each coordinated intersection are fine-tuned periodically based on traffic fluctuations. The ideal traffic signal coordination requires changes of cycle length, split and offset. In the case of ICM, usually institutional issues require the disruption of the arterial traffic to be as little as possible. Freeway operation is usually controlled by the State Department of Transportation (DOT) and diversion of traffic to local streets usually is not favored by local

DOTs and residents. Adjustment of offset will provide the most persuasive approach to overcome this obstacle to maximize the support of ICM from locals. Therefore, our ICM traffic signal coordination is limited to real time offset tuning.

In the proposed algorithm, the offsets of each coordinated intersection are fine-tuned repeatedly for every ten cycles to provide smooth traffic progression for upcoming traffic within the next ten cycles. The proposed algorithm is expected to further improve benefits of traffic signal coordination for ICM strategies.

With the endeavors of Mississippi Department of Transportation (MDOT), a number of Intelligent Transportation System (ITS) equipment and technologies are installed and implemented in Mississippi over previous decades. MDOT has installed ACTRA, a traffic signal control and management system, to manage traffic signals for years. In recent years with the implementation of the state-wide Intelligent Transportation System, traffic cameras have been employed on major roadways across the state and nine dynamic message signs (DMS) have been installed in the Jackson area. These traffic information systems and traffic management systems provide an infrastructure and a foundation to systematically manage corridors in the Jackson area. Interstate 55 (I-55) is the major North/South freeway for commuters and other traffic in the Jackson area. It usually experiences heavy congestions during rush hours and incidents that happen on I-55 cause more serious congestions. For this reason, Interstate 55 (I-55) and its parallel arterial, State Street, are selected as the study site to apply ICM strategies to improve traffic conditions within this corridor. Specifically, the study will focus on traffic congestion on the north and south bound roadways of the I-55 corridor north of Jackson, MS, from High Street to County Line Road. Figure1 is obtained from Google Maps (<https://maps.google.com/>) which shows the study area.

In this report, the current section introduces the background of this project. The following section illustrates ICM related previous projects and research. The system architecture of this project and base line traffic simulation model development are described in the third section. In the fourth section, an optimization approach of this project is mentioned in detail. An expert system approach is presented in the fifth section. The benefits and costs of this project are analyzed in the sixth section. In the seventh section, an upcoming ICM simulation test bed, ETFOMM, is illustrated. In the final section, conclusions and recommendations of this study are proposed.



Figure Study Area

2. LITERATURE REVIEW

In this section, the previous projects and studies related to ICM as well as research concentrated on offset tuning are reviewed. The first sub-section illustrates actual ICM projects conducted in the United States. The second sub-section discusses ICM optimization models presented by other researchers. Previous offset tuning research is introduced in the third sub-section. A summary of the literature review is presented in the final sub-section.

2.1 Integrated Corridor Management Projects in the United States

In the United States, USDOT chose eight pioneer sites to analysis the benefits of ICM: 1) Dallas, Texas; 2) Oakland, California; 3) Houston, Texas; 4) San Antonio, Texas; 5) Minneapolis, Minnesota; 6) San Diego, California; 7) Montgomery County, Maryland; 8) Seattle, Washington [3]. There are three stages for these pioneer sites. *Stage 1 Concept Development*: all eight pioneer sites took part in this stage. *Stage 2 Modeling*: Dallas, Minneapolis and San Diego were chosen for conducting this stage. *Stage 3 Demonstration and Evaluation*: Dallas and San Diego were selected for this stage[4]. In addition to these eight pioneer sites, other states also conducted ICM projects.

2.1.1 Stage 1 Concept Development

Stage 1 reports of Houston, Texas and Seattle, Washington are still not available on the website of Research and Innovative Technology Administration (RITA) of the U.S. Department of Transportation. The other six pioneer sites' reports are reviewed below.

2.1.1.1 Dallas, Texas

In 2008, Dallas Area Rapid Transit (DART), City of Dallas, Town of Highland Park, et al., [5] analyzed the concept of operations for Integrated Corridor US-75 In Dallas, Texas. This report introduced the ICM corridor conditions and participating government agencies. The authors also illustrated planned ICM strategies and analyzed the needs and issues facing these strategies. This report also described involved agencies' functions and corresponding responsibilities.

In the same year, DART, City of Dallas, Town of Highland Park, et al., [6] also delivered another report to illustrate specific high-level requirements of the US 75 corridor. The authors presented the functional requirements and performance requests of the ICM system. The report proposed many requirements of the ICM system not only in the field of concept of operations but also from other aspects. The report illustrated restraints identified by agencies, as well. The authors also constructed technical scopes of the ICM system which was the foundation for evaluation when the proposed ICM system was built.

2.1.1.2 Minneapolis, Minnesota

Minneapolis pioneer team [7] presented a concept of operations for an ICM corridor in Minneapolis, Minnesota: the I-394 corridor. The authors indicated that the study corridor includes three inter-related networks, a freeway network, multiple arterials and a transit system, which were not integrally controlled. In this report, the demands, vision and concept of operation of the ICM corridor were analyzed by a systems engineering method.

Minneapolis pioneer team [8] also delivered another report to describe the system requirement specifications for I-394 Integrated Corridor Management System (ICMS) to support implementation of identified and prioritized ICM strategies. This report presented specific requirements of the entire ICMS and its subsystems. The hardware and interface requirements were also illustrated. The authors mentioned documentation and training requirements, as well.

2.1.1.3 Montgomery County, Maryland

Montgomery county pioneer team [9] developed the concept of operations for ICMS in Montgomery County, Maryland (I-270 Corridor) by a user-oriented approach. The study ICMS is about 20 miles which includes freeway networks, arterial networks, the MARC commuter rail network, the Metrorail network, the MTA commuter bus network, the Metrobus network, and the ride sharing network. This report analyzed features and conditions of the study site and then it investigated operational requirements of the ICMS. Based on these analyses, the authors identified a united control concept to satisfy these requirements.

Montgomery county pioneer team [10] also conducted another report to illustrate high-level and detailed system requirements specification (SRS) for the I-270 corridor. This report described the background of the I-270 corridor and determined the objective and range of SRS. The authors also introduced the I-270 corridor operational conditions, policies and performance assessment factors. This report illustrated the needs and requirements of the I-270 corridor in detail, as well, such as the needs of interface and data collection.

2.1.1.4 Oakland, California

Oakland Pioneer Team [11] presented the draft Concept of Operations for the I-880 Corridor in Oakland, CA for the implementation of Integrated Corridor Mobility. Two major considerations were 1) met the needs of local stakeholder agencies by considering their practical operational, intuitional, and financial limits; 2) concentrated on existing equipment and systems. The author introduced the background of the study corridor and its existing equipment and conditions of Intelligent Transportation Systems (ITS). Five major operational scenarios were considered: normal conditions, highway and arterial incidents, transit incidents, planned and scheduled events, and key unplanned events. Operational and technical feasibility, institutional limits, benefits and costs, and conformation of the regional ITS framework were used as criteria to screen ICM strategies. This report also estimated costs and benefits of implementing ICM strategies.

Oakland Pioneer Team [12] also proposed specific system requirements of ICMS for the I-880 corridor in Oakland, CA. The author described system requirements in detail in the fields of general system requirements, general and functional requirements of ICMS, data requests, and interface requirements. The research team indicated that, although the study corridor is a multi-modal corridor, the coordination of different transportation modes was limited. The author pointed out the objective of ICMS of the study corridor was to operate different transportation networks within the corridor by an integrated approach in order to improve efficiency, mobility and enhance options of transportation modes for people and goods under all situations.

2.1.1.5 San Antonio, Texas

Southwest Research Institute [13] introduced the entire ICM framework o for I-10 corridor in San Antonio, TX. The author mentioned background, needs, objectives, and strategies of the study ICM corridor. Operation polices and limits were presented. Six operational scenarios are

described: daily operation, planned event, incident, transit special event (Fiesta), severe ice storm and evacuation.

2.1.1.6 San Diego, CA

San Diego Pioneer Team [14] introduced concept of operations for ICMS in San Diego, California. A 21 miles section of I-15 corridor from State Route (SR) 52 to SR 78 is the selected ICM corridor. Freeway, arterial and transit networks are major networks in the study corridor. In this corridor, some advanced ITS technologies were already utilized, such as managed lane and 511 advanced traveler information system. Their report described outline of ICMS concept in San Diego and analyzed the needs of current corridor for implementing ICMS and benefits of ICMS applications. The authors also identified involved stakeholders and their current and future responsibilities. This report mainly concentrated on goals and objectives, information needs, strategies and institutional framework of presented ICMS. The operational, technical, institutional problems needed to be solved were also discussed. This report also pointed out respective roles and responsibilities of stakeholders and ICMS strategies for six major operational scenarios: daily operations, freeway incident, arterial incident, transit incident, special event and disaster response.

San Diego Pioneer Team [15]also introduced the specific system requirements of ICMS for I-15 in San Diego, California. Intermodal Transportation Management Subsystem (ITMS) and Decision Support Subsystem were two major subsystems of the I-15 ICMS. The background, operation modes, lifecycle management, user characteristics, and constraints of the ICMS were presented in this report. The author also mentioned assumptions and dependencies of this report. The framework, action verbs, user needs, data requirements and management, and system performance measurements and management were presented in this report, as well.

2.1.2 Stage 2 Modeling

The RITA website introduced results of analysis, modeling and simulation (AMS) of Dallas, TX; Minneapolis, MN; and San Diego, CA. The first, second and third largest benefits of ICM were enhanced travel time reliability, decreased travel time, and advantages of fuel consumption and emissions, respectively. ICM could bring more benefits in high travel demand due to non-recurrent congestions. ICM produced significant benefits for three sites with respect to 10-year net benefit (Minneapolis: \$104M; Dallas: \$264M; and San Diego: \$82M) and benefit-cost ratio (Minneapolis: 20:1; Dallas: 22:1; and San Diego: 10:1)[16].

2.1.3 Stage 3 Demonstration and Evaluation

According to the website of RITA, there are three major objectives of U.S.DOT for stage 3: 1) demonstrate benefits of ICM by corridor performance enhancements applying actual criteria; 2) ICM concepts are transferred from successful deployments to future adopters; 3) verify assumptions that ICM will enhance situational awareness, response and control, and corridor performance as well as ICM could notify travelers better [17].The final availability of Stage 3 are still pending on RITA's website.

2.1.4 ICM Projects in Other States

Hadi, Xiao et al.,[18] introduced the development of ICM in Florida. The objective of their project was to identify suitable ICM strategies for Florida and demonstrate implementation of these strategies. Based on reviewed previous ICM studies and met with state and local government agencies, the authors indicated that information sharing, performance assessment

and forecast of transportation system, decision support tool development were the most needed ICM strategies in Florida. In this project, Integrated Regional Information Sharing and Decision Support system (IRISDS), a web based system, was developed to collect and coordinate historic and real-time traffic data from field and different transportation agencies. Based on collected data, IRISDS could generate real-time transportation system performance measures and predict critical performance indexes and IRISDS could visually display these measures and indexes. In this study, the authors indicated that accurately estimating incident duration and capacity reduction due to the incident were critical to assess traffic delays caused by the incident.

2.2 Integrated Corridor Management Optimization Model

Papageorgiou[19] proposed a linear model for integrated corridor control. The author used the store-and-forward modeling approach to build the model. Total delay time or total time spent in the corridor was used as the objective function. Based on the author's previous experience, the author believed the model with the solution algorithms proposed by another of the author's articles could be utilized in real-time.

Liu, Yu, et al [20] proposed a multi-objective ICM model to optimize traffic conditions of a corridor when incidents occurred on a freeway. The two objectives of this model were 1) maximize freeway throughput; 2) minimize the total travel and queue time at off ramp, detour arterials, and on ramp. The compromised Genetic Algorithm (GA)-based heuristic algorithm was used to solve the proposed model. A case study was conducted which proved the effectiveness of the proposed model. This model was an offline model.

Abu-Lebdeh and Chen [21] evaluated the benefits of integrated corridor control by simulation. Two major strategies, diversion without and with re-timing signal plans, were evaluated. VISSIM was selected as the simulator and travel time was chosen as the performance measure. Based on the simulation results, diversion from the freeway can relieve freeway congestion levels, but increase travel time of the detour arterial. Re-timing signal plans could decrease travel time on detour route. In their paper, the re-timing signal plans were pre-generated.

Hashemi and Abdelghany et al., [22] presented a traffic management system for ICM. The system had three major components. The first and second elements were the network state estimation and a short network state prediction which was conducted by simulation models. Based on the predicted data, the effective traffic management plans were generated by the genetic algorithm (GA). A plan was simulated as a chromosome and specific control activities formed its genes. In this paper, traffic signal timing plans were pre-generated as specific control actions and they were not updated during traffic management period.

Zhang and Gou et al., [23] proposed an Integrated Corridor Traffic Optimization Model (ICTOM) to dynamically manage freeway traffic and diversion traffic. Sequential quadratic programming algorithm was implemented to optimize the diversion rate and signal timing plans on the diversion route. The results indicated ICTOM could effectively relieve the non-recurrent freeway congestion by 9.92% in total traffic delay, including freeway and arterials.

Liu and Hu[24] proposed an integrated control model for integrated corridor according to a maximum flow based control model proposed for parallel oversaturated arterials. It aimed to relieve or solve cyclic resident queue and downstream queue spillover. The key parameter of proposed maximum flow based control model was oversaturation severity index (OSI) which

measured the ratios of unused green, due to resident queue or queue spillover of the downstream intersection, over total phase green. The OSI was classified to temporal oversaturation severity index (TOSI) and spatial oversaturation severity index (SOSI) for unused green resulted from resident queue and queue spillover of the downstream intersection, respectively. Based on TOSI and SOSI, three strategies were proposed: green extension, red extension and downstream red reduction. The three approaches were individually or mixed utilized for current control period, based on TOSI and SOSI of the last control period. The authors used a forward and backward procedure to solve the proposed maximum flow based control model. Authors also proposed methods to estimated freeway performances (density, speed, and travel time) and arterial performances (average delay, travel time and residual capacity of each intersection). The authors developed a simple model to compute diversion rate based on travel time differences between freeway and arterials. Two case studies were conducted to verify effectiveness of proposed maximum flow based control model and the entire integrated control model, respectively. Based on the simulation results, the proposed two models could significantly improve performance of oversaturated arterials and integrated corridor with respect to average delay, average number of stops, and average speed. The ICM model presented by Liu and Hu was online responsive control model.

2.3 Offset Tuning

In the absence of research to implement offset tuning to accommodate the sudden surge on the arterial, we review the general offset optimization and tuning technique, with the focus of real time implementation in mind.

Li and Furth et al, [25]proposed a method to calculate most-likely optimal offsets of arterials which utilized the cycle by cycle green usage reports from ATMS. Mitigating impacts of "early return to green" on progression was the major concern in this paper. The authors applied Monte Carlo Simulation to generate distribution of time length of "early return to green" at each intersection. Based on these distributions, this paper calculated optimal offsets distribution and selected the most likely offsets. The proposed optimization method of offsets in this paper was offline approach.

Shoup and Bullock [26]proposed an offline fine-tune offsets method for an arterial based travel time data. The authors indicated this method mitigated impacts of "early return to green" and downstream queue on progression. This paper set initial offsets based on the end point of green phase and the link free flow travel time. After that the authors adjust the offsets according to average disruptive travel time in several cycles, i.e. stop delay of the first vehicle of a platoon. For this approach, it just considered one directional progression.

Liu and Hu et al, [27] utilized a data-driven approach to optimize offsets of an arterial. The deterministic delay model for two successive intersections was formed first. This paper mathematically formed a relationship between vehicle actuation and coordinated phase green time start point to calculate corresponding conditional distribution. Weight factors were used for two directions of the major street. The objective of this model was to decrease delay of a major direction without significantly increase delay of reverse direction. The proposed optimization method in this paper was also an offline approach.

Day, Haseman et al., [28]proposed two methods to evaluate and improve traffic progression for a corridor. The first method was Purdue Coordination Diagram (PCD). PCD used high resolution detector and signal phase data to generate a figure which combined arrival profiles and green time

profiles. Based on percentages of arrivals on green and total number of arrivals on green, PCD was performed to assess progression and enhance it by offset adjustment. For the second method, Bluetooth MAC address matching technology was used to re-identify vehicles for travel time assessment. The proposed approach was considered as equivalent to optimization for offsets in their paper and is offline.

Gettman, Head, et al, [29] proposed two real-time offset adjustment algorithms, Distributed Offset Adjustment (DOA) and Network Offset Adjustment (NOA), which were actual algorithms in ACS-lite. DOA just considered adjusting the offset between a pair of upstream and downstream intersections individually. NOA adjusted a group of offsets for a highway corridor. DOA and NOA both used incremental step sizes to adjust offsets, such as 2, 4 or 6 seconds. Captured flow, i.e. expectation of arrival on green, was the objective function. The algorithms presented in their paper were online approach, but they were responsive.

Abbas, Bullock et al., [30] proposed an algorithm for real-time offset adjustment for actuated traffic signal coordination system. The key concept of this algorithm was that the smaller difference between distributions of advanced detector occupancy and actuation profiles led to better progression based on diffusion theory of a previous study. This paper calculated skewness of absolute differences between detector occupancy and actuation profiles which was used as a criterion of applying the proposed algorithm. The proposed algorithm adjusted offsets according to move green time in order to cover higher occupancy summation. The proposed algorithm in their paper was responsive, as well.

2.4 Summary

In this section, eight pioneer ICM sites selected by U.S. DOT were reviewed. The available results of Stage 2 modeling demonstrated significant benefits of ICM strategies. In addition to these eight pioneer sites, other states also conducted ICM projects to improve their transportation systems.

Based on the literature review, with respect to ICM optimization models, optimization traffic signals on diversion arterials is a critical part for the proposed ICM strategies. Most reviewed literature related to ICM proposed methods to optimize traffic signals on diversion arterials and only one used online approach in a responsive manner. The literature reviews revealed that the implementation of real-time signal optimization for the proposed ICM strategies was not well studied. Besides, according to the literature review, most offset optimization algorithms and/or models are offline. Although algorithms proposed by some researchers were online approaches, they were responsive methods. So, an offset tuning algorithm with real-time and proactive control features will need to be proposed to supplement for ICM strategies and to advanced traffic signal control systems.

3. SYSTEM ARCHITECTURE AND BASELINE TRAFFIC SIMULATION MODEL DEVELOPMENT

3.1 System Architecture

In this subsection, the Integrated Diversion Traffic Management Strategies (IDTMS) system architecture is discussed in detail. The IDTMS system is developed based on the state wide ITS architecture. The IDTMS system framework, as shown in Figure 2, integrates existing traffic signal hardware, traffic management software, Dynamic Message Sign (DMS), and traffic surveillance components with our proposed diversion optimization model and algorithm.

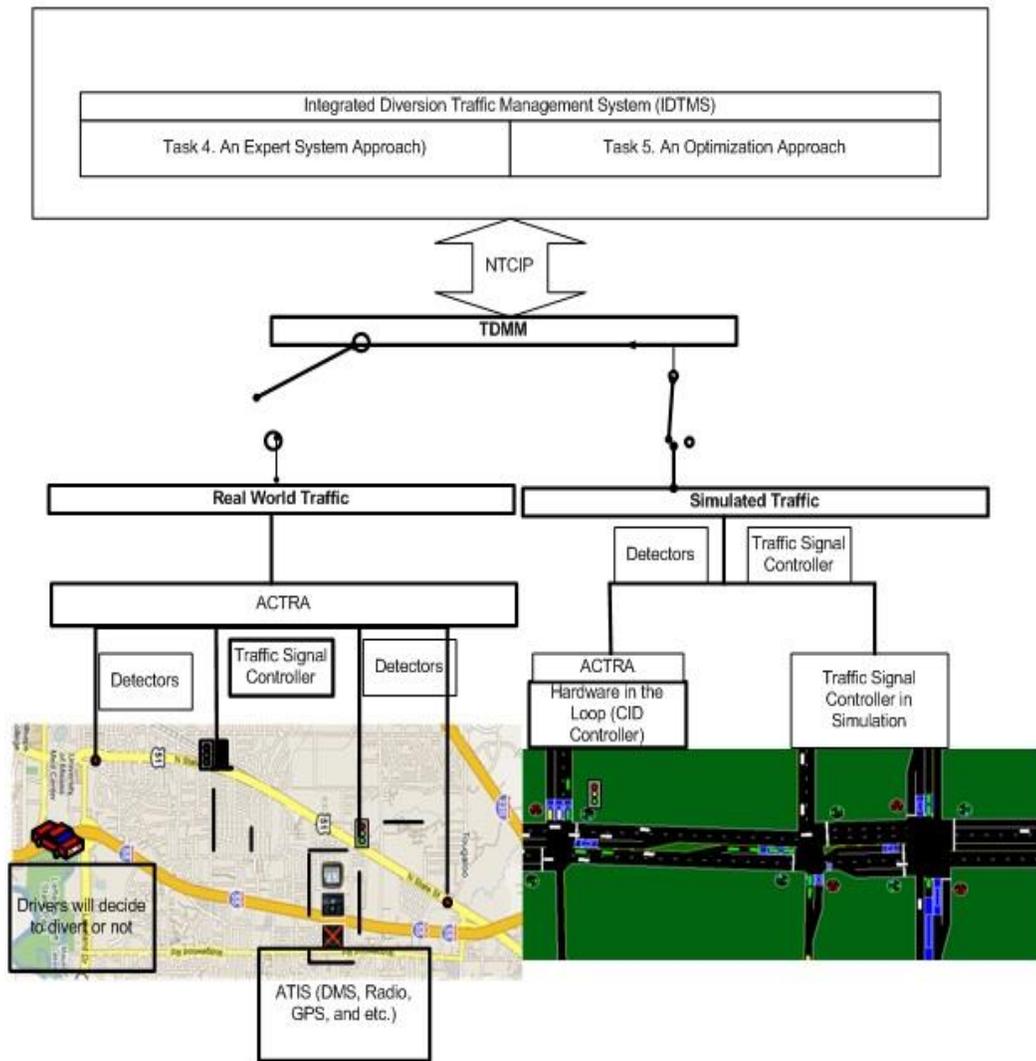


Figure 2 IDTMS Framework

The framework of IDTMS has three key modules, IDTMS optimization module, simulated traffic module, and real world traffic module. The optimization module includes an expert system and an optimization system for traffic diversion. The IDTMS optimization module is developed based on existing ITS architecture in the U.S. It is designed to input/output data using National Transportation Communications for ITS Protocol (NTCIP). The simulation module is configured

to evaluate the proposed IDTMS optimization module. If the diversion optimization model and algorithm are proved effective in reducing delays by simulation, the IDTMS system is also expected to be effective when it is applied to real world traffic. Furthermore, the simulation also provides a tested to develop IDTMS. It would be cost-effective to receive the input from simulated traffic for system development, debugging and testing, the effectiveness are evaluated through simulation. Once replaces the simulated input data from field data, the system can be further field test and fine turned.

The IDTMS optimization module is displayed on the upper of Figure 2. On the left hand side, the expert system is a simple rule-based model developed to decide the optimum traffic signal timing plan for coordinated intersections on the selected detour path in real time. The rules should be easy to follow by engineers and fast to execute. Therefore, the rules are will be formed as a matrix of lookup table and will be predefined by working together with MDOT. From the lookup table, a set of traffic status indicators, such as diversion volume, freeway congestion level, reserved capacity, and time of day are used as index to find out the traffic signal timing planes that are preconfigured into the system. The timing plans are optimized offline to accommodate different traffic patterns and different diversion volumes. The engineers will manually switch to the preconfigured plans. Although manual switch is the preferred method under MDOT existing hardware, software and the initial phase, the switch could be updated automatically through ACTRAL sever. In this case, the NTCIP interface between the expert model and the ACTRA is used to input the traffic detection data and traffic signal data to the expert system and output the selection of a preinstalled traffic signal timing plan to the ACTRA.

The proposed optimization models are shown at the right hand side of the upper proportion of Figure 2. The optimization system includes two separated models, first, the diversion optimization model and algorithm which optimally control integrated corridor traffic in real time. Second, traffic signal offsets among the coordinated intersections on the diversion route are periodically updated to adapt diversion volume variations. The optimization system would use NTCIP as interface to get surveillance data from either traffic simulation or real world traffic management system. The fine-tuned traffic signal offsets would also be outputted to the ACTRA by NTCIP. The expert system and the optimization system can be used in conjunction with one another which depends on the actual situation.

Traffic simulation module is shown on the right side of Figure 2. The research team develops a CORSIM simulation model to replicate the I-55 diversion scenarios. Detailed traffic network information, including geometric features, traffic flow, traffic signals, traffic signs, were coded into the CORSIM network. CORSIM RTE was developed to integrate the optimization system into the simulation. In the simulation, the intersections on diversion route could be either controlled by the ACTRA through Hardware-in-the-Loop, or by the internal traffic signal logic within the simulation, or by both of them. ACTRA and the simulation would access the traffic detections data and traffic signal controls data and send the surveillance data to the IDTMS optimization module by NTCIP interface. The IDTMS optimization module would utilize the surveillance data and then send updated traffic signal offsets back to the simulation.

In addition to CORSIM, the project team also explored ETFOMM to replace CORSIM. ETFOMM, or the Enhanced Transportation Flow Open-source Microscopic Model, was sponsored by US DOT and the project was finished in February 2014. ETFOMM is based on published CORSIM documents with model computing technology and some simulation modeling updates.

Real world traffic module was shown on the left side of Figure 2. MDOT's existing traffic management system including detectors, traffic signal controllers, DMS, were utilized by the ACTRA to get detection data and traffic signal timing plan data. When there is congestion on I-55, the freeway traffic volume, diversion route traffic volume, traffic signal timing plans, etc. will be collected into the ACTRA in real time. Then the ACTRA will communicate with IDTMS optimization module through NTCIP interface. The updated traffic signal offsets will be sent back to the ACTRA from the IDTMS optimization module. The congestion and/or incident information will be displayed on the DMS or other advanced traffic information system service (ATIS) devices, such as radio, GPS, and mobile phone, etc. The IDTMS optimization module would also dynamically adjust the traffic signal offsets based on real-time diversion traffic volume.

3.2 Base Line Traffic Simulation Model Development

In this subsection, a traffic simulation model for the study corridor was built in CORSIM. ETFOMM could use the exact CORSIM data as input. In the study area, I-55 is the major freeway in the north/south direction of the study network. State St. is the major arterial parallel with I-55. Other major roads within the study network include Frontage Rd, County Line Rd, Beasley Rd, Briarwood Dr, Cedars of Lebanon Rd, Old Canton Rd, Northside Dr, Meadowbrook Rd, Lakeland Dr, Woodrow Wilson Ave, and Fortification St, etc. To build the CORSIM simulation model for the study corridor, critical traffic data, such as geometry data, traffic volume, and existing field signal timing plans, etc., are collected. To provide a good preparation for future implementation of the proposed model and algorithm of this project, the research team also performed an inventory check for existing traffic signal control equipment on State St. The details of data collection, the equipment inventory check, and the building of the simulation model are illustrated in the following subsections.

3.2.1 Data Collection

In this research, to establish the baseline scenario, the research team has conducted a comprehensive traffic study to collect traffic data to establish the simulation model and parameters to calibrate the model. In this subsection, the procedures for collecting traffic volumes, time of day factors, discharge headway, travel time and free flow speed of State St and traffic volumes of I-55 are described in detail. The complete collected data are listed in Appendix.

3.2.1.1 Traffic Volume Study for State St

The research team conducted a traffic volume study on State St/U.S. Highway 51 in North Jackson, MS in 2010. Traffic volumes were manually and mechanically counted at 13 locations on State St between Pascagoula St & State St and W County Line Rd & State St. The specific locations are listed in Table 1. Traffic volume data were collected in morning and afternoon peaks. Since funding and other resources limit, not all locations were collected data for both morning and afternoon rush hours. To obtain complete data, the research team used collected data of some representative intersections to estimate traffic volume at other locations which were not counted.

Table 1 Traffic Volume Study Locations

No.	Location	No.	Location
1	Pascagoula St & State St	8	Old Canton Rd & State St
2	Pearl St & State St	9	Meadowbrook Rd & State St
3	Capitol St & State St	10	Northside Drive & State S
4	Amite St & State St	11	Briarwood Dr& State St
5	High St & State St	12	Beasley Rd & State St
6	Fortification St & State St	13	W County Line Rd & State St
7	Woodrow Wilson Ave & State St		

Two factors were calculated to estimate traffic volume. Those factors were called *Downtown Factor* and *Briarwood Factor*. The *Downtown Factor* was used for estimating traffic volume of intersections in downtown areas. It was applied to the following intersections: Pascagoula St & State St, Pearl St & State St, Capitol St & State St, Amite St & State St, High St & State St, Fortification St & State St and Woodrow Wilson Ave & State St. The *Briarwood Factor* was applied to the intersections nearby Old Canton Rd including Meadowbrook Rd & State St, Northside Dr& State St, Briarwood Dr& State St, and Beasley Rd & State St.

1) Downtown Factor

Downtown Factor was calculated using southbound left and through traffic’s peak hour volume at Pascagoula St & State St and the northbound through traffic’s peak hour volume at Capitol St & State St. *Downtown factor* has two parts, outbound factor and inbound factor. The southbound approach at Pascagoula St & State St is considered as the outbound approach. The outbound factor is calculated by dividing the AM peak hour volume with the PM peak hour volume for the through and left movement, respectively and then averaged the results. The same method was applied to calculate the inbound factor. For the inbound factor, since the only peak hour data collected at Capitol St & State St is the northbound through traffic, just the northbound through traffic is used to compute the inbound factor. The specific calculation procedures are provided as the following sample. In this section, the estimated data in the tables is highlighted in blue.

Table 2 AM Peak Hour Flow Rate of Pascagoula St & State St

Pascagoula St Turning Movement	N			S (Out)			E		
	L (None)	T (In)	R (Out)	L	T	R (None)	L (In)	T (Out)	R (Out)
# of Lanes		1	1 TR	2	2		1 TL	1	1 TR
AM Critical Vol. (VPH)		1362	92	184	268		411	531	92
PM Critical Vol. (VPH)		464	172	508	380		140	996	172

Note: (Inbound – In, Outbound – Out, Eastbound – EB, Westbound – WB, Southbound – SB, Northbound – NB, Left Turn – L, Through – T, Right Turn – R)

Calculation procedures:

Outbound factor:

$$AM\ SB\ L / PM\ SB\ L = 184 / 508 = 0.362$$

$$AM\ SB\ T / PM\ SB\ T = 268 / 380 = 0.705$$

$$Outbound\ factor = (0.362 + 0.705) / 2 = 0.5335$$

Inbound factor:

$$Inbound\ factor = AM\ NB\ T / PM\ NB\ T = 1268 / 432 = 2.935$$

High Street is a two-way street in the downtown area of Jackson which is an example about using *downtown factor* to estimate traffic volumes. The estimated and collected High St & State St's peak hour volume is shown in Table 3.

Table 3 Peak Hour Flow Rate of High St & State St

Approach	N			S			W (In)			E		
Device	Manual Counter			Manual Counter			Manual Counter			Manual Counter		
Turning Movement	L (In)	T (In)	R (Out)	L (Out)	T (In)	R (In)	L	T	R	L (In)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
AM Critical Vol. (VPH)	188	1855	115	83	2043	211	481	822	434	693	235	223
PM Critical Vol. (VPH)	64	632	216	156	696	72	164	280	148	236	440	76

In Table 3, PM peak hour flow rate is collected by manual counter. The peak fifteen minute volume occurred in the interval of 5:10 pm – 5:25 pm. AM peak hour flow rate is estimated by multiplying PM peak hour flow rate with the *downtown factor*. For example,

$$\text{(Inbound) AM NB-T} = 632 \times 2.935 = 1855$$

$$\text{(Outbound) AM SB-L} = 156 \times 0.5335 = 83$$

The flow rates of turning movement in AM/PM peak hours are displayed in Figure 3 and Figure 4, respectively. All four approaches are inbound because all of them carry more traffic in the morning than the evening.

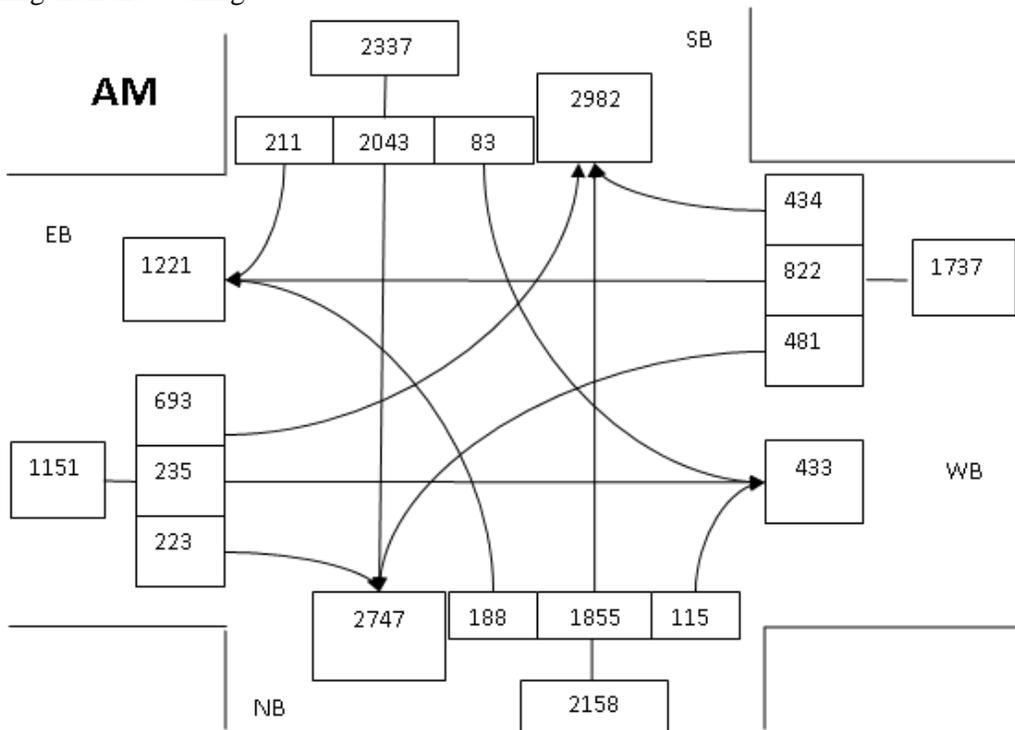


Figure 3 AM Peak Flow Rates of Turning Movements in High St & State St

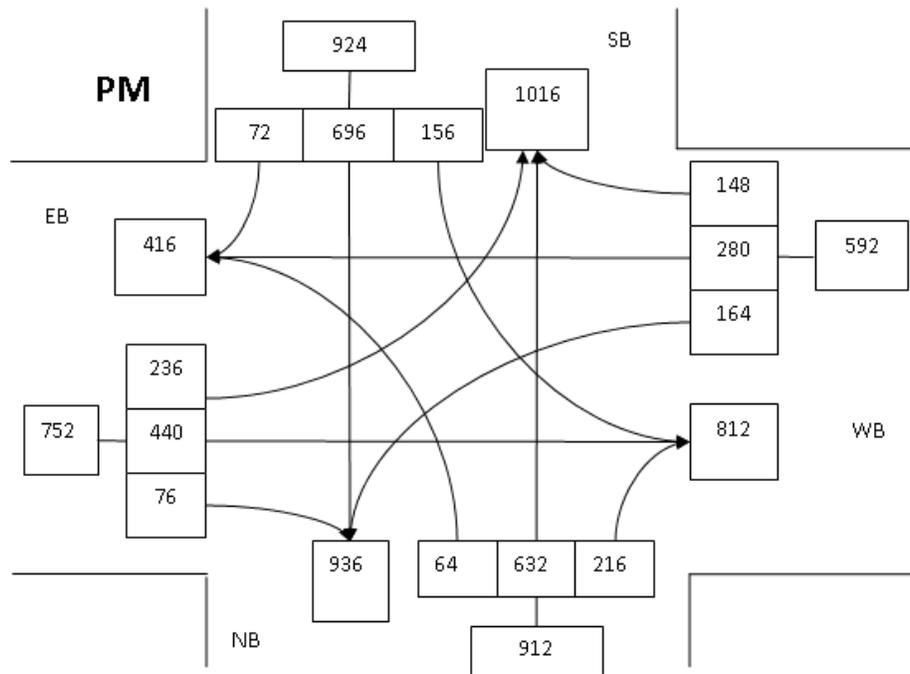


Figure 4 PM Peak Flow Rates of Turning Movements in High St & State St

2) Briarwood Factor

Briarwood Factor is calculated using AM/PM peak hour traffic volume data at Briarwood Dr & State St. This intersection has similar traffic patterns as other intersections which locate outside of the downtown area. *Briarwood factor* is used to estimate the PM peak hour volume at Northside Dr & State St and Meadowbrook Rd & State St as well as the AM peak hour volume at Beasley Rd & State St. *Briarwood factor* is calculated using the similar method as the downtown factor calculation. Specific calculation procedures are shown as following.

Table 4 Peak Hour Flow Rates of Briarwood Dr & State St

Approach	N (Out)			S (In)			W		
	L (None)	T	R	L	T	R (None)	L (In)	T (None)	R (Out)
Device	Radar			NC200			Manual Counter		NC200
# of Lanes		2	1	1	2		2		1
AM Critical Vol. (VPH)		476	152	112	564		153		92
PM Critical Vol. (VPH)		700	240	84	420		114		112

Note: This intersection is a T intersection. Some turning movements of each approach do not exist.

Briarwood factor also has two parts, outbound factor and inbound factor. Briarwood northbound is considered as the critical approach.

$$\text{Outbound factor} = \text{AM NB Traffic} / \text{PM NB Traffic} = (476+152) / (700+240) = 0.668$$

$$\text{Inbound Factor} = \text{AM SB T} / \text{PM SB T} = 564 / 420 = 1.34$$

Meadowbrook Rd is a two-way street. The Meadowbrook Rd & State St intersection is severed as an example about using *briarwood factor* to estimate traffic volumes. Meadowbrook Rd & State St's peak hour flow rate is shown in Table 5.

Table 5 Peak Hour Flow Rates of Meadowbrook Rd & State St

Approach	N (Out)			S			W			E		
Device	Manual Counter			Manual Counter			Manual Counter			Manual Counter		
Turning Movement	L	T	R	L (Out)	T (In)	R (In)	L (In)	T (In)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
AM Critical Vol. (VPH)	28	196	68	116	444	120	124	204	48	96	244	96
PM Critical Vol. (VPH)	42	293	102	174	331	90	93	152	72	144	365	72

Manual counters are used for collecting this intersection's traffic volume data in the AM peak hours. The peak fifteen-minute counts occurred at 7:45 am — 8:00 am. The AM peak hour flow rate is calculated based on the peak fifteen-minute counts. The PM peak hour flow rates are estimated by dividing AM peak hour flow rates with *briarwood factor*. For example,

$$\begin{aligned} \text{(Inbound) PM SB-T} &= 444 / 1.34 = 331 \\ \text{(Outbound) PM NB-L} &= 28 / 0.668 = 42 \end{aligned}$$

The flow rates of turning movements in AM/PM peak hours are displayed in Figure 5 and Figure 6, respectively, in which SB and WB are referred to as inbound, while NB and EB are referred to as to outbound.

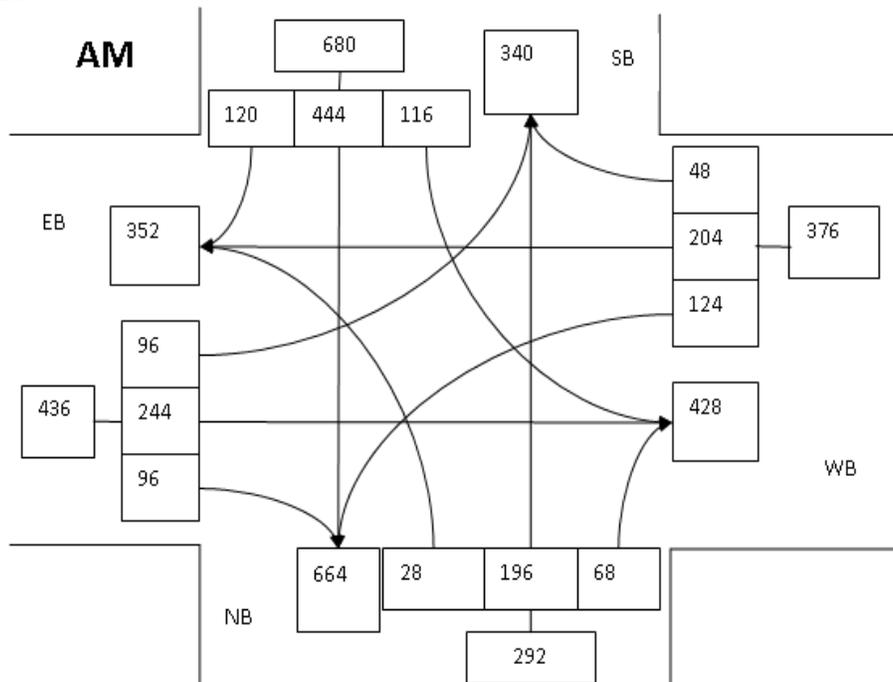


Figure 5 AM Peak Hour Flow Rates of Turning Movements in Meadowbrook Rd & State St

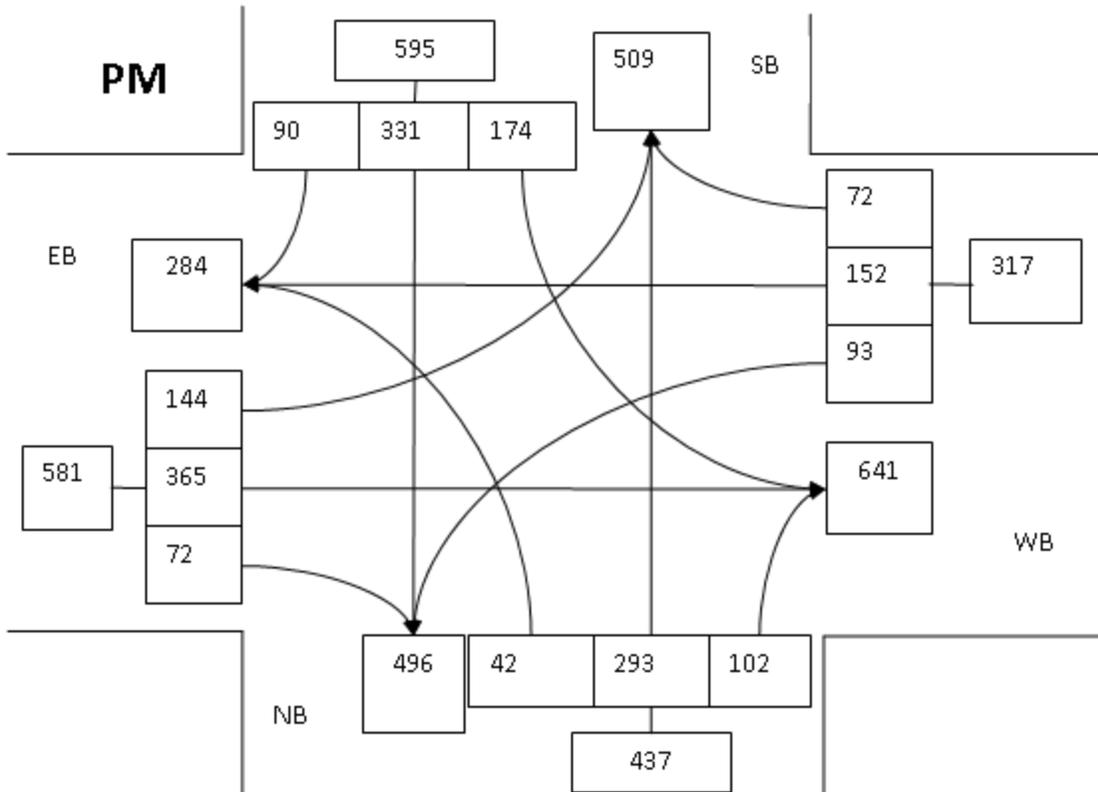


Figure 6 PM Peak Flow Rates of Turning Movements in Meadowbrook Rd & State St

The complete traffic volume data of these 13 intersections are listed in the Appendix. The entire computation procedures of these intersections' estimated traffic volume data are also included.

3.2.1.2 Volume Variations of State St

Hourly variations of traffic volume are analyzed in this section. Traffic counts from radar and NC 200 are analyzed because these data are collected over a long period (from several hours to several days). Six intersections, including Amite St & State St (NB), Capital St & State St (NB), Pascagoula St & State St (SB), Pearl St & State St (NB), Briarwood Drive & State St, and Old Canton Rd & State St, deployed radar and NC 200 to collect data. Two aspects of variations, time of day and peak hour volumes, are studied. The following is an example about the variation study for Pascagoula St/State St (SB).

Traffic volume data of SB at Pascagoula St & State St is collected by Radar detectors. Hourly volumes and time of day of SB are provided in Table 6.

Table 6 Hourly Volumes and Time of Day of SB in Pascagoula St/State St Intersection

Date	Time	Volume	Time of Day (%)
Oct 12 nd 2010	17:00-18:00	832	34.7
	18:00-19:00	340	14.2
	19:00-20:00	229	9.6
	20:00-21:00	176	7.3
	21:00-22:00	161	6.7
	22:00-23:00	148	6.2

	23:00-24:00	62	2.6
Oct 13 th 2010	0:00-1:00	26	1.1
	1:00-2:00	15	0.6
	2:00-3:00	6	0.3
	3:00-4:00	14	0.6
	4:00-5:00	9	0.4
	5:00-6:00	54	2.3
	6:00-7:00	84	3.5
	7:00-8:00	238	9.9
Total	15 hours	2394	100

Figure 7 showed variations of fifteen-minute traffic volume of Pascagoula St & State St Southbound. We found that PM peak hours have much heavier traffic than AM rush hours. There is more through traffic than left turn traffic in AM rush hours which is opposite to that of PM peak hours.

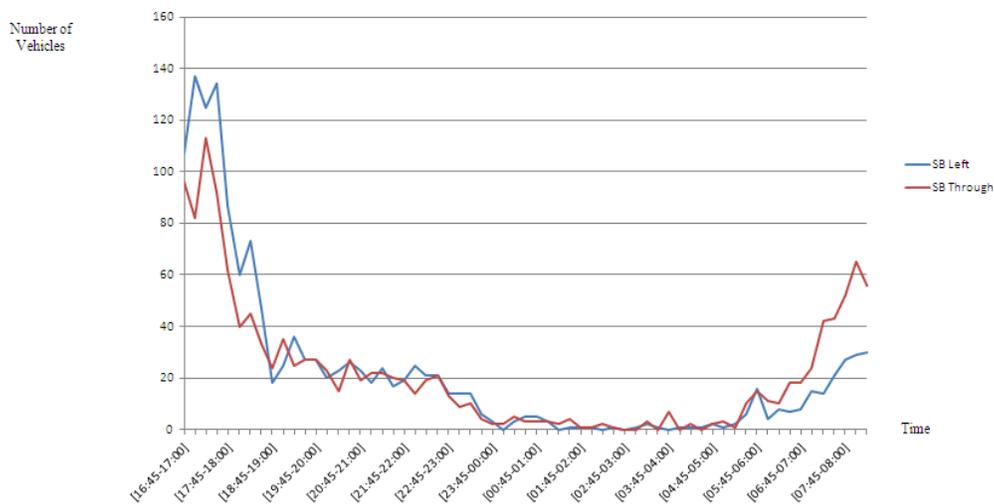


Figure 7 Variations of SB Traffic in Pascagoula St & State St

The complete contents of the volume variation study for other intersections are provided in the Appendix. According to results of the traffic volume variation study, it is found that, for the selected intersections of State St, AM peak was happened around 7:30 am - 8:30 am and PM peak was occurred around 4:45 pm - 5:30 pm.

3.2.1.3 Travel Time Study for State St

Global Position System (GPS) is used in the travel time study. Travel time data between two consecutive intersections are collected by directions. The average speed is calculated based on the travel time data. Variations of travel time and average speed between two neighboring intersections, in terms of directions, AM/PM change, are studied. Standard deviation of travel time and average speed are computed. The following is an example about the travel time study for the section from Pascagoula Street & State St to Pearl Street & State St.

Pascagoula St & State St and Pearl St & State St are in the downtown area. Table 7 and Table 8 show travel time and average speed between these two intersections. In Table 7, AM's travel time was longer than PM's travel time, which means that more traffic traveled from Pascagoula St to Pearl St in the morning.

Table 7 Travel Time and Average Speed from Pascagoula St & State St to Pearl St & State St

Pascagoula St to Pearl St									
Morning					Afternoon				
Trip	Time	Travel Distance (FT)	Travel Time (S)	Speed (MPH)	Trip	Time	Travel Distance (FT)	Travel Time (S)	Speed (MPH)
1	7:45	342	10	23	1	16:48	350	8	30
2	8:27	340	9	26	2	17:57	341	8	29
3	7:30	353	10	24					
MEAN			9.7	24.3	MEAN			8	29.5
STANDARD DEVIATION			0.58	1.25	STANDARD DEVIATION			0.00	0.54

The traffic pattern presented in Table 8 is opposite to the traffic pattern in Table 7. More traffic traveled from Pearl St & State St to Pascagoula St & State St in the afternoon than in the morning. The reasons are that travel times in the afternoon are longer than in the morning and travel speeds in the afternoon are much lower than in the morning. This trend is more obvious than that of north direction. It can be concluded that State St from Pascagoula St to Pearl St carries inbound traffic in morning, while it carries outbound traffic in the opposite direction in afternoon. In this study, the inbound traffic and outbound traffic are referred as to traffic enters the down town area and traffic leaves the downtown area, respectively. The travel time and travel speed data of other intersections are listed in the Appendix.

Table 8 Travel Time and Average Speed from Pearl St & State St to Pascagoula St & State St

Pearl St to Pascagoula St									
Morning					Afternoon				
Trip	Time	Travel Distance (FT)	Travel Time (S)	Speed (MPH)	Trip	Time	Travel Distance (FT)	Travel Time (S)	Speed (MPH)
1	7:43	347	8	30	1	16:44	321	16	14
2	8:25	364	8	31	2	17:55	345	12	20
MEAN			8	30.5	MEAN			14	17
STANDARD DEVIATION			0.00	1.02	STANDARD DEVIATION			2.83	4.19

3.2.1.4 Discharge Headway Measurement on State St

The discharge headway of an approach of an intersection is found by watching the real time traffic along State Street via video streams. ITS group of MDOT developed a website (MDOTTRAFFIC: <http://www.mdottraffic.com/>) which provides real time traffic video streams in Mississippi. The research team watches the live video streams of State St to collect discharge headway data. Due to lack of sight distance, the only intersection that under surveillance along North State Street is Fortification Street & State St. The northbound through approach and southbound through and through/right approaches are observed during the morning peak (7:30-8:10 a.m.) and the afternoon peak (4:30-5:15 p.m.) over three days. The data collected are

analyzed and graphed. Average discharge headway is calculated by averaging twenty groups' data. Since discharge headway becomes to be stable after the study intersection discharged five vehicles, the first five samples in each group are dropped. Each group's average discharge headway is calculated by using the headways from 6th car to the end of the queue. The average discharge headway of 2.346 seconds is found along Fortification Street (NB and SB). The averages of each group's data are shown in Table 9 as well as the average of all groups' data, which is used as the discharge headway for the intersection.

Table 9 Discharge Headway of Fortification St/State St

Fortification Street Discharge Headway Averages					
NB T		SB T		SB T/R	
Group Index	Average Headway (S)	Group Index	Average Headway (S)	Group Index	Average Headway (S)
1	3.170	1	2.050	1	2.680
2	2.583	2	2.560	2	2.166
3	2.000	3	2.509	3	1.938
4	2.850	4	2.088	4	2.090
5	2.077	5	1.962		
6	3.142	6	2.274		
7	2.420	7	2.146		
8	1.957				
9	2.256				
TOTAL AVERAGE (S)			2.346		

3.2.1.5 Free Flow Speed Study for State St

Speed data collected by Radar and NC 200 are used to measure Free Flow Speed (FFS). FFS of an intersection is considered as 85th percentile of speed data collected in the field. The research team assessed FFS for 6 intersections, including Amite St & State St (NB), Capital St & State St (NB), Pascagoula St & State St (SB), Pearl St & State St (NB), Briarwood Drive & State St, and Old Canton Rd & State St. Amite St & State St and Pearl St & State St are two examples for FFS obtained from NC 200 and Radar, respectively. For other intersections, please refer Appendix.

1) Amite St & State St (NB)

The Amite St at State St intersection is collected speed data by NC 200. Three pieces of NC 200 are placed in each of three lanes of NB in this intersection. One piece is used for left lane and the other two pieces are utilized for two through lanes. Highway Data Management (HDM) is the software developed specially for NC 200 data extraction and programming predefined settings by its manufacture. HDM can generate 85th percentile speed based on collected data. 85th percentile speed of three lanes are offered in Table 10 one by one.

Table 10 85th Percentile Speed of Each Lane in NB of Amite St/State St Intersection

	Left Lane	Through Lane 1	Through Lane 2
85 th percentile speed (mph)	23.79	35.52	33.77

In Table 10, left turn vehicles' speed is less than that of through vehicles which are consistent with reality. Speed of left turn vehicles of an approach cannot represent FFS of that approach because left turn drivers need to slow down and make a left turn even if there is nearly no traffic on the intersection. The average of 85th percentile speed in two through lanes, 35 mph, is used as FFS of this approach.

2) Pearl St & State St (NB)

For this intersection, the speed data is collected data by Radar and is available for analysis. As discussed before, speed data of through traffic is used for estimating 85th percentile speed. Figure 8 shows cumulative percentage of speed distribution of NB. According to the figure, 85th percentile speed is round to 30 mph which is used as FFS of this approach.

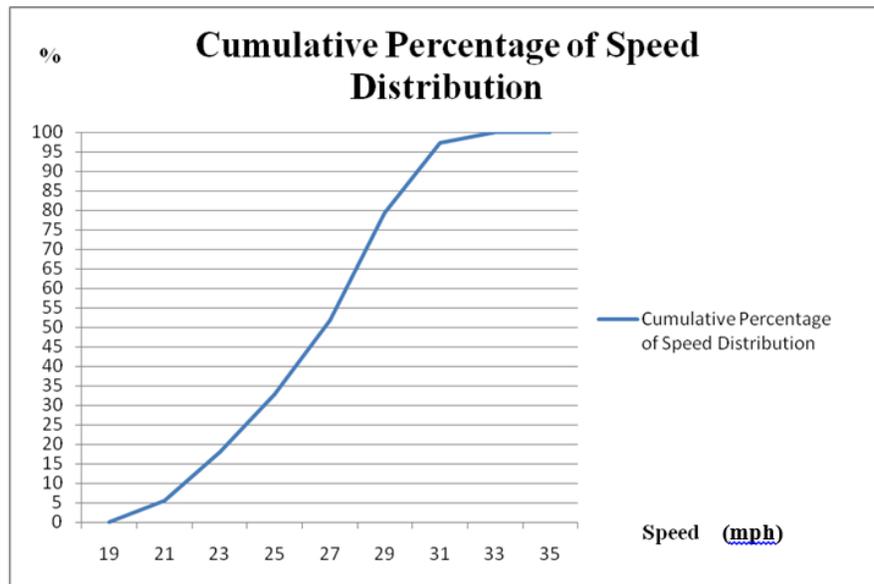


Figure 8 Cumulative Percentage of Speed Distribution of Pearl St&State St NB

3.2.1.6 Traffic Volume Collection for Freeway and Ramp

The research team collected traffic volume data on freeway and on/off ramps of the study corridor by watching surveillance cameras. A total of 20 surveillance cameras are used to collect data at North I-55 on/off ramps. Table 11 shows locations of these cameras.

Table 11 Surveillance Cameras Locations

Number	Location
1	I-55 South of County Line Rd north towards Canton
2	I-55 South of County Line Rd south towards Downtown Jackson
3	County Line Rd East at I-55 towards NorthPark Mall
4	County Line Rd West at I-55 towards Hwy 51
5	I-55N North of Briarwood Dr towards County Line Rd/Madison
6	I-55N South of Briarwood Dr towards downtown Jackson / the Stack
7	Briarwood Dr east of I-55 towards Ridgewood Rd

8	Briarwood Dr west of I-55 towards North State Street.
9	I-55N North at Northside Dr towards Memphis / Natchez Trace
10	I-55N South at Northside Dr towards downtown Jackson.
11	Northside Dr West at I-55 towards N State Street
12	I-55 North of Meadowbrook Rd towards County Line Rd / Memphis
13	I-55 South of Meadowbrook Rd towards Downtown Jackson
14	Meadowbrook Rd East of I-55 towards Ridgewood Rd
15	Meadowbrook Rd West of I-55 towards North State Street
16	I-55 north of the Lakeland Interchange towards Memphis
17	Lakeland Drive North at I-55 - St. Dominic towards Lakeland / I-55 Overpass.
18	Lakeland Dr(MS-25) north at I-55 North towards Flowood
19	I-55 south of the Lakeland Interchange towards Woodrow Wilson / downtown Jackson.
20	Lakeland Dr South at Curran St towards I-55 interchange

PM peak traffic volumes of I-55 are manually counted by watching these cameras. The research team counts the traffic from 4:45 pm to 6:15 pm for two days at each location and gets the peak hour volume by averaging the highest hourly volume in two days. The results of the observation are summarized in Table 12.

Table 12 Surveillance Cameras Locations

Location	Total PM Peak Hour Volume (VPH)	# of Lanes	PM Peak Hour Volume Per Lane (VPH)
I-55 South at County Line Rd towards Downtown Jackson	3838	3	1279
E County Line Rd On-Ramp to I-55 South	591	1	591
I-55 South of E County Line Rd Before Two On-Ramps	2152	3	717
W County Line Rd On-Ramp to I-55 South	547	1	547
I-55 North of Meadowbrook Rd	6437	4	1609
I-55 South of Meadowbrook Rd	4593	4	1148
Meadowbrook Rd Eastbound at I-55	1459	3	486
I-55 South of Meadowbrook Rd On-Ramp	487	2	244
Old Canton Rd to West Frontage Rd Bridge	417	2	208
I-55 South of Briarwood Dr North Side	3630	3	1210
I-55 South of Briarwood Dr North Side On-Ramp	617	2	309
I-55 South of Briarwood Dr South Side	3174	4	794
I-55 South of Briarwood Dr South Side On-Ramp	288	1	288
I-55 South of NorthSideDr South toward downtown Jackson	2693	3	898
I-55 South of NorthSideDr South On-Ramp	1202	2	601
I-55 South of Lakeland Dr	2948	3	983

I-55 South of Lakeland Dr Off-Ramp to Frontage Rd South	1997	4	499
Lakeland Dr West to I-55 South On-Ramp	453	1	453
I-55 North of Lakeland Dr	3679	3	1226
I-55 North to Lakeland Dr East Off-Ramp	697	2	349
Lakeland Dr East Left Turn to Frontage Rd North	914	2	457
Lakeland Dr East Through Traffic	1603	3	534
Lakeland Dr West Through Traffic	1289	3	430
I-55 North to Lakeland Dr West Off-Ramp	67	1	67
Frontage Rd North Right Turn to Lakeland Dr East	339	1	339
Frontage Rd North Left and Through Traffic	89	3	30

3.2.2 Equipment Inventory Check

The research team performs the field inspection for traffic control devices at selected major intersections in the North Jackson area during April 23-28, 2011. A total of 47 intersections along State Street, Frontage Road, and other roads which have direct access to I-55 are selected. We checked inventories of existing traffic signal control devices and traffic detections and evaluate their working status. The complete intersections list is shown in Table 13.

Table 13 Intersections List

Intersection No.	Location	Intersection No.	Location
1	CountyLine Rd & Ridgewood Rd	25	State St & Amite St
2	CountyLine Rd & Frontage Rd	26	State St & Capital St
3	CountyLine Rd & State St	27	State St & Pearl St
4	State St & Beasley Rd	28	State St & Pascagoula St
5	State St & Briarwood Dr	29	Frontage Rd & Lakeland Dr
6	State St & Culley Dr	30	Frontage Rd & Eastover Dr
7	State St & Cedars Lebanon Rd	31	Frontage Rd & Meadowbrook Rd
8	State St & Woodway Dr	32	Frontage Rd & Northside Dr
9	State St & Broadmoor Dr	33	Frontage Rd & Canton Mart Rd
10	State St & Northside Dr	34	Frontage Rd & Briarwood Dr
11	State St & Meadowbrook Rd	35	Frontage Rd & Adkins Rd
12	State St & Ridgeway St	36	Briarwood Dr & County Cork Rd

13	State St & Duling Ave	37	Northside Dr & Manhattan Rd
14	State St & Old Canton Rd	38	Meadowbrook Rd & Manhattan Rd
15	State St & Medical Center	39	Meadowbrook Rd & Old Canton Rd
16	State St & Woodrow Wilson Ave	40	Lakeland Dr & UMMC Access Dr
17	State St & Riverside Dr	41	Lakeland Dr & N Curran Dr
18	State St & Oakwood St	42	Fortification St & Greymont St
19	State St & Pinehurst St	43	High St & North St
20	State St & Manship St	44	High St & Jefferson St
21	Fortification St & Jefferson St	45	High St & Monroe St
22	State St & Fortification St	46	High St & Greymont St
23	State St & High St	47	Jefferson St & Pascagoula St
24	State St & Mississippi St		

The locations of all the intersections are shown in Figure 9.

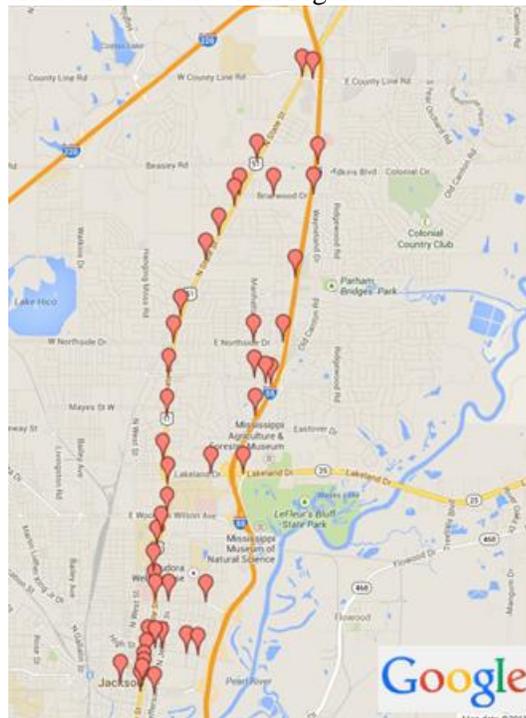


Figure 9 Intersections Locations

For each intersection, traffic detector specifications and status are summarized in the table below.

Table 14 Detector Information Check List

Intersection	Intersection's Name
Type	Loop / Video Detection
Operation	Pulse / Presence
Location	Distance from Stop Bar
Size	6' X 6' / 6' X 50'
Number	Detector's Number
Corresponding Phase	Corresponding Phase Number
Working Status	Good / Bad

There are 43 intersections with loop detections while two intersections are video detection. Video detections are being installed at two intersections when the research team do this inventories check. The intersections using video detection are listed in Table 15.

Table 15 Intersections with Video Detection

Number	Intersection	Note
2	E CountyLine Rd & Frontage Rd	Video Detection
39	Meadowbrook Rd & Old Canton Rd	Video Detection
20	State St & Manship St	Video Detection to be implemented
22	State St & Fortification St	Video Detection to be implemented

The research team inspects traffic signal devices in the traffic signal cabinet, including the conflict monitor, detector units, and the flasher. The check list for those devices is summarized in Table 16.

Table 16 Traffic Signal Devices Check List

Intersection	Intersection's Name
Conflict Monitor	Model
Conflict Monitor Working Status	Good / Bad
Detector Units	Carts and Units
Detector Units Working Status	Good / Bad
Flasher Working Status	Good / Bad
Traffic Signal Timing Plan	NEMA Phasing Diagram

Communications from detectors to intersections and from intersections to the master controller are inspected, as well. All the 47 intersections are using copper wire for communications from detector to intersection. 39 intersections were implemented fiber for communications from intersection to the master controller. The intersections with fiber are listed in Table 17.

Table 17 Intersections with Fiber

Intersection No.	Location	Intersection No.	Location
1	CountyLine Rd & Ridgewood Rd	21	Fortification St & Jefferson St
2	CountyLine Rd & Frontage Rd	22	State St & Fortification St
3	CountyLine Rd & State St	23	State St & High St
4	State St & Beasley Rd	24	State St & Mississippi St
5	State St & Briarwood Dr	25	State St & Amite St
6	State St & Culley Dr	26	State St & Capital St
7	State St & Cedars Lebanon Rd	27	State St & Pearl St
8	State St & Woodway Dr	28	State St & Pascagoula St
9	State St & Broadmoor Dr	29	Frontage Rd & Lakeland Dr
10	State St & Northside Dr	32	Frontage Rd & Northside Dr
11	State St & Meadowbrook Rd	37	Northside Dr & Manhattan Rd
12	State St & Ridgeway St	38	Meadowbrook Rd & Manhattan Rd
13	State St & Duling Ave	40	Lakeland Dr & UMMC Access Dr
14	State St & Old Canton Rd	41	Lakeland Dr & N Curran Dr
15	State St & Medical Center	43	High St & North St
16	State St & Woodrow Wilson Ave	44	High St & Jefferson St
17	State St & Riverside Dr	45	High St & Monroe St
18	State St & Oakwood St	46	High St & Greymont St
19	State St & Pinehurst St	47	Jefferson St & Pascagoula St
20	State St & Manship St		

The device status inspections indicated some problems at some intersections. These issues are summarized in Table 18.

Table 18 Intersections with Issues

#	Intersection	Notes
4	State St & Beasley Rd	There is an error with fiber network communication.
5	State St & Briarwood Dr	The 2nd detection channel didn't work.
6	State St & Culley Dr	Westbound detector units didn't work (on Recall).
7	State St & Cedars Lebanon Rd	2 channels were not working.
		No timing plan (no min green, max green, etc.).
8	State St & Woodway Dr	The 3rd detector unit didn't work (turned off)
		Loop for Φ8 was damaged (on Recall)
9	State St & Broadmoor Dr	Westbound loops didn't work (on Recall)
10	State St & Northside Dr	Φ3 and Φ7 were on Recall
12	State St & Ridgeway St	Fiber broken
		Φ 2 is on Max Recall

13	State St & Duling Ave	State St is on Max Recall
14	State St & Old Canton Rd	State St is on Max Recall
15	State St & Medical Center	No communication with ACTRA server
16	State St & Woodrow Wilson Ave	Flasher didn't work properly and a new controller was needed.
19	State St & Pinehurst St	State St is on Max Recall
		Φ2 and Φ6 were turned off
20	State St & Manship St	Φ6 loop didn't work. Φ2 A and B didn't work. Φ1 was turned off.
21	Fortification St & Jefferson St	Under reconstruction
		Loops for Φ1, Φ4, and Φ5 were working and others were broken.
23	State St & High St	Loops for Φ1 and Φ5 were working and others were broken.
26	State St & Capital St	No detection at this intersection. The controller was not pre-programmed.
36	Briarwood Dr & County Cork Rd	All approaches were based on recall. No call came in.
39	Meadowbrook Rd & Old Canton Rd	Video detection was not working properly.
42	Fortification St & Greymont St	Φ1 and Φ5 were based on recall. Southbound and Eastbound loops didn't work.

As listed in Table 18, 20 out of 47 intersections are having issues with detections, communications, etc., which means at least 43% intersections require additional maintenance in order to handle regular traffic. The repair needs to be complete before surge detour traffic diverted from I-55 to those intersections.

3.2.3 Simulation Model Building

Data collected from above activities provided us network traffic information and traffic control device information. Those information includes freeway/arterials traffic volumes and turning percentages, as well as variations with time of day. Those data are utilized in building the CORSIM simulation model for the study'. The built traffic network consists of 1,567 links and 931 nodes, in which 74 nodes are signalized intersections. The entire study corridor in CORSIM is shown in Figure 10. The constructed CORSIM simulation model is well calibrated according to collected traffic data in previous subsections.

Roadway data is provided by MDOT's GIS map. The detail tuning packet length, intersection dimensions, ramp configurations, number of lanes are manually observed from Google maps. Free flow speeds are concluded by combination of measured GPS data and speed limits.

The calibration is conducted as follows. First, traffic volumes are calibrated. CORSIM only requires volumes on entry nodes and turning percentages at intersections and freeway off ramps. The traffic volumes on freeway and major roads in the area, such as I-55, State St, Lakeland Dr, Frontage Rd, Meadowbrook Rd, Northside Dr, Beasley Rd, County Line Rd, Woodrow Wilson Ave, Fortification St, Pearl St, High St, and Pascagoula St, were calibrated based on the field data. The turning percentages at major intersections and ramps were fine-tuned in order to keep the traffic volume profile consistent on major roads. Then, travel times on major roads in simulation are compared with actual field data and google map data. The google map travel time on State St.,

which incorporates with real-time traffic information, is collected in the rush hour. The actual travel time on State St. is collected by GPS data of a probe vehicle. The comparison results are shown in Table 19.

Table 19 Travel Time Comparison of State St

	Location	Time (Second)					
		Simulation Data		Field Data		Google Map Data	
		NB	SB	NB	SB	NB	SB
1	Pascagoula St to Pearl St	48.3	39.4	8-10	8-16	28	24
2	Pearl St to Capital St	35.0	20.9	7-9	7-33	26	23
3	Capital St to Amite St	35.3	18.9	7-23	7-38	29	24
4	Amite St to High St	81.6	70.6	26-41	19-36	60	60
5	High St to Fortification St	105.6	98.5	44-54	41-58	120	120
6	Fortification St to Woodrow Wilson Ave	297.1	321.3	113-137	96-146	240	240
7	Woodrow Wilson Ave to Old Canton Rd	166.2	177.0	46-70	45-66	120	120
8	Old Canton Rd to Meadowbrook Rd	213.2	215.7	169-187	131-136	240	180
9	Meadowbrook Rd to Northside Dr	69.6	82.5	43-63	62-71	120	120
10	Northside Dr to Briarwood Dr	253.6	269.5	216-253	190-212	240	240
11	Briarwood Dr to Beasley Rd	71.9	59.5	32-33	43	60	56
12	Beasley Rd to County Line Road	163.2	173.7	126	-	120	180

From Table 19, we can see that links' travel time of State St in simulation is generally larger than actual travel time we collected in field. It is caused by the following reasons. First, we use the maximum flow rate, the product of 4 times maximum 15 minutes volume, of intersections on State St to build our simulation model which is heavier than actual peak hour volume. The purpose of this approach is that we want to see whether or not State St could adapt extra detour volume from I-55 in the worst case. Second, due to funding and resource limit, we only collect morning and/or afternoon traffic data at critical intersections on State St. We obtain traffic data of other time periods of these critical intersections by estimation. Third, we only have one probe vehicle to collect traffic time data, due to funding restraint. The vehicle may directly traverse an intersection or several close spaced intersections when it met green time. The collected travel time in this situation cannot represent actual travel time of these links. When compare simulation travel time to the Google travel time, they are close. However, there still exists some difference. The primary reason for the difference is the volatility of google map travel time. Google map measures real-time traffic. As a result, google map travel time is particularly impacted by traffic flow fluctuations. Based on our observations, google map travel time can differ by fifty percent for the same segment in two minutes. In our simulation, the traffic pattern is stable in the rush hour. There are no obvious traffic flow fluctuations in the simulation. In addition, the simulation averages the travel time for the whole rush hour which is more accurate than google map travel time, which we only collected five samples due to the limit of time and resource. Another reason

for the travel time difference is that google map only provides travel time measured in minutes when the travel time is more than one minute. This measurement is applicable for a long segment which costs more than 10 minutes driving. However, for the segments in this study, whose travel time ranging from a few seconds to five minutes, this measurement is too rough. It could cause the travel time difference up to 60 seconds. We may conduct a comprehensive travel time study on State St, if MDOT prefer and funding is available in the future. The completed CORSIM simulation network is shown in Figure 10.

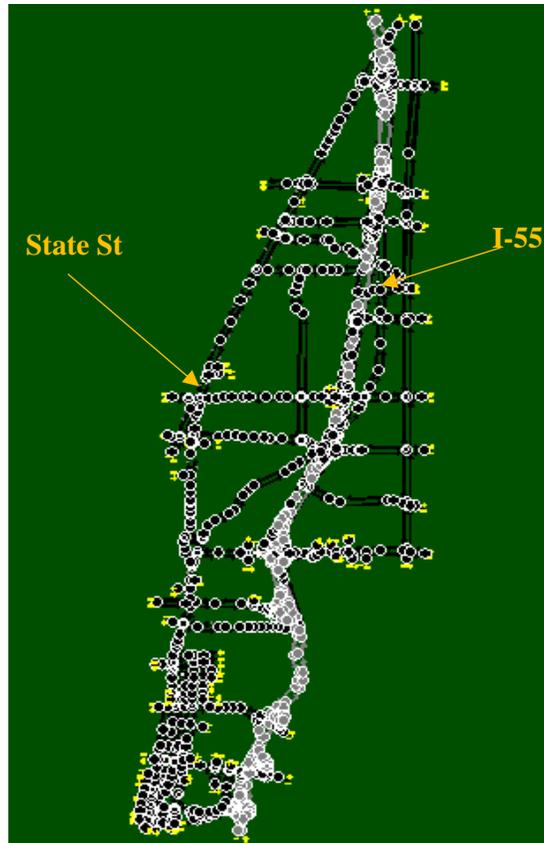


Figure 10 I-55 Corridor CORSIM Network

4. AN OPTIMIZATION APPROACH

In this section, the details of the optimization approach within the framework of system architecture described in the previous section are presented. Two approaches, an ICM optimization model to minimize freeway and Surface Street altogether and a proactively online offset tuning algorithm are proposed. The model and algorithm are also validated by case studies of the study ICM corridor.

4.1 An ICM Optimization Model

4.1.1 An ICM Optimization Model

An ICM optimization model from the research team's previous study[31] is applied in the system. The objective of this model is to minimize the total delay on the freeway and the diversion route. Specifically, the objective function of this model has three components: freeway delay, diversion route intersections' control delay, and detour extra travel time delay.

The objective function and constraints of the model are described as follows:

$$\text{Min} \left(D_F + D_D + \sum_{i=1}^n D_i \right) \quad (1)$$

S.T.

$$\sum_{p_i=1} (gt_{p_i} + Y_{p_i} + AR_{p_i}) = C_i \quad 1 \leq i \leq n \quad (2)$$

$$gt_{iag}^{\min} \leq gt_{iag} \leq gt_{iag}^{\max} \quad (3)$$

$$C_{i\min} \leq C_i \leq C_{i\max} \quad (4)$$

$$V_D = kV_F \quad (5)$$

Where,

- n - Number of intersections on the diversion route,
- i - The i^{th} intersection on an diversion road,
- p - Phase p of an intersection I ,
- D_F - Total freeway delay,
- D_D - Total diversion delay,
- D_i - Total intersections control delay on the diversion route,
- C_i - Cycle length of the i^{th} intersection in second,
- $C_{i\min}$ - Minimal cycle length of i^{th} intersection in second,
- $C_{i\max}$ - Maximal cycle length of i^{th} intersection in second,

- gt_{iag} - Green time for lane group g of approach a at intersection i in second,
- gt_{iag}^{min} - Minimal green time for a lane group g of approach a at intersection i in second,
- gt_{iag}^{max} - Maximal green time for lane group g of approach a at intersection i in second,
- V_D - Diversion volume,
- V_F - Freeway volume,
- k - Diversion rate.

There are three sets of decision variables in the model.

- $LDMS$ - DMS message level, $LDMS \in [1, 8]$. Each level is associated with a predefined diversion probability. The diversion volume guided by a certain DMS level is calculated according to the associated probability.
- C - The intersection cycle length of traffic controller.
- GT - The set of green times for all the studied intersections, gt_{iag} is the green time for lane group g of approach a at intersection i .
- O_i - Signal offset of the i^{th} intersection in second.

Equation (2) represented that the summation of green time, yellow time and all red time from all phases at an intersection i should be equal to the cycle length of the intersection i . Equation (3) showed the green time boundaries of i^{th} intersection, $1 \leq i \leq n$. Equation (4) limited the cycle length of i^{th} intersection. Equation (5) indicated that the diversion volume equaled to the freeway demand multiplied by the diversion rate.

This model is a nonlinear model with linear constraints and bounds constraints. MATLAB provides FMINCON function in the Optimization Toolbox for solving the problems[32]. The FMINCON is a gradient-based method which uses Sequential Quadratic Programming (SQP) algorithm with Hessian matrix for optimal solution search. The principal idea of SQP used in FMINCON is the Quadratic Programming (QP) sub-problem formulation based on a quadratic approximation of the Lagrangian function.

$$L(x, \lambda) = f(x) + \sum_{i=1}^m \lambda_i \cdot g_i(x) \quad (6)$$

Where $L(x, \lambda)$ is the Lagrange function. $f(x)$ is the objective function. λ is vector of the Lagrange multipliers and the $g(x)$ is the constraint functions.

The QP sub-problem of the Lagrange function $f(x)$ is described as follows and it can be resolved by a QP algorithm. The analytical procedures of the QP algorithm, which were initially provided in our previous study[31], are summarized by Equation (7) – (22).

$$\begin{aligned}
& \min_{d \in R^n} \frac{1}{2} d^T H_k d + \nabla f(x_k)^T d \\
& \nabla g_i(x_k)^T d + g_i(x_k) = 0, \quad i = 1, \dots, m_e \\
& \nabla g_i(x_k)^T d + g_i(x_k) \leq 0, \quad i = m_e + 1, \dots, m
\end{aligned} \tag{7}$$

The solution of QP problem provides the search directions. Search step length will be determined by a line search procedure to obtain a sufficient decrease in the merit function. The new solution point is then computed by Equation (8).

$$x_{k+1} = x_k + \alpha_k d_k \tag{8}$$

Where α_k is the step length parameter and the d_k is the search direction at step k .

At each major step, the Hessian of the Lagrangian function $L(x, \lambda)$ is approximated by positive definite quasi-newton method in Equation (9).

$$H_{k+1} = H_k + \frac{q_k q_k^T}{q_k^T s_k} - \frac{H_k^T s_k^T s_k H_k}{s_k^T H_k s_k} \tag{9}$$

Where:

$$s_k = x_{k+1} - x_k \tag{10}$$

$$q_k = \left[\nabla f(x_{k+1}) + \sum_{i=1}^m \lambda_i \nabla g_i(x_{k+1}) \right] - \left[\nabla f(x_k) + \sum_{i=1}^m \lambda_i \nabla g_i(x_k) \right] \tag{11}$$

In the SQP implementation, $q_k^T s_k$ is kept positive at each update to keep the Hessian positive definite, even though it might be positive indefinite at the solution point. A procedure repeatedly halves the most negative element of $q_k^T s_k$ until it is greater than or equal to a small negative tolerance. If $q_k^T s_k$ is still not positive after the above procedure, q_k will be modified by adding a vector v multiplied by a constant scalar w .

$$q_k = q_k + wv \tag{12}$$

Where,

$$v_i = \nabla g_i(x_{k+1}) \cdot g_i(x_{k+1}) - \nabla g_i(x_k) \cdot g_i(x_k), \text{ if } (q_k)_i w < 0 \text{ and } (q_k)_i (s_k)_i < 0$$

$$v_i = 0, \text{ otherwise}$$

The w will be systematically increased until $q_k^T s_k$ becomes positive. At the same time, a QP problem of search direction in the following form is solved at each major SQP step by active set method to provide search direction d for the QP sub-problem.

$$\begin{aligned}
& \min_{d \in R^n} \frac{1}{2} d^T H_k d + c^T d, \\
& A_i d = b_i, \quad i = 1, \dots, m_e, \\
& A_i d \leq b_i, \quad i = m_e + 1, \dots, m.
\end{aligned} \tag{13}$$

There are two phases in the solution procedure. A feasible solution is computed in the first phase and then the solution is iteratively updated towards convergence in the second phase. The active set A_k is an estimation of the active constraints on the constraint boundaries of the solution. d_k is formed based on A_k that is updated at each iteration k . A feasible subspace for d_k is formed based on Z_k which satisfies $A_k Z_k = 0$, which means Z_k is orthogonal to A_k . Z_k is formed from the last number of inactive constraints in the QR decomposition matrix of A_k .

d_k is sought from quadratic function of p by the projected gradient method of quadratic function.

$$d_k = Z_k p \tag{14}$$

Where p can be obtained by differentiating the following quadratic function and set it to zero.

$$q(p) = \frac{1}{2} p^T Z_k^T H Z_k p + c^T Z_k p \tag{15}$$

$$\frac{\partial q(p)}{\partial p} = \nabla q(p) = Z_k^T H Z_k p + Z_k^T c \tag{16}$$

$$Z_k^T H Z_k p = -Z_k^T c \tag{17}$$

Once d_k is obtained, the new solution point is computed by Equation (18):

$$x_{k+1} = x_k + \alpha d_k \tag{18}$$

Where α have only two choices for the quadratic nature of the objective function. When the constraints are in the active set d_k , if one unit along the d_k steps to the minimum of the function restricted to the null-space of active set without violation of the constraints, then this is the solution of QP. Otherwise, it will be calculated by Equation (19).

$$\alpha = \min_{i \in \{1, \dots, m\}} \left\{ \frac{-(A_i x_k - b_i)}{A_i d_k} \right\} \tag{19}$$

The solution to the QP problem of search direction produces a search direction vector d_k for QP sub-problem of objective function. A new iteration for the QP sub-problem will be formed if α_k is obtained from a merit function. The α_k produces sufficient decrease in the merit function.

$$\Psi(x) = f(x) + \sum_{i=1}^{m_e} r_i \cdot g_i(x) + \sum_{i=1}^{m_e} r_i \cdot \max[0, g_i(x)] \tag{20}$$

Where r_i is given by,

$$r_i = (r_{k+1})_i = \max_i \left\{ \lambda_i, \frac{(r_k)_i + \lambda_i}{2} \right\}, i = 1, \dots, m \quad (21)$$

The initial r_i is obtained by the Euclidean norm.

$$r_i = \frac{|\nabla f(x)|}{|\nabla g_i(x)|} \quad (22)$$

4.1.2 Case Study

In order to produce a scenario in which I-55 is seriously congested, a dummy work zone which blocks one lane of the three-lane of I-55 southbound is created in the simulation. The work zone is 500 *ft* in length and located on a freeway segment between Northside Drive and Meadowbrook Road. The project team also manually designed a diversion route for detour traffic from the freeway. Figure 11 shows the freeway mainline traffic and the diversion traffic on State St.

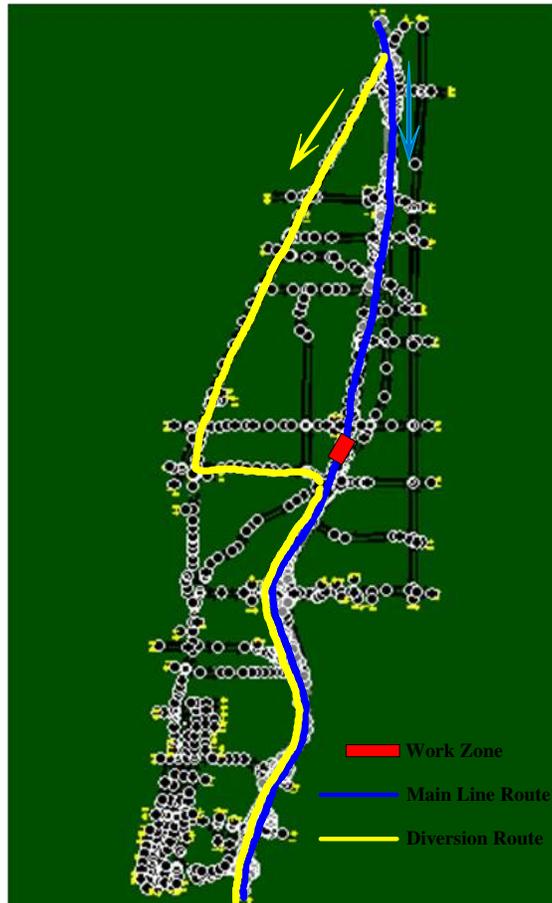


Figure 11 I-55 and Diversion Route Traffic

There are four intersections on the diversion route modeled in the IDTMS system. These intersections are: N State St & County Line Rd, N State St & Beasley Rd, N State St & E

Northside Dr, and N State St & Meadowbrook Rd. The traffic signals of intersections on diversion route are coordinated.

The intersections on the diversion route were programmed as fixed-time control to accommodate the proposed model. The traffic signal timing plans are originally extracted from MDOT’s ACTRA server and then are optimized using TRANSYT-7F. In addition, ACTRA system is installed and MDOT’s ACTRA data are imported. The research team configured a hardware-in-the-loop with the CORSIM simulation. Selected intersections on the diversion route are controlled by ACTRA in the simulation.

The project team proposes three scenarios in order to evaluate the feasibility of the IDTMS system.

- Base Scenario – There is no DMS sign on I-55 for the diversion traffic. The traffic on the freeway will not reroute no matter how congested the freeway is.
- Rerouting only Scenario – DMS sign is implemented on I-55. The traffic on the freeway will reroute based on the freeway and diversion route traffic conditions. No signal optimization performed at arterials.
- Signal Optimization Scenario – In addition to diversion, the traffic signals on the diversion route will be optimized by the proposed model.

The simulation is run from 4:00 to 6:00 PM, corresponding to the evening peak hour and preceded by a one-hour “warm-up” period. Within each run, performance measures including network wide delay, intersection control delay, diversion route extra travel time, and freeway travel time are output and summarized. The results are shown in Table 20.

Table 20 Network Wide Average Statistics

	Base Scenario	Rerouting only Scenario	Signal Optimization Scenario
Freeway Volume (Vehicle/Hour)	6199	6199	6199
Diverted Volume (Vehicle/Hour)	0	122	455
Network Wide Delay(Second/Mile)	48.0	44.4	38.4
Network Wide Average Speed (Mile/Hour)	28.3	29.2	30.6
Freeway Delay(Second/Mile)	31.2	29.4	10.2
Freeway Speed(Mile/Hour)	40.6	41.8	53.3

The network wide delay, including freeway and arterials all-together, dropped 7.5% after the freeway traffic rerouted to the diversion route. Compared to the rerouting only scenario, the

network wide delay reduced significantly from 44.0 sec/mile to 38.4 sec/mile in the signal optimization scenario which indicates that rerouting freeway traffic to the urban street will also cause considerable congestion if the traffic signal is not adjusted in response to the diversion. In Table 19, the freeway speed in the three scenarios is 40.6 mph, 41.8 mph, and 53.3 mph, respectively. The freeway speed in the Routing-only Scenario is 41.8 mph which is slightly better than the Base Scenario. However, the freeway speed of Signal-Optimization Scenario is 53.3 mph which is much higher than the Routing-only Scenario. The reason that the freeway speed is significantly improved in the Signal-Optimization Scenario is that diverted traffic volume increases from 122 vph to 455 vph to the parallel arterial. The traffic signals on the arterial are optimized so that the arterial is able to accommodate more freeway traffic. Hence the system could raise the percentage of diverted freeway traffic automatically in order to achieve system optimality. Figure 12 shows the effects of the IDTMS system in reducing network delay.

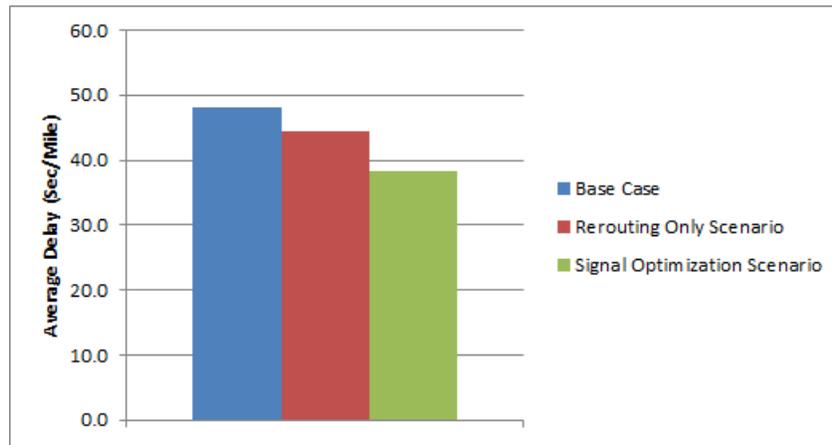


Figure 12 Network Wide Average Delay in Each Scenario

4.2 Proactive Real Time Offset Tuning Algorithm

For the proposed ICM optimization model in section 4.1, based on results of the case study, it could effectively alleviate delays and improve traffic conditions when capacity drops on I-55. It has two places that need to be improved to consist with real world conditions. First, the proposed ICM optimization model is based on fixed-time control. While, most signalized intersections on State St are configured as actuated control, although during the peak hours and division, those controllers are most likely to be defacto fixed time controllers. Second, the proposed ICM optimization model assumes that the diversion probabilities according to different DMS levels are known. In reality, the number of diversion vehicles cannot be predetermined and/or controlled.

The ICM optimization model shows that diversion traffic could results in serious congestions on the diversion route. Traffic signal optimization for signalized intersections on the detour route is necessary which could significantly improve traffic conditions of the ICM corridor. A traffic signal optimization algorithm is proposed to optimize actuated control signalized intersections by explicitly considering the detour traffic from freeways. The algorithm is expected to be ready for field implementation without significant hardware and software improvements.

Traffic signal coordination is a traditional and effective approach to provide smooth traffic progressions in order to decrease delays and increase travel speed. According to Chapter 6 Coordination of Traffic Signal Timing Manual, the objective of traffic signal coordination is to

make sure there is continuous movement along an arterial or traverse major streets of a network with minimum delays and stops which would decrease fuel consumption and vehicle emissions [33].

Traffic signal coordination is based on normal traffic without the consideration of the sudden surge of diversion traffic which varies according to traffic conditions of a freeway, e.g. queue length and travel speed etc. These characteristics of detour traffic disrupt cycle arrival profiles of intersections and dilute and eliminate benefits provided by traffic signal coordination.

The ideal traffic signal coordination requires changes of cycle length, split and offset. In the case of ICM, usually institutional issues require to disrupt the arterial traffic timing plan as little as possible. Freeway operation is usually operated by the State DOT and diversion of traffic to local streets usually is not favored by local DOTs and residents. Offset is one of the crucial factors of traffic signal coordination and is the key for traffic platoons to traverse downstream intersections with minimum stops and delays. Adjustment of offset will provide the least changes in timing plan and the most persuasive approach to interrupt local traffic due to diversion as little as possible to maximize the support of the ICM. Therefore, our ICM traffic signal coordination is limited to real time offset tuning.

Real time offset tuning is divided into two major categories: responsive and proactive. Responsive real time offset tuning approach adjusts offsets based on “past” traffic. With respect to the proactive real time offset tuning method, it tunes offsets according to upcoming and well predicted traffic. A well designed proactive system performs better than responsive real time offset tuning. Thus, a proactive real time offset tuning algorithm is developed for ICM strategies to reduce delays and number of stops of traffic on the diversion path in this project.

In general, for traffic signal coordination, there are two methods for synchronizing coordinated phases of intersections: selected the start points or the end points of green time of the coordinated phases of intersections as the reference points for coordination. In this project, CORSIM is selected as the simulator for validating the effectiveness of the proposed offset tuning algorithm. Based on the research team’s experiences, CORSIM uses the end points of green time of coordinated phases of intersections as reference points for coordination. According to *CORSIM Reference Manual*, the term, yield point, is referred to as the end point of green time of coordinated phases of an intersection in CORSIM [34]. In this report, the terms, offset and yield point, are used which indicate the start point and end point of the green time of coordinated phases of intersections, respectively.

4.2.1 Critical Features of the Proposed Algorithm

Proactive offset tuning is one of the most important features of the proposed algorithm. The proposed algorithm is designed to facilitate one directional progression (the diversion direction). Since the cycle length and splits are changed, negative impact on delays of the non-diversion approaches is minor or none. The proposed algorithm explicitly considers the propagation of sudden surge of detour traffic and the platoons of detour traffic by predicting upcoming traffic according to historical and online traffic data. A data driven model to predict upcoming detour traffic from the freeway is proposed using collected and stored historical traffic data. The collected historical traffic data is expected to be organized and updated weekly, monthly, seasonally and yearly. The historical variations and patterns gathered with real time on-line data are statistically applied to capture the future detour traffic from the freeway. For forecasting upcoming arrival profiles of surface streets on the diversion path, it is different with that of the freeway. Since the roads connected the freeway and its parallel major arterials are usually minor

streets, they may not be coordinated for the daily traffic. So, there is no coordinated historical arrival data for these streets. On the other hand, the traffic patterns for regular traffic and detour traffic in a short time period, such as 5 minutes, 10 minutes or 15 minutes, are not varied significantly. Thus, the real time arrival data in a previous short time period is believed as a good and reasonable predication of upcoming surface street traffic.

Traffic delays are very sensitive to offset changes. The simulation experiments indicate even with only a second change of the offset, traffic delays may have up to 20—30% variation. Thus, for offset tuning, the high resolution vehicle arrival data is needed. The time interval to collect vehicle arrival profiles is one second in this project. For the diversion arterial, to collect historic real time vehicle arrival profiles, an upstream detector is needed to be installed at the upstream endpoint of links on the diversion path, in addition to stop bar detectors for actuated controllers.

The proposed algorithm tunes offsets in real-time for a projection period in the future. Since the offset of an actuated signal controller changes, the controller would experience a transition period to accommodate the gradual changes of the offset, resulting in traffic coordination disruption. For this project, the projection period for offset tuning is 10 coordination cycles.

The Purdue Coordination Diagram (PCD) method proposed by Day, Haseman et al. [28], are modified in this project. The predicted arrival profile for next ten cycles is aggregated to convert a hypothesis profile. The offset is fine-tuned to best accommodate the hypothesis profile.

4.2.2 Framework of the Proposed Algorithm

The major framework of the proposed algorithm in each projection period has six critical steps, the order of which follows the propagation of diversion traffic flow. In Step 1, the cycle diversion traffic from the off ramp of the ICM corridor to the diversion arterial is obtained. When traffic arrives at the stop bar of a coordinated intersection, the arrival profile at the stop bar (without dividing into turning movements) is obtained in Step 2. Offsets of a coordinated intersection are fine-tuned at Step 3 based on arrival profiles established in Step 2. In Step 4, the profiles of movements on diversion lanes of a coordinated intersection at the stop bar are identified and generated by combing information from Step 2 and newly estimated turning percentages of the diversion direction. Based on information from Steps 3 and 4, Step 5 predicts arrival profiles at the upstream detector of the next coordinated intersection. Departure detour volume of a coordinated intersection is estimated in Step 6. After that, the traffic propagates to the stop bar of the next coordinated intersection, which is looped back to Step 2. The processes continue and Steps 2-6 are repeated for each of coordinated intersections on the diversion arterial.

For the convenience of offset tuning and uniformity, all arrival profiles are based on cycle:

- Step 1: Average cycle diversion volume from the off ramp;
- Step 2: Arrivals profiles at stop bar;
- Step 3: Offset tuning;
- Step 4: Arrival profiles of the diversion lanes at stop bar;
- Step 5: Predict upstream arrival profiles at the downstream intersection;
- Step 6: Departure detour volume.

Since the predicted detour platoons propagate coordinated intersections from the first coordinated intersection to the final one, they arrive at each coordinated intersection at different times. Offsets are tuned for coordinated intersections sequentially according to arrival time difference of detour platoons. The time difference of applying the proposed offset tuning algorithm between two

successive coordinated intersections is chosen as the free flow travel time between these two intersections.

4.2.2.1 Step 1: Average Cycle Diversion Volume from Off Ramp

Estimating diversion volume is the first step for most diversion optimization models. Most transportation researchers used mathematical methods to obtain an optimal diversion rate [19, 20]. Since drivers' decision to divert or not cannot be enforced, the optimal diversion rate is difficult to be realized, if not impossible. In this report, surveillance detectors are suggested to be installed on both the upstream endpoint of the ramp exit of the ICM corridor and the upstream endpoint of the diversion direction of each coordinated intersection. These surveillance detectors collect high resolution (second by second) vehicle arrival data. This data is also stored year around as historic data of regular traffic conditions.

When the freeway capacity drops, such as during an incident or work zone, the upstream detector on an off ramp could provide real-time vehicle arrival data. When the total number of vehicles on an off ramp is obviously larger than that of historic data in the same time period, it indicates diversion happened. The difference is real-time diversion volume for this time period. Although the detour volume may vary during diversion, it is reasonable to assume that the collected real-time diversion volume is nearly constant for a short time period, such as 30 minutes. For the proposed offset tuning algorithm, the last five cycles' real-time vehicle arrival data of the off ramp on the diversion route is used to estimate real-time diversion volume at the beginning of each projection horizon.

The average value of the difference between the last five cycles' real time total vehicle arrival and the corresponding time period's historical total arrival is referred to as the average cycle detour volume for the next 10 coordination cycles. The calculated cycle detour volume from the off ramp is assumed nearly constant or not significantly varied for next 10 coordination cycles. During the diversion period, it may appear that, in a (or several) projection horizon(s), the total real time off-ramp arrivals of the last 5 cycles is less than that of historical data. It indicates that few drivers choose to divert in this project horizon. The average cycle diversion volume is assumed to 0 in this situation.

For Steps 2-6 of major framework of the proposed algorithm, the method at the first coordinated intersection is not the same as rest of the intersections. So, steps 2-6 are illustrated separately for the first coordinated intersection and rest of the intersections. For the final intersection, only steps 2 and 3 are implemented, since no downstream intersection needs to be coordinated.

4.2.2.2 Step 2: Arrival Profiles at the Stop Bar

1) For the First Intersection

For the first intersection on the diversion path, the upstream detector is installed on the immediate upstream link at the diversion direction of this intersection. The real-time arrival data collected by the upstream detector in the last five cycles already includes regular traffic and detour traffic after diversion happened. The average arrival profiles of last five cycles are referred to as the forthcoming arrivals of next 10 coordination cycles. The stop bar arrival profile of the first intersection is obtained by shifting the predicted arrival profile of the upstream detector to stop bar. The shift time is the free flow travel time between the upstream detector to stop bar.

2) For Other Intersections

Since steps 2-6 are repeated for offset tuning of each intersection, the predicted upstream arrivals of an intersection is already obtained from the last loop of the proposed algorithm for offset tuning of its upstream intersection. The stop bar arrival profiles of the intersection is propagated by shifting its predicted upstream arrival profiles to stop bar. The time shift is the free flow travel time from the upstream endpoint of the intersection to its stop bar. However, if there is additional major traffic entering the arterial between two successive intersections, an upstream detector is required to install on the link which the additional traffic comes from, to collect historical high resolution arrival data of that traffic. Aforementioned, we use the corresponding time period historical traffic as the prediction of upcoming traffic. The predicted arrival profiles of additional traffic is also propagated to stop bar by the same method. In this case, the total stop bar arrival profiles are the summation of predicted upstream arrivals at the stop bar and the stop bar arrivals of additional traffic. In this paper, a platoon vector with the size of a cycle length is designed to track the possible time stamp within a cycle that detour vehicles may exist (the detail method is illustrated at 4.2.2.3 below). When upstream arrival profiles are shifted to predict stop bar arrival profiles, the platoon vector also needs to be shifted the same time interval.

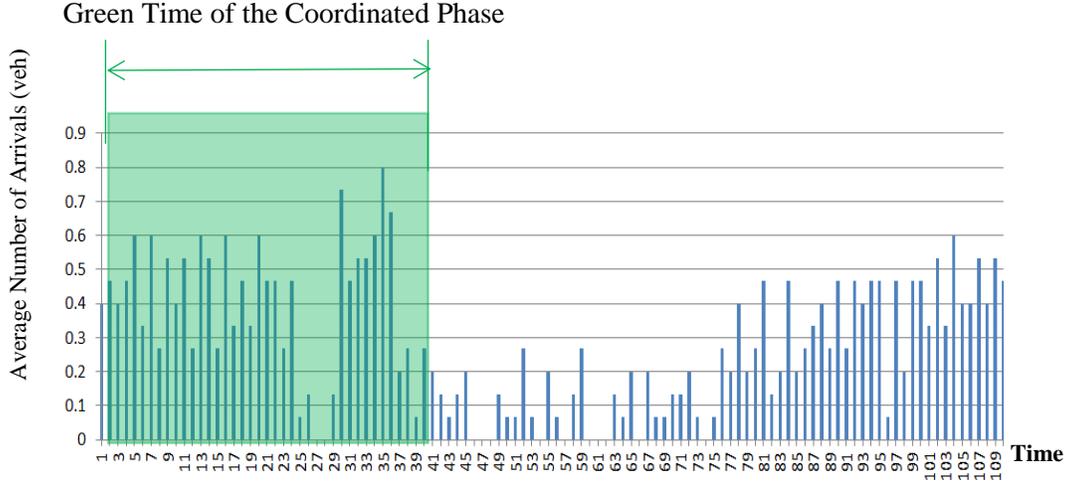
4.2.2.3 Step 3: Offset Tuning

1) For the First Intersection

The PCD method [28] is revised and used in this step. For PCD method, the average cycle arrival profiles and cycle probability of green profiles are combined to build a diagram. Based on the diagram, Day, Haseman et al. could visually measure the quality of progression and utilize percent on green (POG), i.e. ratio of total arrivals on green and total arrivals, as the performance measure to adjust offset. In this project, the historic cycle probability of green profiles is neither collected nor utilized, since it would be largely changed due to impacts of heavy detour traffic. Instead, the predicted (cycle) arrival profiles, as discussed in the previous step, is used.

Total arrivals on green or expected total arrivals on green in a cycle are popular performance measures and/or objective functions for offset optimization [28-30]. The performance measure, cycle total arrivals on green, is also used as the objective function for offset tuning in this project.

The specific offset tuning procedures are conducted as following. Based on predicted upcoming arrival profiles at the stop bar, the total number of arrivals during green interval is counted when it is assumed green starts at the beginning of a cycle. Advance the green start one second, another count of total numbers of arrives on green is obtained. Figure13 shows an instance that assumes green starts at 2 second and then the number of arrivals covered by green time (green shadow in the figure) is the total arrivals on green. After these same procedures are performed for every second in a cycle, the second with maximum value of total number of arrivals on green is considered to be the best offset. According to the got best offset, the corresponding best yield point is easily obtained. The best offset or yield point is applied for actuated controller based on controller requirements for the first intersection.



The proposed algorithm explicitly facilitates detour platoon progression. Instead of tracking specific detour platoon’s physical location, the proposed method tries to use probe vehicles to find the first available platoon departure time and the last possible departure time within the coordinated cycle where the detour platoons may exist. A platoon vector with the size of a cycle length is designed to store the information and the index of the vector is the time stamp in the cycle. From the possible first departure time to the last departure time, the value of the platoon vector is assigned to “1”. For values outside of the first and last departure times, their values are assigned to “zero”.

2) For Other Intersections

It is nearly the same method of the first intersection for tuning offsets that implement for other intersections. The differences are two aspects. First, if two successive intersections are very close, i.e. the free flow travel time between them less than 5 seconds. To decrease delays and avoid unnecessary vehicle acceleration and deceleration for travel through these two intersections, the offset of an intersection equals to summation of the computed offset of its upstream intersection and free flow travel time between them. The equation is shown below.

$$Offset = Offset_{upstream} + FFT \quad (23)$$

Where,

Offset: The offset of an intersection (second),

Offset_{upstream}: The offset of the upstream intersection (second),

FFT: free flow travel time between two intersections (second).

Second, two constraints are added to facilitate detour traffic’s progression. When the green time of the diversion direction of an intersection is higher than the possible time period of the platoon vector, the platoon time period needs to be covered when fine-tuning the offset. On the other hand, the offset tuning is performed by searching within the possible time period in the platoon vector. In the latter case, the platoon vector needs to be reset, since the possible time period for coordinating of detour traffic within a cycle in the intersection is shrunk. These two constraints are helpful for detour traffic to traverse intersections with the minimum stops and delays.

In Steps 3 and 5, it is needed to notice that the detour traffic could be released by multiple phases. The entire green time of these phases need to be considered for offset tuning and predict upstream arrivals of downstream intersection. There are two major possibilities: 1) the movement of detour traffic are assigned to multiple phases in a traffic signal timing plan; 2) Although the movement of detour traffic is assigned to only one phase, this phase also be allocated more green time, since it is frequently concurrently be assigned green time with other non-conflicting phases.

4.2.2.4 Step 4: Arrival Profiles of the Diversion Lanes at Stop Bar

For this step, the method implemented for all coordinated intersections are the same. Diversion lanes are the lanes which discharge detour traffic. Since heavily detoured traffic enters the arterial, turning percentages of the diversion direction of each intersection are changed. New turning percentages need to be estimated. Based on Step 2, the average cyclic total number of vehicle arrivals at the stop bar is known. The offset of the upstream intersection is tuned and the number of detour traffic departure from the upstream intersection is calculated in Step 6 for all intersections, except the first intersection. For the first intersection, its cycle detour volume from the off ramp is already obtained in Step 1. Regular traffic of each turning movement within the diversion direction is estimated by the equations below.

$$N_{regular} = N_{total} - N_{detour} \quad (24)$$

$$N_{regular_T} = N_{regular} * percent_T_regular \quad (25)$$

$$N_{regular_L} = N_{regular} * percent_L_regular \quad (26)$$

$$N_{regular_R} = N_{regular} * percent_R_regular \quad (27)$$

Where,

$N_{regular}$: Cycle total number of regular traffic (veh),

N_{total} : Cycle total number of traffic under diversion (veh),

N_{detour} : Cycle detour volumes departure from the off ramp or the upstream intersection (veh),

$N_{regular_T}$, $N_{regular_L}$, $N_{regular_R}$: Cycle number of vehicles for regular through, left, and right turns (veh),

$percent_T_regular$, $percent_L_regular$, $percent_R_regular$: regular turning percentages for left, through and right turn vehicles (%).

Based on $N_{regular_T}$, $N_{regular_L}$, $N_{regular_R}$, and N_{detour} , the new turning percentages are easily obtained. By adding detour traffic to corresponding turning movements, new turning movement percentages can be calculated. According to new turning percentages, lane configuration of detour lanes, and total stop bar arrival profiles from Step 2, the stop bar arrivals profiles of movements on the diversion lanes can be estimated as the equation below second by second.

$$SA_{DL} = SA_{Total} * NewTurningPercentages_{DL} \quad (28)$$

Where,

SA_{DL} : stop bar arrival profile of movements on the diversion lanes (veh),
 SA_{Total} : Total stop bar arrival profile of the diversion approach of an intersection (veh),
 $NewTurningPercentages_{DL}$: New turning percentages under diversion for movements on the diversion lanes (%).

4.2.2.5 Step 5: Predict Upstream Arrival Profiles at the Downstream Intersection

The method that Day, Haseman et al., [28] to predict changes of arrival profiles due to implementation of new offset is enhanced and used for all intersections. Figure 14 shows complete procedures. Figure (a) is the upstream real time arrival profiles in the last five cycles of the downstream intersection. The arrival profiles are referred to as upcoming upstream arrival profile of the downstream intersection for next ten cycles with no offset changes and detour volume variation of the subject intersection. Figure (b) shows predicted changes due to the impacts of offset tuned of the subject intersection. Figure (c) shows complete predicted upstream arrival profiles at the downstream intersection after offset tuned and detour volumes varied of the subject intersection.

Figure (a) is obtained by the upstream detector in the diversion direction of the downstream intersection. Figure (b) can be obtained based on Figure (a) by applying the method presented by Day, Haseman et al. Although detour traffic has notable impacts on demand for the diversion direction of arterials, they have little or no impacts on demands for other approaches of an intersection. It is assumed that the detour traffic travels on one direction of the major street. As a result, the arrivals from side streets do not change. The arrival profiles of the diversion direction of the downstream intersection are determined by traffic from side streets and the diversion direction of major streets at the subject intersection. Since arrival profiles from side streets at the subject intersection shown in Figure (b) are not changed, only departures at the diversion direction of major streets at the subject intersection need to be predicted due to detour volume variations in each projection horizon. The predicted upstream arrival profiles of the downstream intersection are shown in Figure (c).

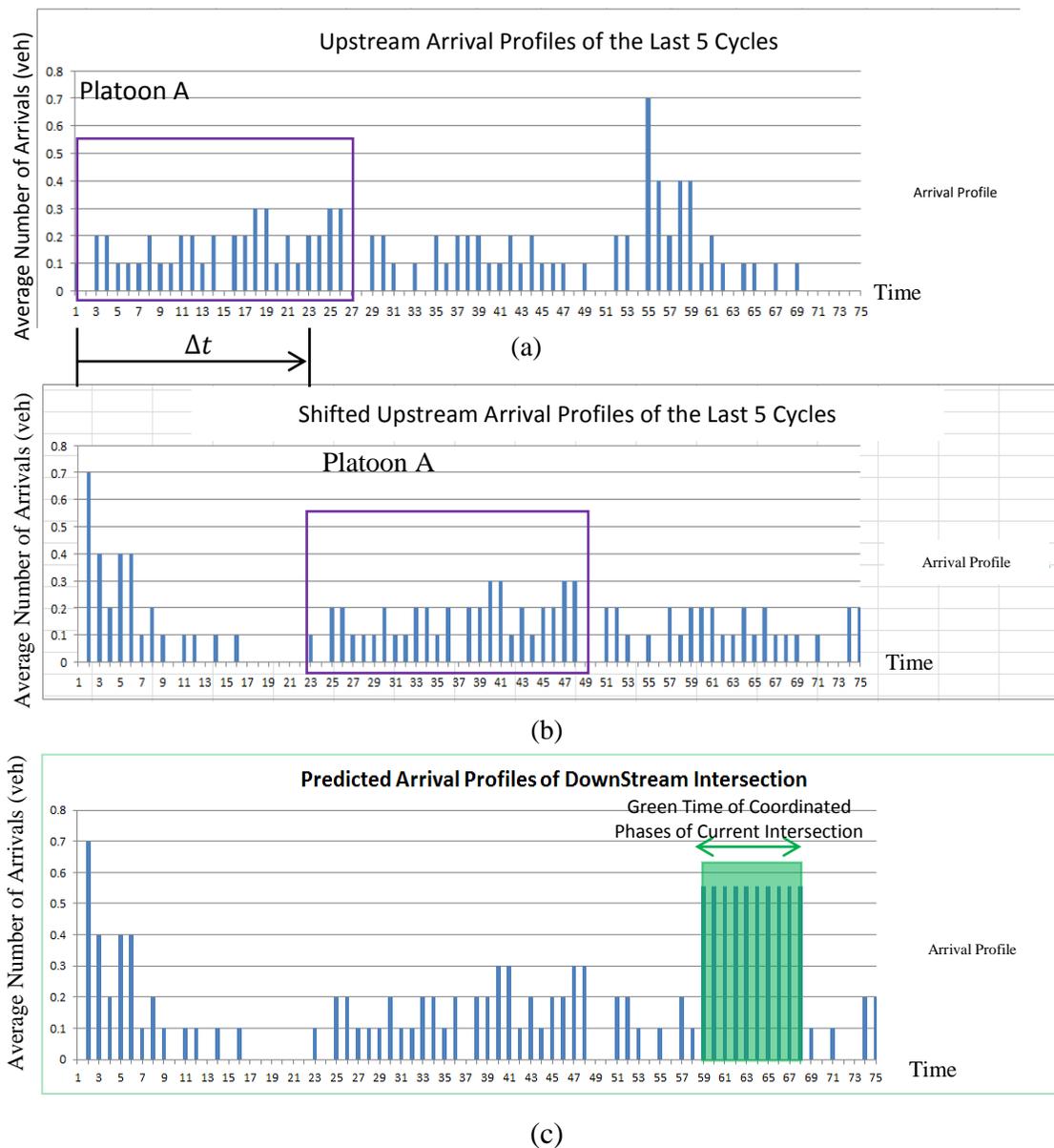


Figure14 Procedures for Predicting Upstream Arrivals of the Downstream Intersection

The method to estimate departures in the diversion direction of major streets of an intersection is explained as follow. Based on Step 4, arrival profiles at stop bar of detour lanes can be obtained. From Step 3, the new tuned offset of the intersection is known. The specific green time started in a cycle of the intersection is identified. Based on this information, the total arrivals on green and total arrivals on red on the diversion lanes can be determined. The total arrivals on red are actually resident queues at the beginning of green in a cycle. After green start, vehicles queued at the stop bar during red are released at saturation discharge headway until the queue is cleared. Based on the saturation discharge headway, the total saturation departure rate (veh/sec) for the diversion lanes of the intersection is calculated with the equation below.

$$SD = \frac{1}{DischargeHeadway} \times N_{DL} \quad (29)$$

Where,

SD : Total saturation departure rate of detour lanes(veh/sec),

$DischargeHeadway$: Average discharge headway per lane (sec/veh),

N_{DL} : Number of detour lanes for discharging detour traffic.

Therefore, the second by second departure profiles at the stop bar on the diversion lanes can be determined by the equations below.

$$AC = SD - V_a \quad (30)$$

$$Nr_{next} = Nr_{current} - AC \quad (31)$$

$$DP = \begin{cases} SD, & Nr_{next} > 0 \\ V_a + Nr_{current}, & Nr_{current} > 0 \text{ and } Nr_{next} \leq 0 \\ V_a, & Nr_{current} = 0 \end{cases} \quad (32)$$

Where,

AC : Available capacity to clear resident queue (veh),

V_a : Arrival profiles of the current second (veh),

$Nr_{current}$: Resident queue length of current second (veh); $Nr_{current} = \sum N_r$ at the beginning of green,

$\sum N_r$: Total number of arrivals on red of a cycle (veh),

Nr_{next} : Resident queue length of the next second (veh),

DP : Departure profiles of the current second (veh).

SD is the capacity that vehicles can be released from the stop bar of diversion lanes in one second. According to arrival profiles at the stop bar of the diversion lanes at the current second, the available capacity AC to clear resident queues can be estimated. Then, the remainder queue lengths of the next second, Nr_{next} , could be computed. The departure profile for each second has three possibilities: 1) $Nr_{next} > 0$, the resident queue cannot be cleared in the current second. So, the departure profile of the current second equals to SD ; 2) $Nr_{current} > 0$ and $Nr_{next} \leq 0$, the queue is cleared at the current second. The departures of the current second equal to the summation of the remainder queue lengths and arrivals of the current second; 3) $Nr_{current} = 0$, queue is already cleared. Vehicles could directly departure when they arrival at the stop bar of diversion lanes.

The predicted stop bar departure profiles are used to replace corresponding time period arrival profiles in (b) of Figure14 to obtain the predicted upstream arrival profiles of the downstream intersection, as shown in (c) of Figure14. Specifically, the arrival profiles from 59 second to 68 second in Figure (b) are replaced by predicted stop bar departures of diversion lanes of the subject intersection to obtain (c) of Figure14.

4.2.2.6. Step 6: Departure Detour Volume

1) For the First Intersection

For the first intersection, the cycle diversion volume from the off ramp of an ICM corridor is obtained from Step 1. It is assumed that the possibility of detour vehicles arrive at the stop bar of

the first intersection at each second of a cycle is equal. The departure detour volume within a cycle of the first intersection is estimated as the equation below.

$$DD_{First} = \frac{DetourGreen_{First}}{CycleLength} \times DetourVolume_{OffRamp} \quad (33)$$

Where,

DD_{First} : Estimated cycle departure detour volume from the first intersection (vehs),

$DetourGreen_{First}$: Green time of detour lanes at the first intersection (seconds),

$CycleLength$: Coordination cycle length (seconds),

$DetourVolume_{OffRamp}$: Detour traffic from the off ramp in each cycle obtained from the Step 1 (vehs).

2) For Other Intersections

The estimated departure detour volume for an intersection is based on the green time duration, number of lanes, and capacity per lane of diversion lanes and the departure detour volume from the upstream intersection. The departure detour traffic of the subject intersection is calculated as the equations below.

$$DD_{estimated} = \frac{DetourGreen_{current}}{DetourGreen_{upstream}} * DD_{upstream} * \frac{CP_{current} * ND_{current}}{CP_{upstream} * ND_{upstream}} \quad (34)$$

$$DD_{current} = \begin{cases} DD_{estimated}, & \text{when } DD_{estimated} \leq DD_{upstream} \\ DD_{upstream}, & \text{when } DD_{estimated} > DD_{upstream} \end{cases} \quad (35)$$

Where,

$DD_{estimated}$: Estimated departure detour volume of the subject intersection in a cycle (veh),

$DetourGreen_{current}$: Green time of diversion lane of the subject intersection (second),

$DetourGreen_{upstream}$: Green time of diversion lanes of the upstream intersection (second),

$DD_{upstream}$: Departure detour volumes of the upstream intersection in a cycle (veh),

$DD_{current}$: Departure detour volumes of the subject intersection in a cycle (veh),

$CP_{current}$: Capacity per lane of diversion lanes of the subject intersection (veh/h),

$ND_{current}$: Number of detour lanes of the subject intersection,

$CP_{upstream}$: Capacity per lane of diversion lanes of the upstream intersection (veh/h),

$ND_{upstream}$: Number of detour lanes of the upstream intersection.

4.2.3 Case Study

4.2.3.1 Background

To evaluate the effectiveness of the proposed proactive real time offset tuning algorithm, a case study is conducted. Figure 15 shows the entire study area. A hypothetical capacity reduction due to an accident or work zone has occurred; the location is indicated as the purple rectangle in Figure 15. Figure 15 is obtained from Google Maps (<https://maps.google.com/maps?hl=en>) which shows the study area. The red arrows in the figure indicate the diversion path.

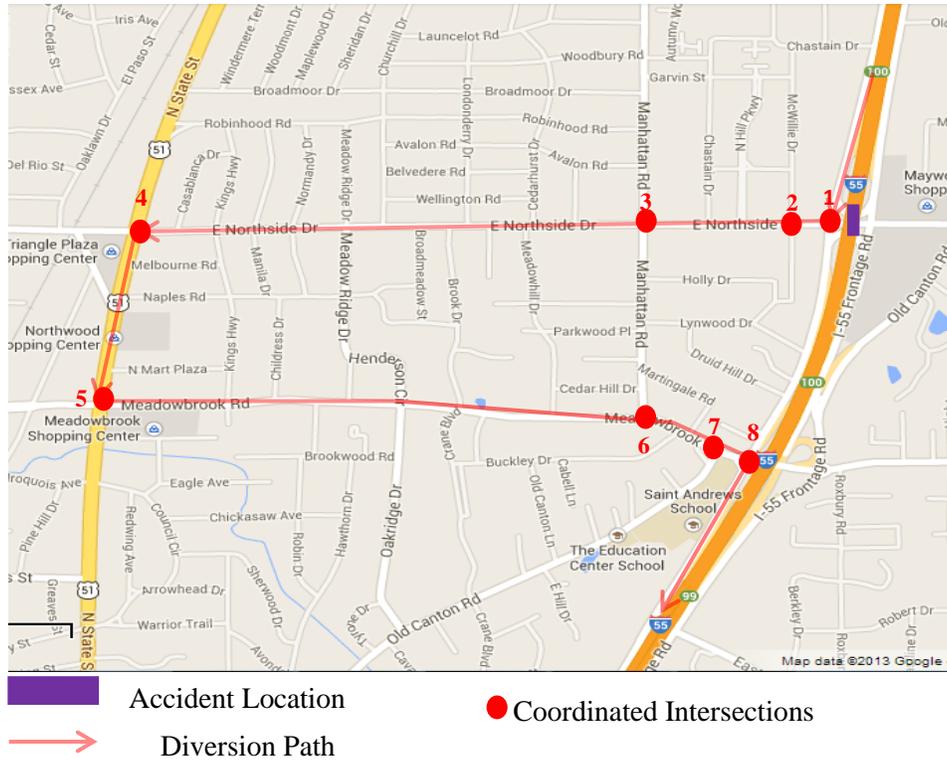


Figure 15 Case Study Area for Proactive Real Time Offset Tuning Algorithm

For the capacity reduction section of I-55, the directional number of lanes is 3. It is assumed that the operator of Traffic Management Center immediately identifies the capacity reduction and distributes the information via DMS sign and radio. As a result, drivers start to make a detour when the capacity reduction happens. The 8 red dots in Figure 15 are 8 coordinated intersections on the diversion path. Additional detail information of this hypothetical capacity reduction is listed in Table 21.

Table 21 Detail Information of a Hypothetical Capacity Reduction

Time period	Duration (seconds)	Start (seconds)	End (seconds)	Duration (seconds)	Capacity	Duration (seconds)	Diversion rate
1 (warm up)	900	0	900	900	Full	900	0%
2 (Capacity Reduction)	9000	900	2700	1800	1 lane closed (the next lane reduce 70% capacity)	5400	5%
		2700	4500	1800			10%
		4500	6300	1800			15%
		6300	8100	1800	1 lane closed	1800	10%
		8100	9900	1800	the most right lane reduce 50% capacity	1800	5%
3 (Capacity restored)	900	9900	10800	900	Full	900	0%

4.2.3.2 Simulator and Different Scenarios

TSIS CORSIM (version 5.2, Build 5) is selected as the simulator. The CORSIM simulation model built by the research team is well calibrated in this project. The signal timing plans of 8 coordinated intersections were directly obtained from the field traffic signal controller. Since field timing plans are not coordinated, these signal timing plans are optimized by TRANSYT-7F for coordination. CORSIM Run-Time Extension (RTE) is utilized for implementing proposed proactive real time offset tuning algorithm. In this project, number of vehicles currently on the link and number of vehicles currently departure from the link are used to simulate the high resolution link arrival data. Besides, red turn on red is not allowed for all the coordinated intersections.

Due to a sudden loss of more than 50% capacity on the freeway, some drivers choose to make diversions due to the speed reduction and the accident information received. Gou's dissertation indicated that diversion rates in the United States was in the range from 1.6% to 46% reported by previous studies in the U.S.A.[35]. We used an intermediate maximum value of 20%. The temporary variations of the diversion rate during the entire simulation are assumed as in Table 21. In addition to the reference [35], the maximum diversion rate is selected based on the research team's simulation experiences for the study network shown in Figure 15, as well. The maximum diversion rate is chosen to balance the congestion levels between freeway and the parallel arterial by watching simulation animations. Drivers only divert to nearby arterials to achieve travel time savings. Two diversion scenarios are developed in this project.

Scenario 1 (Offline Coordination without the Consideration of Diversion Traffic and Online Offset Tuning): Instead of comparing with the outdated traffic signal timing plan in the field, we raise the performance measurement bar of the proposed algorithms. The signal timing plans of coordinated intersections on the diversion path are optimized by TRANSYT-7F for regular traffic. It should be noticed that the performance of scenario 1 has already been improved upon the existing traffic signal timing plan. Performance from this scenario is used as a benchmark case to evaluate the performance of the proposed algorithm.

Scenario 2 (Diversion with Proactive Real Time Offset Tuning): The signal timing plans optimized by TRANSYT-7F are also used for coordinated intersections. In addition, the proposed proactive real time offset tuning algorithms are implemented for these 8 coordinated intersections. The coordinated cycle length optimized by TRANSYT-7F is 75 seconds. The proposed algorithm is used to fine-tune the offsets of 8 coordinated intersections for every 10 cycles, i.e. 750 seconds (12.5 minutes). Although diversion happens at 900 seconds, the proposed algorithm begins to be implemented at 1,500 seconds. The 10 minutes delay is used for collecting real-time diversion data.

Since the expected historic database of ICM corridor currently has not been built, a simulation run is conducted first to generate arrival data sets of freeway and surface streets under normal conditions, i.e. no incident happen. The pre-generate arrival data sets of freeway and arterials are referred to as historic arrival data of freeway and arterials, respectively, for scenarios 1 and 2. Then, to eliminate impacts of randomness on performance of the proposed algorithm, each 20 simulation runs with different random number of seeds are conducted for Scenarios 1 and 2, respectively.

4.2.3.3 Results of Case Study

Table 22 contains performance data of all coordinated intersections. In Table 22, the performance data of the detour direction and performance data of both directions or that of the entire intersection are provided. For the first intersection, compared to scenario 1, all performance data of the detour direction and that of the entire intersection in scenario 2 are significantly improved. With respect to the second intersection, notable improvements are also observed. Percentage of stopped vehicle is the performance measure which gets the highest improvement in the diversion direction and the both directions (the diversion direction and its opposite direction). For the third and fourth intersections, performance data of scenario 2 is better than that of scenario 1, but improvements to the both directions are not as significant as that of the first and second intersections.

Table 22 Performance Measures of All Coordinated Intersections

Intersection	Approach		Travel Time Per Vehicle (seconds/vehicle)	Control Delay Per Vehicle (seconds/vehicle)	Stopped Vehicles Percent	Volume (vph)	LOS***
1	SB (Scenario 1)	Average	164.52	107.62	65.6	1378.94	F
		standard deviation	18.86	10.34	0.86	15.28	
	SB (Scenario 2)	Average	124.27	83.89	63.63	1386.18	F
		standard deviation	21.73	12.29	1.31	14.47	
	Difference		-40.25	-23.74	-1.98	7.24	
	Difference (%)		-24.5%	-22.1%	-3.0%	0.5%	
	Intersection Level	Scenario 1	113.01	73.65	62.81	1021.6	E
		Scenario 2	89.5	60.58	64.38	1027.54	E
		Difference	-23.51	-13.07	1.57	5.94	
		Difference (%)	-20.8%	-17.7%	2.5%	0.6%	
2	WB (Scenario 1)	Average	21.21	10.95	52.9	870.72	B
		standard deviation	0.4	0.37	1.53	16.43	
	WB (Scenario 2)	Average	17.41	7.79	33.78	881.23	A
		standard deviation	0.71	0.65	2.48	17.08	
	Difference		-3.8	-3.16	-19.12	10.51	
	Difference (%)		-17.9%	-28.9%	-36.2%	1.2%	
	Both Direction*	Scenario 1	22.58	12.9	56.22	727.47	B
		Scenario 2	19.44	10.2	40.79	735.42	B
		Difference	-3.13	-2.7	-15.43	7.95	
		Difference (%)	-13.9%	-20.9%	-27.4%	1.1%	
3	WB (Scenario 1)	Average	41.56	15.55	45.25	819.66	B
		standard deviation	1.22	1.06	2.15	15.44	
	WB (Scenario 2)	Average	38.34	12.89	38.58	831.28	B

		standard deviation	0.9	0.81	1.89	15.62	
	Difference		-3.22	-2.66	-6.67	11.62	
	Difference (%)		-7.7%	-17.1%	-14.7%	1.4%	
	Both Direction*	Scenario 1	40.81	15.18	48.23	717.28	B
		Scenario 2	39.18	13.89	44.63	724.99	B
Difference		-1.63	-1.29	-3.6	7.71		
Difference (%)		-4.0%	-8.5%	-7.5%	1.1%		
4	WB (Scenario 1)	Average	373.62	257.93	94.79	602.9	F
		standard deviation	23.07	14.7	0.8	19.23	
	WB (Scenario 2)	Average	337.04	233.53	93.71	635.59	F
		standard deviation	25.62	16.56	0.86	26.39	
	Difference		-36.57	-24.41	-1.09	32.69	
	Difference (%)		-9.8%	-9.5%	-1.1%	5.4%	
	Both Direction*	Scenario 1	228.02	158.24	89.99	565.5	F
		Scenario 2	210.36	145.99	89.24	584.31	F
		Difference	-17.66	-12.25	-0.75	18.81	
		Difference (%)	-7.7%	-7.7%	-0.8%	3.3%	
	5	SB (Scenario 1)	Average	76.78	59.14	81.46	927.11
standard deviation			7.65	5.69	1.59	21.73	
SB (Scenario 2)		Average	85.12	64.43	82.86	952.35	E
		standard deviation	10.28	6.76	2.31	23.9	
Difference		8.33	5.29	1.4	25.24		
Difference (%)		10.9%	8.9%	1.7%	2.7%		
Both Direction*		Scenario 1	63.8	49.22	81.34	768.9	D
		Scenario 2	69.89	53.14	82.2	789.35	D
		Difference	6.09	3.92	0.86	20.46	
	Difference (%)	9.5%	8.0%	1.1%	2.7%		
6	EB (Scenario 1)	Average	31.06	6.47	31.89	576.04	A
		standard deviation	0.45	0.37	1.27	17.49	
	EB (Scenario 2)	Average	31.22	6.58	30.47	599.01	A
		standard deviation	0.48	0.43	1.41	15.36	
	Difference		0.16	0.11	-1.42	22.97	
	Difference (%)		0.5%	1.7%	-4.4%	4.0%	
	Both Direction*	Scenario 1	21.62	5.2	26.93	560.86	A
		Scenario 2	23.15	6.49	31.44	573.57	A
		Difference	1.53	1.29	4.51	12.71	
Difference (%)		7.1%	24.9%	16.7%	2.3%		
7	EB	Average	27.67	19.05	66.03	594.75	B

	(Scenario 1)	standard deviation	0.73	0.69	1.58	20.4	
	EB (Scenario 2)	Average	26.25	17.59	59.88	617.21	B
		standard deviation	0.93	0.84	1.92	19.4	
	Difference		-1.42	-1.46	-6.16	22.46	
	Difference (%)		-5.1%	-7.7%	-9.3%	3.8%	
	Both Direction*	Scenario 1	26.34	18.41	58.62	527.25	B
		Scenario 2	26.42	18.4	58.71	542.05	B
		Difference	0.08	-0.01	0.09	14.8	
		Difference (%)	0.3%	-0.1%	0.2%	2.8%	
	8**	EB (Scenario 1)	Average	8.74	1.35	3.08	968.21
standard deviation			0.17	0.13	0.78	18.33	
EB (Scenario 2)		Average	8.81	1.43	3.54	991.39	A
		standard deviation	0.18	0.13	0.5	19.62	
Difference		0.07	0.08	0.46	23.18		
Difference (%)		0.8%	6.0%	14.9%	2.4%		
EB (Scenario 1)		Average	20.11	16.7	66.74	516.18	B
		standard deviation	0.47	0.45	1.3	12.95	
EB (Scenario 2)		Average	20.35	16.93	61.4	515	B
		standard deviation	0.74	0.68	1.68	11.93	
Difference		0.24	0.23	-5.34	-1.18		
Difference (%)		1.2%	1.4%	-8.0%	-0.2%		
ALL 8 Intersections (Both Direction*)	Scenario 1	66.4	42.56	56.51	736.21	D	
	Scenario 2	60.66	39.1	54.79	747.45	D	
	Difference	-5.74	-3.46	-1.72	11.25		
	Difference (%)	-8.6%	-8.1%	-3.0%	1.5%		
ALL 8 Intersections (Detour Direction)	Scenario 1	86.71	55.97	55.33	888.26	E	
	Scenario 2	76.78	49.74	51.13	902.64	D	
	Difference	-9.93	-6.22	-4.2	14.38		
	Difference (%)	-11.5%	-11.1%	-7.6%	1.6%		

Note: *both directions: the diversion direction and its opposite direction.

** EB of Intersection 8 has two links are impacted by a traffic signal

***LOS is determined according to highway capacity manual 2010 [36].

With respect to intersection 5, performance data of scenario 2 in the diversion direction and both directions is worse than that of scenario 1. There are two reasons. First, there are 2.7% more volume entered intersection 5 of scenario 2 than that of scenario 1. LOS of the diversion direction of intersection 5 is already E. When an intersection nears its capacity, a few more vehicles entering the intersection may result in significant performance reduction. This is also verified by the standard deviation of performance measures in scenario 2 being larger than in scenario 1. From Figure 15, the detour vehicles make left turn at intersections 4 and 5. There is only one left turn bay for intersections 4 and 5. When the detour volume is heavy, congestions may appear at these two intersections. Figure16 is a screenshot of CORSIM animation. It clearly shows that

congestion occurs at these two intersections, especially for intersection 4. This situation is proved by LOS in Table 22. In the table, LOS of the detour direction of intersections 4 and 5 are F and E, respectively. This situation results in less detour traffic entering downstream intersections 5-8. As a result, the effects of the proposed algorithm for intersections 5-8 are not as good as that of intersections 1-4. The reason is that the proposed algorithm explicitly facilitates progression of detour traffic and little detour traffic can traverse downstream intersections 5-8. One suggestion to Mississippi DOT would be that increase the capacity of intersections 4 and 5, especially for left turn lane of the diversion direction, when the ICM strategies would be implemented.

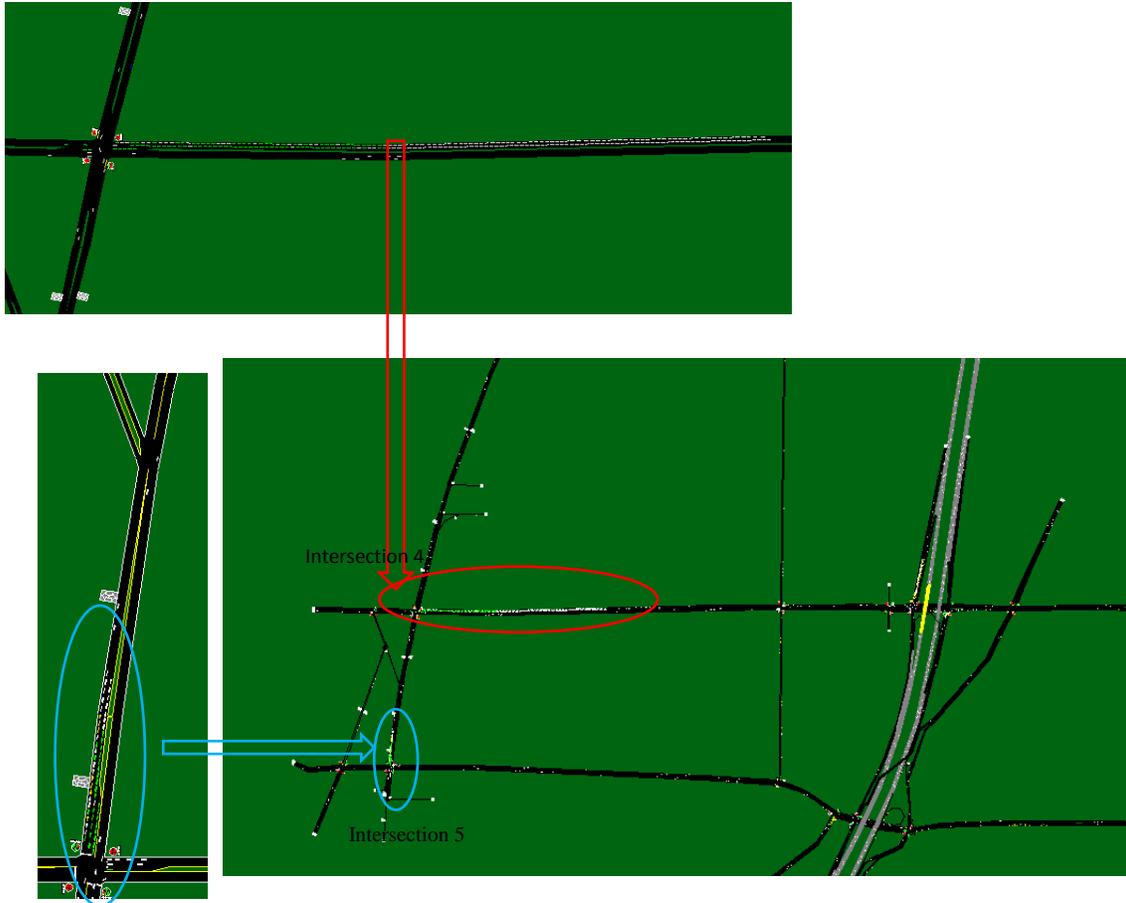


Figure16 Congestions of Intersections 4 and 5

Although effects of the proposed algorithm for individual intersections 5-8 are not perfect, the algorithm significantly improve progressions for the detour direction of the 8 intersections and it also obviously enhances traffic conditions of the entire diversion arterial. The simulation statistics indicates that the proposed approach has a saving of 11.5% in travel time and 11.1% in control delay in the diversion direction. The LOS of the detour direction of the entire arterial is improved from E to D by implementing the proposed algorithm. The proposed algorithm does not sacrifice the opposite direction of detour approach on all intersections at all. The statistical analysis indicates average vehicle control delay for both the diversion direction and its opposite direction of all intersections has decreased by 8.1%, with lower control delay at 95% confidence interval.

5. AN EXPERT SYSTEM APPROACH

5.1 Purpose and Approach

An ICM optimization model and a proactive real time offset tuning algorithm are proposed in the last section. The proposed offset tuning algorithm is utilized to establish “an expert approach”. A look-up table is created for traffic engineers to switch the timing plan manually before the proposed field traffic surveillance devices are installed, some minor geometric improvement at intersections such as extending/adding turning bays and the proposed models are field tested. During these periods, MDOT could use the “lookup” tables at the discretions of traffic engineers whenever there is a need. Furthermore, the expert system is also a backup system when the optimization approach is failed due to equipment breakdown, maintenance of the system, bugs in the systems and/or other reasons.

Under this approach, signal timing plans will be pre-generated by the proposed offset tuning algorithm, based on different scenarios. The scenarios are established according to the input from MDOT engineers, such as combinations of incident severity, congestion levels of diversion paths, time of day, and diversion traffic etc. It may need some time to complete enhancements described in the last paragraph.

For the system approach, incident scenarios are pre-defined in simulation. After simulation, the offsets generated by the proposed offset tuning algorithm are recommended for corresponding scenarios.

There are so many unknown variables and scenarios we will need to produce for an implementable “lookup” table, due to the funding and time limitations of this project, we show case how to use the proposed system to grenade the table. We will discuss our outcomes with MDOT engineers, once feedbacks from MDOT, improvement and refinement of the systems will be made. It is expected that there will be several iterations between the project team and MDOT engineers before a final plan could be produced.

5.2 Case Study

5.2.1 Background of the Case Study

A case study is conducted to validate the effectiveness of the proposed offset tuning algorithm when it is used as the offline method. In addition, the case study is the show case of massive simulation runs to generate lookup table for the expert approach.

For the location of this case study, the segment of I-55 between Woodrow Wilson Ave and Fortification St is selected which has a higher frequency of accidents occur than other locations, based on MDOT’s suggestion. Figure 17 shows the selected study area obtained from Google Map (<https://maps.google.com/>).

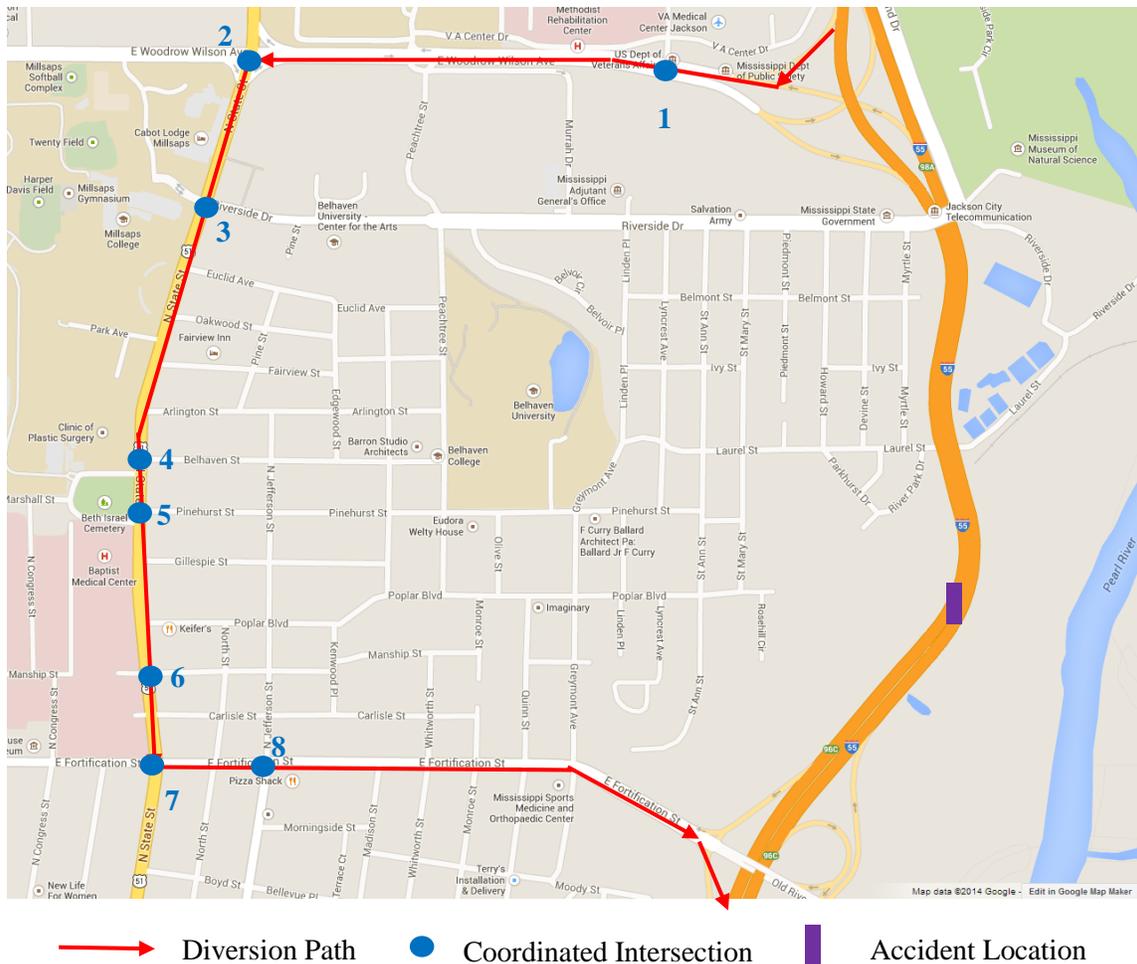


Figure 17 The Study Area for the Expert System Approach

About one year of accident data from I-55 in the Jackson area is provided by MDOT (Amrik Singh, unpublished data). One of the accidents is selected as the example case to illustrate the expert system construction. For the selected accident, only middle lane was closed for 119 minutes. The directional number of lanes of this segment of I-55 is 3. In general, when the middle lane is closed, it also has impact on the capacity of left and right lane. It is assumed that the left and right lanes of this section of I-55 are lost 50% capacity due to the middle lane closed. Based on the selected accident data, three time periods are modeled: 1) Network warm up; 2) Accident; 3) Capacity restoration. Table 23 shows the details of this accident. It is assumed that the operators at Traffic Management Center (TMC) of MDOT immediately identify the accident happened and distribute the accident information to drivers by DMS and/or radio. Drivers instantly make a diversion, due to the segment of I-55's capacity is heavily reduced. It is also assumed that the operators at TMC inform drivers at once when the accident is cleared and capacities of all lanes are restored. Then, drivers will not seek diversion and keep traveling on I-55. Gou's dissertation indicated that diversion rates in the United States was in the range from 1.6% to 46% reported by previous studies in the U.S.A. [35]. As the same as the case study in section 4, due to no field data related to diversion rates of the study network, the selected maximum diversion rate is selected by watching the simulation animation to roughly equilibrate congestions level between I-55 and State St. Table 23 summaries capacity reduction and the diversion rate in

our case studies. The optimized signal timing plans out of this section could be implemented in the future when similar scenarios happen.

Table 23 Capacity Reduction and Diversion Rates in Expert System Case Study

	Time Period	Duration (sec)	Start Time (Sec)	End Time (Sec)	Duration (sec)	Capacity Reductions	Diversion Rate
1	Network Warm up	900	0	900	900	None	0
2	Diversion	7140	900	2700	1800	(The middle lane closed. The left and right lane reduce 50 % capacity)	0.1
			2700	4500	1800		0.15
			4500	6300	1800		0.2
			6300	8040	1740		0.25
3	Capacity Restoration	900	8040	8940	900	None	0

Three scenarios are initially proposed to measure the performance of the timing plan to be proposed in this section. The details of these scenarios are listed below.

- Scenario 0 (Existing signal timing plans): the field signal timing plans are used to accommodate diversion traffic from I-55 when the accident happened. This scenario assesses the ability of existing field timing plans to adapt for sudden surge of detour traffic. This scenario is also served as the base line scenario.
- Scenario 1 (Signal timing plan optimized by TRANSYT-7F): in this scenario, the 8 signalized intersections on the diversion route are coordinated and the coordination signal timing plans are optimized by TRANSYT-7F. The coordination signal timing plans optimized by TRANSYT-7F takes the average additional diversion volume into consideration for optimization. The coordination cycle length optimized by TRANSYT-7F is 102 seconds.
- Scenario 2 (Proactive real time offset tuning algorithm: offline mode): in this scenario, the coordination signal timing plans optimized by TRANSYT-7F in scenario 1 is still utilized. Based on the signal timing plans optimized by TRANSYT-7F, the proposed offset tuning algorithm is implemented to generate appropriate offsets for future use.

5.2.2 Improvements of the Study Network

Before conducting simulation for the presented three scenarios, the research team performed some simulation runs for Scenario 0 to test whether the existing network needs to enhance to accommodate heavy detour traffic. The objective of this action is to avoid serious congestions that appear due to lack of capacity which could not be solved by traffic management, such as traffic signal timing optimization.

When the research team performed simulation runs for Scenario 0, the existing network experienced serious congestions. Figure 18 shows the serious congestions on State St. According to Figure 18, queues start to accumulate from intersection 7 (Fortification St at State St) and spilled back close to intersection 3 (Riverside Dr at State St). From Figure 17, detour traffic at intersection 7 needs to make a left turn from State St to Fortification St. Due to lack of capacity to discharge left turn vehicles at SB of intersection 7, serious congestions shows in Figure 18. So, an additional left turn bay of SB at intersection 7 is needed which is added into CORSIM simulation network.

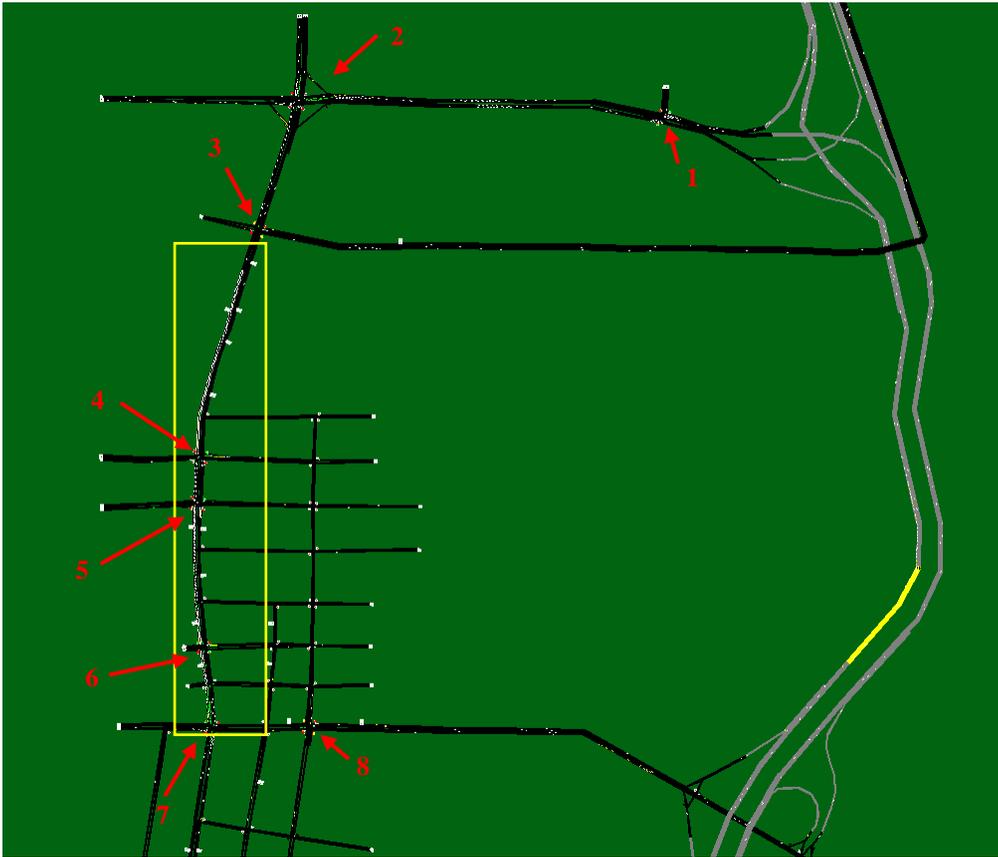


Figure 18 Serious Congestions on State St

After a left turn bay added at intersection 7, another simulation run is conducted to test whether other intersections need to improve capacity. According to Figure 19, intersection 2 also experiences heavy congestions. The reason is the same with intersection 7 that detour traffics make left turn at intersection 2 from Woodrow Wilson Ave to State St. Since the capacity of intersection 2 WB is not enough for discharging sudden surge left turn traffic, intersection 2 is seriously congested. So, an additional left turn lane is needed and it is added at intersection 2WB in CORSIM simulation network.



Figure 19 Congestions at Intersection 2

5.2.3 Results of Case Study

A simulation run is conducted first to generate arrival data sets of freeway and surface streets of the study ICM corridor under normal conditions, i.e. no incident happen, since the expected historic database of ICM corridor has not been built yet, the pre-generate arrival data sets of freeway and arterials are referred to as historic arrival data of freeway and arterials, respectively, for all three scenarios. Then, 20 simulation runs for Scenario 0 are performed to generate results of Scenario 0 with little impacts of roundness. Table 24 shows results of diversion direction and its opposite direction for the 8 coordinated intersections for Scenario 0. According to Table 24, with respect to control delay per vehicle, most intersections' control delay per vehicle of detour and its opposite directions are less than 20 seconds/vehicle, except intersections 2 and 7, which corresponding LOS are A or B. The LOS of diversion direction of intersections 2 and 7 are E. It indicates that intersections 2 and 7 with the existing signal timing plans are bottleneck of the diversion arterial, even if improvements of existing network mentioned above complete. For other intersections, traffic conditions under diversion are good after improvements complete. So, we only concern performance enhancements of intersections 2 and 7 as well as the entire arterial after existing signal timing plans are optimized in Scenarios 2 and 3.

It also could be found that the standard deviations of all performance measures of total 8 intersections are minor. Due to this characteristic, we just do one simulation run for Scenarios 2, 3 and 4 which could well represent performances of each scenario. To make sure the traffic patterns for different scenarios are identical, one utilized random number of seeds from 20 simulation runs of scenario 0 is used for simulations for scenarios 2 and 3. The results of the

selected one simulation run of Scenario 0 from 20 simulations are referred to as the baseline case for comparison.

Table 24 Results of 8 Coordinated Intersections of Scenario 0 (20 Simulation Runs)

	Intersection	Direction		Travel Time Per Vehicle	Control Delay Per Vehicle	Stopped Vehicles Percent	Volume	LOS*	
			Units	Seconds/Vehicle	Seconds/Vehicle	(%)	vph		
Scenario 0 Existing Signal Timing Plans	1	WB (Detour)	Average	18.82	8.88	46.7	1526.52	A	
			Standard Deviation	0.44	0.39	1.37	18.69		
		EB	Average	29.08	11.7	47.53	983.47	B	
			Standard Deviation	0.66	0.51	2.35	16.77		
	2	WB (Detour)	Average	70.47	58.08	86.3	1176.95	E	
			Standard Deviation	3.82	3.48	1.27	20.45		
		EB	Average	53.45	44.48	75.29	957.97	D	
			Standard Deviation	1.02	1	0.92	11.79		
	3	SB (Detour)	Average	37.12	12.51	48.53	1712.32	B	
			Standard Deviation	0.83	0.74	2.58	19.17		
		NB	Average	30.06	16.5	63.06	438.5	B	
			Standard Deviation	0.99	0.94	1.89	15.26		
	4	SB (Detour)	Average	21.29	10.94	34.85	1302.4	B	
			Standard Deviation	1.06	0.9	2.03	21.76		
		NB	Average	10.81	4.95	20.99	520.59	A	
			Standard Deviation	0.56	0.54	1.27	9.38		
	5	SB (Detour)	Average	16.56	10.03	19.72	1245.29	B	
			Standard Deviation	0.64	0.55	1.25	22.1		
		NB	Average	25.41	15.59	38.99	429.43	B	
			Standard Deviation	1.41	1.37	2.87	10.56		
6	SB (Detour)	Average	25.43	13.5	49.85	1113.38	B		
		Standard Deviation	2.43	2.03	2.85	18.23			
	NB	Average	20.28	11.61	52.04	591.99	B		
		Standard Deviation	0.93	0.89	2.76	15.15			

7	SB (Detour)	Average	69.65	55.03	86.2	933.41	E
		Standard Deviation	5.42	4.75	1.63	16.82	
	NB	Average	52.53	32.06	74	863.73	C
		Standard Deviation	1.1	1.03	1.23	11.55	
8	EB (Detour)	Average	29.32	18.49	59.51	895.07	B
		Standard Deviation	1.21	1.15	2.27	20.71	
	WB	Average	37.32	16.15	59.29	455.54	B
		Standard Deviation	0.81	0.8	1.94	12.91	

Note: * LOS is determined according to highway capacity manual 2010 [36].

Table 25 shows comparison between results of Scenarios 0 and 1. For intersection 2, except volume, all performance measures in detour direction, opposite direction of detour direction and both directions are significantly improved in scenario 1. With respect to intersection 7, in Scenario 1, performances of diversion direction are improved, although performances of its opposite direction are decreased slightly. For both directions, performance measures of intersection 7 in Scenario 1 are significantly enhanced, as well. For the LOS of the detour direction, both intersections 2 and 7 in Scenario 1 improves from E to C after signal timing plans are optimized. For the total 8 coordinated intersections, all performances measures of Scenario 1 in the diversion direction and both directions are notably improved than that of Scenario 0. Control delay per vehicle of Scenario 1 reduces 7.77 seconds (34.6%) in the detour direction and 4.22 seconds (19.0%) in both directions than that of Scenario 0. For the entire arterial, LOS of the detour direction and both directions are improved from C to B, compare Scenarios 0 with 1. Thus, we can find that, after signal timing plans of the diversion arterial optimized, performances of bottleneck intersections and the entire arterial are significantly enhanced. The signal optimization or retiming for the parallel signalized arterial of ICM corridor is crucial for ICM strategies.

Table 25 Performance Measures of Scenario 0 and Scenario 1

Intersection	Direction		Travel Time Per Vehicle	Control Delay Per Vehicle	Stopped Vehicles Percent	Volume	LOS*
		Units	Seconds/Vehicle	Seconds/Vehicle	%	vph	
2	WB (Detour)	Scenario 1	39.9	28.02	77.85	1190.75	C
		Scenario 0	71.27	59.01	87.96	1204.63	E
		Difference	-31.37	-30.99	-10.11	-13.88	
		Difference (%)	-44.0%	-52.5%	-11.5%	-1.2%	
	EB	Scenario 1	55.36	45.66	84.08	959.22	D
		Scenario 0	54.72	45.65	76.5	958.33	D
		Difference	0.64	0.01	7.58	0.89	
		Difference (%)	1.2%	0.0%	9.9%	0.1%	
	Both Direction	Scenario 1	46.8	35.89	80.63	2149.97	D
		Scenario 0	63.94	53.09	82.88	2162.96	D
		Difference	-17.14	-17.2	-2.25	-12.99	
		Difference (%)	-26.8%	-32.4%	-2.7%	-0.6%	

		(%)					
7	SB (Detour)	Scenario 1	37.78	25.93	75.44	971.76	C
		Scenario 0	76.37	60.75	89.08	951.61	E
		Difference	-38.59	-34.82	-13.64	20.15	
		Difference (%)	-50.5%	-57.3%	-15.3%	2.1%	
	NB	Scenario 1	69.52	48.66	85.57	862.94	D
		Scenario 0	52.86	32.31	73.5	870.11	C
		Difference	16.66	16.35	12.07	-7.17	
		Difference (%)	31.5%	50.6%	16.4%	-0.8%	
	Both Direction	Scenario 1	52.71	36.62	80.2	1834.71	D
		Scenario 0	65.14	47.17	81.64	1821.72	D
		Difference	-12.43	-10.55	-1.43	12.99	
		Difference (%)	-19.1%	-22.4%	-1.8%	0.7%	
Total 8 intersections	Detour Direction	Scenario 1	27.33	14.7	47.56	10031.55	B
		Scenario 0	35.69	22.47	53.05	10026.74	C
		Difference	-8.37	-7.77	-5.49	4.81	
		Difference (%)	-23.4%	-34.6%	-10.4%	0.0%	
	Both Direction	Scenario 1	30.89	17.93	50.56	15279.96	B
		Scenario 0	35.49	22.15	54.1	15270.67	C
		Difference	-4.6	-4.22	-3.54	9.28	
		Difference (%)	-12.9%	-19.0%	-6.5%	0.1%	

Note: * LOS is determined according to highway capacity manual 2010 [36].

To evaluate the effectiveness of the proposed proactive real time offset tuning algorithm when it was running in offline mode, Scenario 2 is conducted to evaluate the performance of TRANSYT 7F if average additional diversion volumes are considered in TRANSYT-7F. Table 26 shows results from Scenarios 1 and 2. In Table 26, in addition to intersections 2 and 7, performance measures of intersection 6 are also listed, since significant performance differences appear between Scenarios 1 and 2 at this intersection. Comparing results of Scenarios 1 and 2, the travel time per vehicle and control delay per vehicle at the detour direction and both directions of intersections 2 and 7 are increased in the proposed algorithm. Control delays at intersection 6 for diversion directions and both directions are significantly enhanced by applying the proposed algorithm. More importantly, for the total 8 coordinated intersections, control delay per vehicle of Scenario 2 increases 0.33 second (2.2%) in detour direction and 0.49 second (2.7%) in both directions than that of Scenario 1. It is indicated that the performance of TRANSYT-7F slightly leads that of the proposed algorithm when the diversion rates are accurately incorporated into TRANSYT-7F. The margin of performance is narrow, less than 3%.

There are several reasons that one of the performance indexes, the control delay, of the proposed algorithm cannot outweigh TRANSYT-7F when accurate diversion rates are known in advance. First and the foremost, the proposed algorithm only consider the one way coordination without the considerations the non-detour direction and without considering the split and cycle length while TRANSYT-7F optimizes all of those variables. Second, TRANSYT-7F directly optimizes the delays generated from simulation and use the same simulation to evaluate the outcomes of the optimization. Our proposed algorithm separates offset turning model and the evaluation model.

When TRANSYT-7F is applied in real world, the evaluation and the optimization models will be separated, the performance of TRANSYT-7F will definitely be degraded. Third, our proposed algorithm is designed for real time implementation and the incorporating TRANSYT-7F into real time optimization would be challenging. Fourth, for the proposed algorithm, it doesn't need to know accurate diversion rates for offset tuning while TRANSYT 7F does. In real world, accurate diversion rates are nearly impossible to obtain in advance currently. In the section 4, the case study already shows that the proposed algorithm could obviously outperform Transy-7F when no diversion rates information is available. Last but not the least, the optimization model of TRANSYT-7F is delay based model and control delay is selected as the performance index for optimization. While, the objective function of the proposed algorithm is maximizing number of arrivals on green. So, it favors the reductions in the number of stops.

Despite the control delay narrow gap, the proposed algorithm does have significant advantages over TRANSYT-7F. First of all, the proposed algorithms provide offsets to accommodate the maximum number of arrivals on green; therefore, significantly number of stops is reduced. From Table 26, with respect to stopped vehicle percent, Scenario 2 decreases 8.6% in detour direction and 5.0% in both directions for total 8 coordinated intersections than that of Scenario 1. It will provide more uniform drive experiences, less fuel consumption and emissions. In addition, for the control delay per vehicle, there is less than 1 second difference in detour direction and both directions between Scenarios 1 and 2. It is nearly no difference for drivers between the proposed algorithms and TRANSYT-7F. Second, TRANSYT-7F is an offline optimization model which focuses to obtain optimal results. TRANSYT-7F's genetic algorithm is not considered to be a real time algorithm. For the proposed algorithm, it is utilized for real time traffic signal control and its computation time is ideal for real time implementation.

Table 26 Performance Measures of Scenario 1 and Scenario 2

Intersection	Direction		Travel Time Per Vehicle	Control Delay Per Vehicle	Stopped Vehicles Percent	Volume	LOS*
		Units	Seconds/Vehicle	Seconds/Vehicle	%	vph	
2	WB (Detour)	Scenario 2	50.9	39.16	76.37	1188.06	D
		Scenario 1	39.9	28.02	77.85	1190.75	C
		Difference	11	11.14	-1.48	-2.69	
		Difference (%)	27.6%	39.8%	-1.9%	-0.2%	
	EB	Scenario 2	53.87	44.26	83.87	957.88	D
		Scenario 1	55.36	45.66	84.08	959.22	D
		Difference	-1.49	-1.4	-0.21	-1.34	
		Difference (%)	-2.7%	-3.1%	-0.2%	-0.1%	
	Both Direction	Scenario 2	52.23	41.44	79.72	2145.94	D
		Scenario 1	46.8	35.89	80.63	2149.97	D
		Difference	5.43	5.55	-0.91	-4.03	
		Difference (%)	11.6%	15.5%	-1.1%	-0.2%	
6	SB (Detour)	Scenario 2	24.55	13.89	38.63	1118.65	B
		Scenario 1	37.4	24.52	75.5	1129.39	C
		Difference	-12.85	-10.63	-36.87	-10.74	
		Difference	-34.4%	-43.4%	-48.8%	-1.0%	

		(%)					
	NB	Scenario 2	19.31	10.88	34.7	588.43	B
		Scenario 1	16.72	8.38	28.17	592.91	A
		Difference	2.59	2.5	6.53	-4.48	
		Difference (%)	15.5%	29.8%	23.2%	-0.8%	
	Both Direction	Scenario 2	22.75	12.85	37.28	1707.08	B
		Scenario 1	30.28	18.97	59.2	1722.3	B
		Difference	-7.54	-6.11	-21.93	-15.23	
		Difference (%)	-24.9%	-32.2%	-37.0%	-0.9%	
7	SB (Detour)	Scenario 2	38.81	27.5	72.94	971.32	C
		Scenario 1	37.78	25.93	75.44	971.76	C
		Difference	1.03	1.57	-2.5	-0.44	
		Difference (%)	2.7%	6.1%	-3.3%	0.0%	
	NB	Scenario 2	74.81	53.9	87.84	862.05	D
		Scenario 1	69.52	48.66	85.57	862.94	D
		Difference	5.29	5.24	2.27	-0.89	
		Difference (%)	7.6%	10.8%	2.7%	-0.1%	
	Both Direction	Scenario 2	55.74	39.91	79.95	1833.36	D
		Scenario 1	52.71	36.62	80.2	1834.71	D
		Difference	3.03	3.29	-0.26	-1.34	
		Difference (%)	5.7%	9.0%	-0.3%	-0.1%	
Total 8 intersections	Detour Direction	Scenario 2	27.39	15.03	43.49	10029.76	B
		Scenario 1	27.33	14.7	47.56	10031.55	B
		Difference	0.07	0.33	-4.07	-1.79	
		Difference (%)	0.3%	2.2%	-8.6%	0.0%	
	Both Direction	Scenario 2	31.21	18.42	48.06	15224.88	B
		Scenario 1	30.89	17.93	50.56	15279.96	B
		Difference	0.31	0.49	-2.51	-55.08	
		Difference (%)	1.0%	2.7%	-5.0%	-0.4%	

Note: * LOS is determined according to highway capacity manual 2010 [36].

Considering the significant reduction in number of stops, the narrow gaps in control delays and other benefits stated above, the proposed algorithm is suggested for MDOT use for offline mode. Moreover, the proposed algorithm will be continuously and further improved and evaluated, for example, to adjust the offset to benefit detour and non-detour directions simultaneously. Table 27 shows the performance measures between Scenarios 0 and 2 with respect to total 8 coordinated intersections. From Table 27, all performance measures of Scenario 2, except volume, in detour direction and both directions are notably improved than those in Scenario 0. Control delay per vehicle is the most significantly improved performance measure. Compare to Scenario 0, Scenario 2 saves 31.85 hours control delay in both directions for the entire accident period. The LOS of the diversion arterial improves from C to B in the detour direction and both directions by implementing the proposed algorithm.

Table 27 Total 8 Intersections' Performance Measures between Scenario 0 and 2

Direction			Travel Time Per Vehicle	Control Delay Per Vehicle	Stopped Vehicles Percent	Volume	LOS*
			Seconds/Vehicle	Seconds/Vehicle	%	vph	
Total 8 intersections	Detour Direction	Scenario 2	27.39	15.03	43.49	10029.76	B
		Scenario 0	35.69	22.47	53.05	10026.74	C
		Difference	-8.3	-7.44	-9.56	3.02	
		Difference (%)	-23.3%	-33.1%	-18.0%	0.0%	
	Both Direction	Scenario 2	31.21	18.42	48.06	15224.88	B
		Scenario 0	35.49	22.15	54.1	15270.67	C
		Difference	-4.28	-3.73	-6.04	-45.79	
		Difference (%)	-12.1%	-16.8%	-11.2%	-0.3%	

Note: * LOS is determined according to highway capacity manual 2010 [36].

As mentioned before, CORSIM uses yield point to coordinated signals. The yield points generated by the proposed algorithm and their corresponding offsets for each of 8 coordinated intersections for the entire diversion periods are listed in Table 28. In Table 28, although the diversion happened at 900 seconds, the proposed algorithm starts to be implemented at 1500 seconds. It is delayed 600 seconds to collect real-time diversion data. After that, the proposed algorithm is implemented repeatedly for every 10 coordination cycle length, i.e. 1020 seconds. Aforementioned, the proposed algorithm is applied based on the signal timing optimized by TRANSYT-7F. The initial offset of each coordinated intersection in Table 28 are optimized by TRANSYT-7F. The feasible offsets range in CORSIM is from 0 second to one second before the end of cycle length. The viable offsets range in Table 28 is from 0 to 101 second.

Table 28 Offsets and Yield Points of Coordinated Intersections Generated by the Proposed Algorithm

Intersection		Initial Value*	Projection Horizon 1	Projection Horizon 2	Projection Horizon 3	Projection Horizon 4	Projection Horizon 5	Projection Horizon 6	Projection Horizon 7	Projection Horizon 8
		Start Time (second)	0	1500	2520	3540	4560	5580	6600	7620
1	Offset	34	30	93	41	75	46	40	44	17
	Yield Point	86	82	43	93	25	98	92	96	69
2	Offset	42	100	61	9	43	14	8	12	87
	Yield Point	68	24	87	35	69	40	34	38	11
3	Offset	79	59	95	42	76	53	41	45	31
	Yield Point	98	78	12	61	95	72	60	64	50
4	Offset	93	69	3	52	84	63	50	55	41
	Yield Point	47	23	59	6	38	17	4	9	97
5	Offset	21	2	38	87	17	98	85	90	76
	Yield Point	46	27	63	10	42	21	8	13	101
6	Offset	0	4	61	9	43	14	8	12	93
	Yield Point	20	24	81	29	63	34	28	32	11
7	Offset	8	19	74	16	58	29	22	27	96
	Yield Point	41	52	5	49	91	62	55	60	27
8	Offset	9	37	73	19	69	34	31	22	2
	Yield Point	52	80	14	62	10	77	74	65	45

*Note: Initial offset of intersections 1-8 are optimized by TRANSYT-7F.

In general, for traffic signal coordination, traffic signal engineers and researchers are more concerned about the offset or yield point difference between two successive intersections, i.e. difference of green start or end times between two successive intersections. In this report, the yield point difference between two successive intersections is referred to as internal yield points. Then, after the yield point of the first intersection is selected, yield points of all other intersections could be calculated. Table 29 shows internal yield points among 8 intersections based on Table 28. Table 29 shows 9 yield points for each intersection. Although use multiple yield points could better adapt variations of detour traffic, it is not convenient for manually implementation in the real world signal system during diversion. Thus, only one yield point for each intersection is recommended for utilization to build the lookup table. We can easily selected internal yield points between intersections 2 and 1, 3 and 2, 4 and 3, 5 and 4 as well as 7 and 6, based on number of occurrence for each internal yield point. With respect to internal yield points between intersection 6 and 5, 19 appear twice and it close to yield points 18, 20 and 21. 19 is selected for utilization as the internal yield point for intersection 6 and 5. For intersections 8 and 7, internal yield points are random. To avoid transition delays due to implement new offset, 11 is chosen as the internal yield point between intersections 8 and 7 for implementation.

Table 29 Internal Yield Points among 8 Coordinated Intersections

	From Intersection	To Intersection	Initial	1	2	3	4	5	6	7	8	Selected
Internal Yield Points (second)	2	1	84	44	44	44	44	44	44	44	44	44
	3	2	30	54	27	26	26	32	26	26	39	26
	4	3	51	47	47	47	45	47	46	47	47	47
	5	4	101	4	4	4	4	4	4	4	4	4
	6	5	76	99	18	19	21	13	20	19	12	19
	7	6	21	28	26	20	28	28	27	28	16	28
	8	7	11	28	9	13	21	15	19	5	18	11

Since the proposed algorithm also fine-tuned offsets of the first intersection instead of using a fixed offset, the yield points for the first intersection are varied according to upcoming traffic which is shown in Table 28. For traffic signal coordination in our network, the first intersection functions as a gate to form platoons. Then, we can generate yield points for all 8 intersections, as shown in Table 30. Although this case shows the procedures to select appropriate yield points. Offset and yield point can be interchangeable. Implementing offsets or yield points depends on field signal system requirements.

Table 30 Recommend Yield Points for 8 Intersections under Diversion

Intersection	1	2	3	4	5	6	7	8
Yield point (second)	86	28	54	101	3	22	50	61

Because of funding and time constraints, in section 5, we use a case study to show how to select offsets or yield points generated by the proposed algorithm when diversion occurs for a chosen accident scenario. Following the same approach, we can obtain an implementable lookup table after we run simulations for different capacity reduction scenarios. We will discuss with MDOT for selecting typical accident scenarios to build a complete lookup table for MDOT use, when

funding is available in the future. In the meantime, the proposed algorithm will be continuously improved.

6. BENEFIT STUDY

In Sections 4 and 5, the proactive real time offset tuning algorithm is proposed for real time traffic signal control when diversions happen. It is also recommended as a tool to create an expert system for predetermined frequently appeared incident scenarios. In this section, the benefits of implementing the proposed offset tuning algorithm are estimated. The costs for improvements of the study ICM corridor, e.g. install upstream detectors and add new turning bays, in order to apply the algorithm are estimated, as well.

6.1 Benefits

6.1.1 Benefits of Delay Time Saving

6.1.1.1 Delay Time Saving for the Study ICM Corridor Due to Incidents

Based on Table 27, with respect to control delay per vehicle, the proactive real time offset tuning algorithm could save 16.8 % for both directions than that of existing signal timing plans. For the case study of Section 5, a total of 31.85 hours control delay in both directions. That is based on a 119 minutes accident at the freeway and an implementation of the proposed algorithms on arterials. It is assumed that control delay saving is linearly change with the accident duration time. It is also assumed that the proposed algorithm is applied when an accident maybe lasts over 60 minutes. In reality, very few drivers would choose to detour, if an accident could be cleared quickly. The few detour traffic due to an accident with short duration is expected to accommodate well by the existing traffic signal timing. According to accident data provided by MDOT (Amrik Singh, unpublished data), the annual number of accidents happened on I-55 which lasts over 60 minutes is 169 and the average duration of these accidents is 105 minutes. So, the total annual control delay saving for implementing proposed algorithm for ICM strategies could be estimated as below:

$$\frac{31.85}{134} * 105 * 169 = 4,217.75 \text{ hours}$$

6.1.1.2 Delay Time Saving for State St

The proposed algorithms and improvement on roadway, traffic surveillance and traffic control device will benefit the peak hour traffic significantly, as evidenced below.

In addition to implement the proposed algorithm for ICM strategies, the algorithm also could be used for coordination on State St during daily rush hours to decrease delays. It is assumed that applying the proposed algorithm for coordination on State St could also save 16.8% of control delay per vehicle for both directions than that of existing traffic signal timing plans. Table 31 shows that the total hourly control delays for coordinated directions (SB and NB) of all signalized intersections on State St is 593,237.5 seconds (164.79 hours). It is assumed that the proposed algorithms will be used for coordination of State St during morning, noon and afternoon rush hours of workdays. According to data provided by a traffic engineer of MDOT, the morning, noon and afternoon rush hours of State St are 7 a.m. to 8 a.m., 11:30 a.m. to 12:30 p.m., and 4 p.m. to 5 p.m., respectively (Amrik Singh, unpublished data). Since the noon peak hour is not as critical as morning and afternoon rush hours, half of noon peak hour is considered for estimating benefits of using the proposed algorithm. So, the total time duration for daily peak hours of State

St for benefit-cost study are 2.5 hours. Based on these data, the annual delay time saving for implementing the proposed algorithm for coordination on State St could be calculated as below:

$$164.79 * 16.8\% * 2.5 * 5 * 52 = 17995.07 \text{ hours}$$

Table 31 Total Hourly Control Delays and Volume for SB and NB of All Signalized Intersections on State St

#	Intersections		SB	NB	Total Control Delay for Both Directions (second)
1	State St & County Line Rd	Control Delay	20.7	14.5	14865.1
		Volume	543	250	
2	State St & Beasley Rd	Control Delay	30.8	27.5	31523.8
		Volume	586	490	
3	State St & Briarwood Dr	Control Delay	2.8	1.5	1313.5
		Volume	295	325	
4	State St & Culley Dr	Control Delay	3.6	5.3	3498.1
		Volume	452	353	
5	State St & Cedars of Lebanon Rd	Control Delay	3.5	6.1	3352.8
		Volume	395	323	
6	State St & Northside Dr	Control Delay	18.6	11.9	12134.3
		Volume	360	457	
7	State St & Meadowbrook Rd	Control Delay	26.9	33.6	29133
		Volume	426	526	
8	State St & Duling Ave	Control Delay	7.7	6.4	6751.9
		Volume	443	522	
9	State St & Old Canton Rd	Control Delay	15.0	17.1	24308.1
		Volume	468	1011	
10	State St & Woodrow Wilson Ave	Control Delay	62.1	45.5	108540.2
		Volume	1402	472	
11	State St & Riverside Dr	Control Delay	30.9	35.2	60797.5
		Volume	1259	622	
12	State St & Bellhaven St	Control Delay	16.5	19.9	22150.3
		Volume	538	667	

13	State St & Webster St	Control Delay	4.3	13.9	11964.3
		Volume	539	694	
14	State St & Pinehurst St	Control Delay	8.7	6.8	9194.2
		Volume	530	674	
15	State St & Manship St	Control Delay	13.1	12.9	17701.5
		Volume	594	769	
16	State St & Fortification St	Control Delay	31.3	33.5	41523.5
		Volume	510	763	
17	State St & High St	Control Delay	21.9	19.6	26957.5
		Volume	685	610	
18	State St & Mississippi St	Control Delay	7.8	7.4	9006.6
		Volume	557	630	
19	State St & Emite St	Control Delay	11.9	21.8	29018.9
		Volume	559	1026	
20	State St & Capitol St	Control Delay	7.1	8.9	8744
		Volume	413	653	
21	State St & Pearl St	Control Delay	17.2	25.0	31033.2
		Volume	781	704	
22	State St & Pascagoula St	Control Delay	38.7	39.4	53362.2
		Volume	542	822	
23	State St & Tombigbee Rate	Control Delay	21.8	29.2	36363
		Volume	421	931	
Total Hourly Control Delays for SB and NB of All Signalized Intersections on State St (second)					593237.5

6.1.1.3 Total Annual Benefits of Delay Time Saving

According to the report of a previous study of the research team, operating cost per hour for private occupancy vehicles (POV) and truck are \$26.15 and \$83.68, respectively[37]. It is assumed that no trucks choose to divert to State St when an accident happened on I-55 and 5% trucks travels on State St. Therefore, the annual benefits of delay time saving are computed below:

$$4,217.75 * 26.15 + 17995.07 * (0.95 * 26.15 + 0.05 * 83.68) = \$632,628$$

6.1.2 Benefits of Number of Stops Reduction

A previous study presented that the monetary cost of each vehicle stop is \$0.014 [38]¹. Based on Table 31, we can obtain the total hourly volumes of State St for both SB and NB is 27,592 vph. Table 27 shows a decrease of 6.04 % with respect to stopped vehicle percent by applying the proposed algorithm than the existing signal timing plans. It is assumed that the same percentage drop achieved when the proposed algorithm uses for traffic signal coordination during rush hours on State St. The same method is used for calculating annual benefits of number of stops decrease for the study ICM corridor and State St.

Total annual saving for the ICM corridor due to number of stops decrease:

$$15225 * 6.04 \% * \frac{105}{60} * 169 * \$0.014 = \$3,808$$

Total annual saving for State St due to number of stops reduction:

$$27,592 * 6.04 \% * 2.5 * 5 * 52 * \$0.014 = \$15166$$

Total annual saving with respect to number of stops reduction:

$$\$3,808 + \$15166 = \$18,974$$

6.1.3 Total Annual Benefits for Implementing the Propose Algorithm

As calculated above, the total annual benefits of applying the proposed algorithm for delay time saving and number of stops reduction are \$632,628 and \$18,974, respectively. The total monetary saving for performing the proposed algorithm is \$651,602.

6.2 Costs

There are two types of improvement costs for this project: 1) the cost for adding of new turning bays for increasing capacities of diversion routes to accommodate detour traffic; 2) the cost for installing upstream detectors in order to implement the proactive real time offset tuning algorithm. These costs are analyzed as following.

6.2.1 Costs for Adding New Turning Bays

From the case study of Section 5, we knows that an additional left turn bay needs to be added at intersections which detour vehicles make left turn. In the case study of Section 5, the accident happened on SB of I-55 and SB of State St is selected as the diversion route. When the accident occurred on NB of I-55, the NB of State St would be used as the detour route. In this situation, an additional right turn bay needs to be added for intersections on the diversion path which detour traffic makes a right turn. Table 32 shows intersections on potential diversion routes in the study area which are needed to add turning bays. A total of 37 turning bays are needed to add for detour traffic.

¹ The cost of \$0.014 per stop is obtained from a reference and that data is based on information about 10 years ago. We feel the cost is higher in recent years due to gas price hike. Therefore, the benefit from stop reduction might be much higher.

Table 32 Add/Extending Turning Bays for Intersections on Potential Diversion Routes

Intersection	SB	NB	WB	Total
State St @ Pascagoula St	1L			1
State St @ Pearl St			1L, 1R	2
State St @ High St	1L	1R	1L, 1R	4
State St @ Fortification St	1L	1R	1L, 1R	4
State St @ Woodrow Wilson St	1L	1R	1L, 1R	4
Old Canton Rd @ Lakeland Dr		1 R	1L	2
State St @ Meadowbrook	1L	1R	1R	3
State St @ Northside Dr	1L	1R	1L, 1R	4
State St @ Briarwood Dr	1L	1R	1L, 1R	4
State St @ Beasley Rd	1L	1R	1L, 1R	4
State St @ CountyLine Rd	1L	1R	1L, 1R	4
Total turning bays need to be added				36

According to a document from Texas A&M University (TAMU), the cost of adding new lanes for large surface streets is \$750,000 per lane-mile [39]. It is assumed that the length of adding an additional turning bay for intersections on potential diversion routes is on average 500 ft. The cost of adding turning bays for improving capacities of diversion routes is calculated below:

$$\frac{36 * 500}{5280} * 750,000 = \$2,556,818$$

6.2.2 Costs for Installing Upstream Detectors

By street view of Google Map (<https://maps.google.com/>), the research team identify that, in surface streets, 21 intersections needs to install upstream detectors for three approaches and 36 intersections need to install upstream detectors for two approaches. Also from street view of Google Map, 16 off-ramps need to install upstream detectors in which 7 off-ramps is one lane and 9 off-ramps are two lanes. For intersections of surface streets, the number of lanes in an approach of most intersections is 2. Some intersections have more than 2 lanes for each approaches. So, it is assumed that the average number of lanes on an approach for intersections of surface streets is 3.

According to data provided by a traffic engineer of MDOT, the installation cost for new loop detector without communication is \$3000/lane and communication cost for each intersection to obtain second by second detector data in Traffic Management Center is \$1000--\$5000 (Amrik Singh, unpublished data). In this study, the average value, \$3000, is used for the average communication cost for each intersection and an off-ramp. The costs for installation new loop detectors, communication cost and their total cost are calculated below, respectively.

Installation cost: $(21 * 3 * 3 + 36 * 2 * 3 + 7 * 1 + 9 * 2) * 3000 = \$1,290,000$

Communication cost: $(21 + 36 + 16) * 3000 = \$219,000$

Total cost: $\$1,290,000 + \$219,000 = \$1,509,000$

6.3 Benefit and Cost Analysis

After the benefits and costs of implementing the proactive real time offset tuning algorithm are computed, respectively, 10 year's net benefit and 10 year's benefit to cost ratio are computed. The results are listed below:

$$10 \text{ year's net benefit: } 651,602 * 10 - (2,556,818 + 1,509,000) = \$2,450,202$$

$$10 \text{ year's benefit to cost ratio: } \frac{651,602 * 10}{2,556,818 + 1,509,000} = 1.6$$

Compared with the same data of Dallas, TX; Minneapolis, MN; and San Diego, CA in Section 2 literature review from the website of RITA of U.S.DOT [16], the 10 year's net benefits and 10 year's benefits to cost ratio of the study corridor are smaller than that of these three pioneer sites. There are three major reasons caused this situation. First, the benefits of emission and fuel consumption are not estimated due to lack of enough data. Second, traffic data from installation of new upstream detectors could be used for other ITS technologies which could bring more benefits. Third, Mississippi is a rural state and traffic volume is not as heavy as Texas, Minnesota, and California. However, the 10 year's net benefits of applying the proposed algorithm for the study corridor is nearly \$2.5 million which indicates the proposed algorithm is worth for application in the field.

7. UPCOMING ICM SIMULATION TESTBED

7.1 Introduction

The enhanced traffic Flow Open-source Microscopic Model (ETFOMM) is a new traffic simulation software funded by USDOT and is produced by a small business in Mississippi. We replace CORSIM with ETFOMM to implement and validate our proactive real time offset tuning algorithm. Furthermore, we are creating a future ICM development platform and test based on ETFOMM.ETFOMM includes simulation engine DLL, communication Application Programming Interface (API), input editor, and a 3D animator. The ETFOMM could be used on distributed/cross platform (IOS/LINUX/MacOS/Android) and achieve Internet data communication by multiple network protocol (TCP/IP, HTTP). In addition, ETFOMM could be integrated with almost any programming language such as C, C++, FORTRAN, PHP, VB, JAVA, and so on. Figure 20 displays ETFOMM software components.

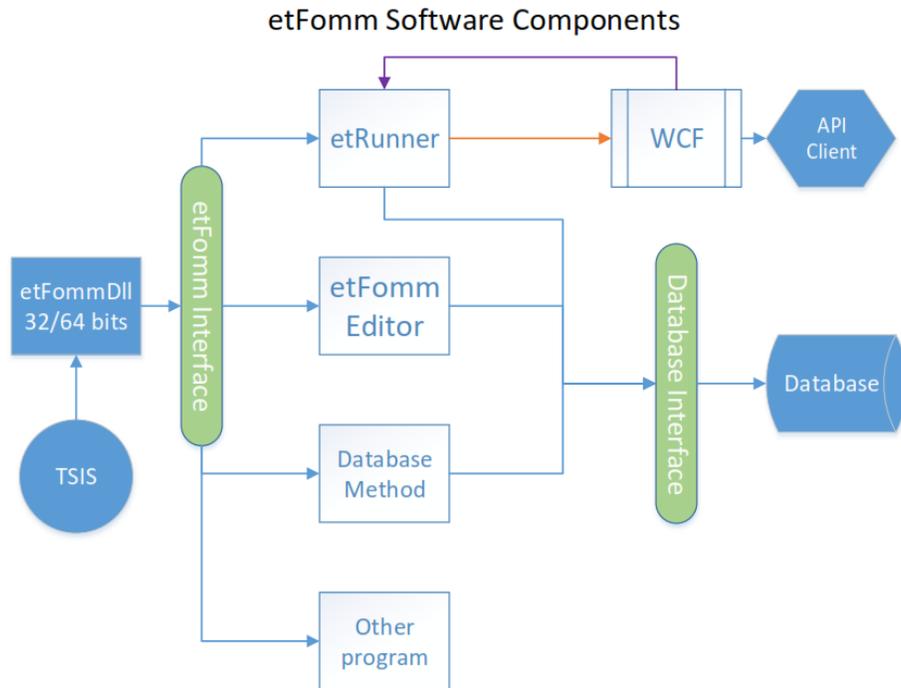


Figure 20 ETFOMM Architecture

The ETRunner is the control program of ETFOMM DLL and API. The responsibilities of ETRunner are getting network inputs from ETFOMM API, passing TRF files to ETFOMM DLL, and control WCF Server. The ETFOMM API is used WCF technology to provide functions for data exchange between ETRunner and user definition program (API Client application).

7.2 Approach

We replace CORSIM with ETFOMM to implement and validate the proposed proactive real time offset tuning algorithms. We only use ETFOMM.dll for core traffic simulator and ETFOMM API for data communication tool.

We choose C++ to implement the proposed proactive real time offset tuning algorithms as ETFOMM app. The ETFOMM API provides simple, completed, and flexible functions to users. We could directly use ETFOMM API functions to implement our algorithm. We only develop a client application to implement the proposed algorithm.

The data structure and functions of ETFOMM are directly used in the proactive real time offset tuning algorithm. The data we need to exchange includes traffic network information, freeway link information, street link information, vehicle information, and traffic signal information. The data structure of ETFOMM shows below:

```
public value struct WCF_NETWORK_INPUTS { ... }  
public value struct WCF_FTC_DATA {... }  
public value struct WCF_PHASE_DATA {... }  
public value struct WCF_AC {... }  
public value struct WCF_DETECTOR_DATA {... }  
public value struct Wcf_freeway_link {... }  
public value struct WCF_FREEWAY_NETWORK_INPUTS {... }  
public value struct Wcf_street_link {... }  
public value struct WCF_STREET_NETWORK_INPUTS {... }  
public value struct WCF_ENTRYNODES_DATA {... }  
public value struct WCF_VFData {... }  
public value struct WCF_VSData {... }  
public value struct WCF_VEHICLE_TYPE_DATA {... }
```

The attributes of data in ETFOMM are easy to understand and flexible to use. Based on ETFOMM API, we use set/get functions of data. Figure 21 demonstrates data flow of the application. In the ETFOMM application, the network information, freeway link information, and street link information are read at the begging of the simulation only once. After initialization of ETFOMM and API, the exchange data between ETFOMM DLL and application are vehicle detector information and traffic signal information.

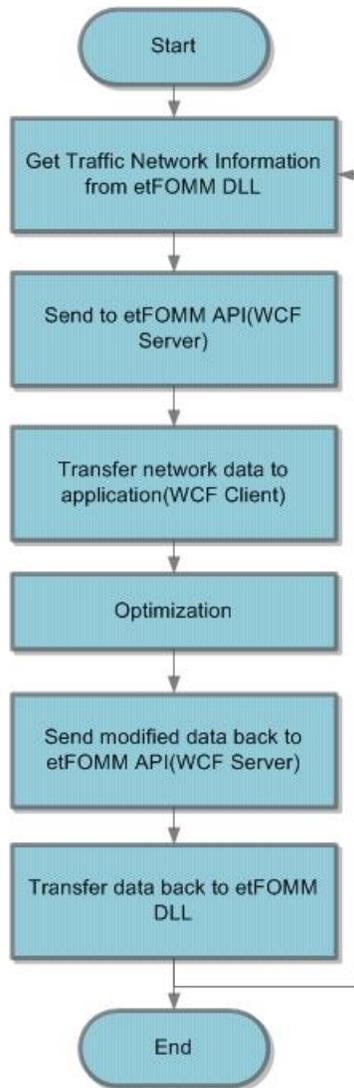


Figure 21 Data Flow of ETFOMM

We use functions from ETFOMM API, or the functions in the WCF Server interface. The functions' details show below:

```

[OperationContract]
int GetClientState();
[OperationContract]
void SetClientState(int s);
[OperationContract]
float GetServerTimestep();
[OperationContract]
void SetServerTimestep(float t);
//timestep interval
[OperationContract]
float GetAPITimestepInterval();
[OperationContract]
void SetAPITimestepInterval(float s);
  
```

The time step and client state functions are basic configuration functions in ETFOMM API. We call these functions as ETFOMM default example.

```

[OperationContract]
void SetServerNetworkInput(array <WCF_NETWORK_INPUTS> ^ wcf_network_inputs);
[OperationContract]
array <WCF_NETWORK_INPUTS>^ GetServerNetworkInput();
[OperationContract]
void SetServerFreewayNetworkInput(array <WCF_FREEWAY_NETWORK_INPUTS> ^
wcf_freeway_network_inputs);
[OperationContract]
array <WCF_FREEWAY_NETWORK_INPUTS>^ GetServerFreewayNetworkInput();
[OperationContract]
void SetServerStreetNetworkInput(array <WCF_STREET_NETWORK_INPUTS> ^
wcf_street_network_inputs);
[OperationContract]
array <WCF_STREET_NETWORK_INPUTS>^ GetServerStreetNetworkInput();
//freeway links
[OperationContract]
void SetServerFreewayData(array <Wcf_freeway_link> ^ wcf_freeway_links);
[OperationContract]
array <Wcf_freeway_link>^ GetServerFreewayData();
//surface street links
[OperationContract]
void SetServerStreetData(array <Wcf_street_link> ^ wcf_street_links);
[OperationContract]
array <Wcf_street_link>^ GetServerStreetData();
//conditional turn percentages
[OperationContract]
void SetServerCondTurnpctData(array <WCF_COND_TURNPCTS> ^
wcf_cond_turnpct_data);
[OperationContract]
array <WCF_COND_TURNPCTS>^ GetServerCondTurnpctData();
[OperationContract]
void SetServerEntryNodeData(array <WCF_ENTRYNODES_DATA> ^ wcf_entry_nodes);

```

The network data, freeway link data, and street link data are transferred once in the initialization of application.

```

//Fix Time control
[OperationContract]
void SetServerFTCSignalData(array<WCF_FTC_DATA> ^ wcf_ftc);
[OperationContract]
array <WCF_FTC_DATA>^ GetServerFTCSignalData();
//Actuated Control
[OperationContract]
void SetServerACData(array <WCF_AC>^ wcf_acl);
[OperationContract]
array <WCF_AC>^ GetServerACData();
[OperationContract]
void SetServerFVehicleData(array<WCF_VFData>^ wcf_fveh);
[OperationContract]
array<WCF_VFData>^ GetServerFVehicleData();
[OperationContract]
void SetServerSVehicleData(array<WCF_VSData>^ wcf_sveh);
[OperationContract]
array<WCF_VSData>^ GetServerSVehicleData();

```

The traffic signal and vehicle detection functions are called at every simulation steps.se. the usage of these functions is very simple. In addition, it is much easier to use than other traffic simulation software, especially CORSIM RTE.

7.3 Simulation Result

Three scenarios are considered in the traffic simulation. Scenario 0 is existing signal timing plans. 1 is signal timing plans optimized by TRANSYT-7F. Scenario 2 is proactive real time offset tuning algorithm. The scenario 1 is used different input file with scenario 0 and 2. And scenario 0 and 2 use the same input file. All scenarios' simulations use the same random number of seed file. Table 33 shows travel time range in ETFOMM. The scenario 0 is as same as real data. It demonstrates ETFOMM simulation is as same as real roadway state.

Table 33 Travel Time of Real Roadway and ETFOMM Simulation

Woodrow Wilson Ave to Fortification St	Travel Time
Real Travel Time	96(sec)
ETFOMM Simulation	69.99(sec)

Table 34 displays total 8 intersections' key performance measures in ETFOMM simulation.

Table 34 Simulation Results of Three Scenarios in ETFOMM

Intersection	Direction		Travel Time Per Veh	Delay Travel Per Veh	Control Delay Per Veh	Delay Queue Per Veh	Delay Stop Per Veh	Volume
Total 8 intersections	Detour Direction	Scenario 1	52.33	51.24	48.29	39.23	30.17	8640.9
		Scenario 0	69.99	68.88	67.93	60.58	45.13	8640
		Difference	-17.66	-17.64	-19.64	-21.35	-14.96	0.9
		Difference (%)	-25.2%	-25.6%	-28.9%	-35.2%	-33.1%	0.0%
	Both Direction	Scenario 1	59.85	59.12	56.19	47.12	37.13	12782.64
		Scenario 0	68.73	65.32	68.13	59.96	44.12	12797.83
		Difference	-8.88	-6.2	-11.94	-12.84	-6.99	-15.19
		Difference (%)	-12.9%	-9.5%	-17.5%	-21.4%	-15.8%	-0.1%
Total 8 intersections	Detour Direction	Scenario 2	53.47	52.81	49.11	40.88	31.69	8732.05
		Scenario 1	52.33	51.24	48.29	39.23	30.17	8640.9
		Difference	1.14	1.57	0.82	1.65	1.52	91.15
		Difference (%)	2.2%	3.1%	1.7%	4.2%	5.0%	1.1%
	Both Direction	Scenario 2	60.15	60.12	57.14	48.19	38.12	12791.23
		Scenario 1	59.85	59.12	56.19	47.12	37.13	12782.64
		Difference	0.3	1	0.95	1.07	0.99	8.59
		Difference (%)	0.5%	1.7%	1.7%	2.3%	2.7%	0.1%

From Table 34, the proactive real time offset tuning algorithm has significant improvements with existing network. And the proposed algorithm has similar performance as TRANSYT-7F.

One important performance measure of traffic simulation software is commuting time consumed. In this aspect, ETFOMM has excellent computing efficiency. Table 35 shows time simulation time for CORSIM and ETFOMM.

Table 35 Simulation time of CORSIM and ETFOMM

		Average CPU Time	Average Simulation Time
CORSIM	S0 and S1		About 480-500
	S2		About 500-520
ETFOMM	S0 and S1	165.72	89.85
	S2	385.56	195.9

The average simulation time of ETFOMM is much shorter than that of CORSIM. Thus, users could save plenty of time on simulation when they use ETFOMM. Another prominent benefit of computing efficiency is that ETFOMM is capable for implementing as a real time decision support system which could provide significant advantages than traditional offline decision support system.

7.4 Summary

The ETFOMM software is the newest traffic simulation software with updated CORSIM proven algorithms and theories. Furthermore, it has several advantages such platform independent, scalability, compatibility, and integration. It could be interfaced with most programming language and communicate on the Internet. Moreover, the basic simulation algorithm in ETFOMM is based on trust and proven traffic simulation software CORSIM with the-state-of-the-art computing technologies. The computing time and operating efficiency of ETFOMM could surpass CORSIM and other software. The advanced computing technologies such as native 64 bit architecture and distributing computing used in ETFOMM makes it is capable of an online decision support system. It is also utilized the popular. More important, ETFOMM provide convenient APIs for traffic research such as ICM rated studies. It has many built-in functions such as related detectors and traffic signals. Currently, ETFOMM has the ability to integrate with the proposed algorithm successfully. However, ETFOMM is being developed by New Global System for Intelligent Transportation Management (NGSIM). The more functions/tools would be developed and released by NGSIM. ETFOMM software could provide more components on API which could simulate proposed algorithm efficiently and reasonable. In the future, ETFOMM would be an exceptional choice for improving and expand this algorithm.

8. CONCLUSIONS AND RECOMMENDATIONS

Integrated Corridor Management (ICM) is one of the recent U.S. DOT research initiatives. ICM utilizes the capacity of the entire corridor opposite to the traditional approach that use capacities of different transportation modes separately. ICM is a promising technology for mitigating recurrent and non-current congestions. The infrastructure of ITS in Mississippi has been steadily improved. That provides a foundation for ICM implementation. Jackson, MS is the largest city in Mississippi. I-55 and State St are the major freeway and arterial in North/South directions for commuting traffic and other traffic in Jackson. The corridor is selected as the study site for ICM strategies implementation.

The research team proposes a system architecture of ICM system. There are two major modules, the optimization approach and the expert system. The optimization approach is used as the online method for optimally managing sudden surge of detour traffic from freeways to arterials. The expert system is utilized as a decision support tool for DOTs and other transportation agencies. It is also served as a backup system to handle the situation that the optimization module cannot work due to critical equipment is malfunction and/or other reasons. In this project, CORSIM is chosen as the simulator to develop, debug, test and evaluate the proposed models. Critical traffic data collection and analysis of the study corridor for simulation model construction and calibration are introduced.

Traffic signal coordination is an effective and traditional approach to alleviate congestions and reduce delays. In this project, an ICM optimization model and a proactive real-time offset tuning algorithm applying traffic signal coordination are proposed. The ICM optimization model minimizes diversion corridor delay: freeway delay, diversion route intersections' control delay, and detour extra travel time delay. The effectiveness of the proposed ICM optimization model is verified in a case study. The case study indicated, for corridor wide delay, diversion traffic from freeway to its parallel arterial could decrease 7.5% delay. By implementing the ICM optimization model, it could reduce 20% of network wide delay by applying diversion strategy and signal optimization for detour arterial simultaneously.

In addition, a proactive real time offset tuning algorithm also developed by the research team for ICM strategies. This algorithm only fine tuning offset which minimize institutional obstacles and maximize support from locals. To evaluate the effectiveness of the proposed algorithm, the traffic signal timing plans optimized by TRANSYT-7F are used as the benchmark. According to the results, the proposed algorithm exceeds performances of TRANSYT-7F when the diversion rates variations are not known in advance. When the diversion rates are predetermined and average additional detour traffic volume are taking into consideration for traffic signal optimization by TRANSYT-7F, the proposed algorithm reduces the number of stops significantly and the average control delays has comparable performances with TRANSYT-7F. Therefore, the proactive real time offset tuning algorithm is proposed for MDOT to utilize in real time as well as offline. The benefits and costs of implementing the proposed algorithm are presented. The 10 year's net benefit is nearly \$2.5 million and the benefits to costs ratio of 10 year is 1.6. It indicates that the proposed algorithm is worth for implementation.

Finally, the upcoming ICM simulation test bed based on US DOT's new microscopic simulator, ETFOMM, is introduced and assessed. There are two major benefits of this ICM simulation test bed. First, it has built-in ICM functions which are beneficial for ICM studies, such as diversion function. The proactive real time offset tuning algorithm is expected to become a built-in ICM function very soon. Second, ETFOMM integrated advanced computing technologies and applies

popular 64 bit architecture. These two features make computation time of ETFOMM significantly decrease which let it has the capacities to become an online decision support system.

For future research, the proactive real time offset tuning algorithm will continuously improve to get better performance. The actual field implementation of the proposed algorithm will be conducted when funding is available. Proposals for future research will be discussed and sent back to MDOT separately.

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APPENDIX

- **Intersection AM/PM Flow Rate**

1. Pascagoula St & State St

This intersection has no WB traffic since Pascagoula Street is a one-way street from east to west. No left turn in NB direction and no right turn in SB direction. Table 36 described the devices used for collecting data in each bound.

Table 36 Traffic Data Source

Pascagoula St & State St			
Approach	EB	NB	SB
Device	Manual Count	Manual Count	Radar

Table 37 and Table 38 provided AM and PM peak hour traffic flow rate. The AM peak 15-minute volume was in the interval 8:27 am - 8:42 am, which was used to calculate AM peak hour flow rate. PM peak 15-minute traffic volume in 5:00 pm - 5:15 pm was used to calculate PM peak hour flow rate.

NB and EB AM peak hour flow rate was estimated by using the “downtown factor” because it is in the downtown area. The AM peak hour flow rate of an inbound approach was obtained by PM peak hour flow rates of the corresponding approach multiplying the inbound factor. The AM peak hour flow rates of an outbound approach was calculated by PM peak hour flow rates in the corresponding approach multiplying the outbound factor. The calculation procedures at Pascagoula St & State St were showed below.

(Inbound) NB-T: $464 \times 2.935 = 1362$

(Outbound) EB-T: $996 \times 0.5335 = 531$

Table 37 AM Peak Hour Volume of Pascagoula St & State St

Pascagoula St (AM)	NB			SB (Out)			EB		
	L (None)	T (In)	R (Out)	L	T	R (None)	L (In)	T (Out)	R (Out)
Turing Movement (In or Out)									
# of Lanes		1	1 TR	2	2		1 TL	1	1 TR
Critical Vol. (VPH)		1362	92	184	268		411	531	92

Table 38 PM Peak Hour Volume of Pascagoula St & State St

Pascagoula St (PM)	NB			SB (Out)			EB		
	L (None)	T (In)	R (Out)	L	T	R (None)	L (In)	T (Out)	R (Out)
# of Lanes		1	1 TR	2	2		1 TL	1	1 TR
Critical Vol. (VPH)		464	172	508	380		140	996	172

Based on the above two tables, Figure 22 and Figure 23 were generated to show the traffic volume for each approach. From the figures, it is easy to identify the inbound and outbound traffic in AM and PM peak periods. NB on State St is an inbound approach and have more traffic in AM than PM peak period. SB on State St and EB on Pascagoula St are outbound which have opposite traffic patterns to NB.

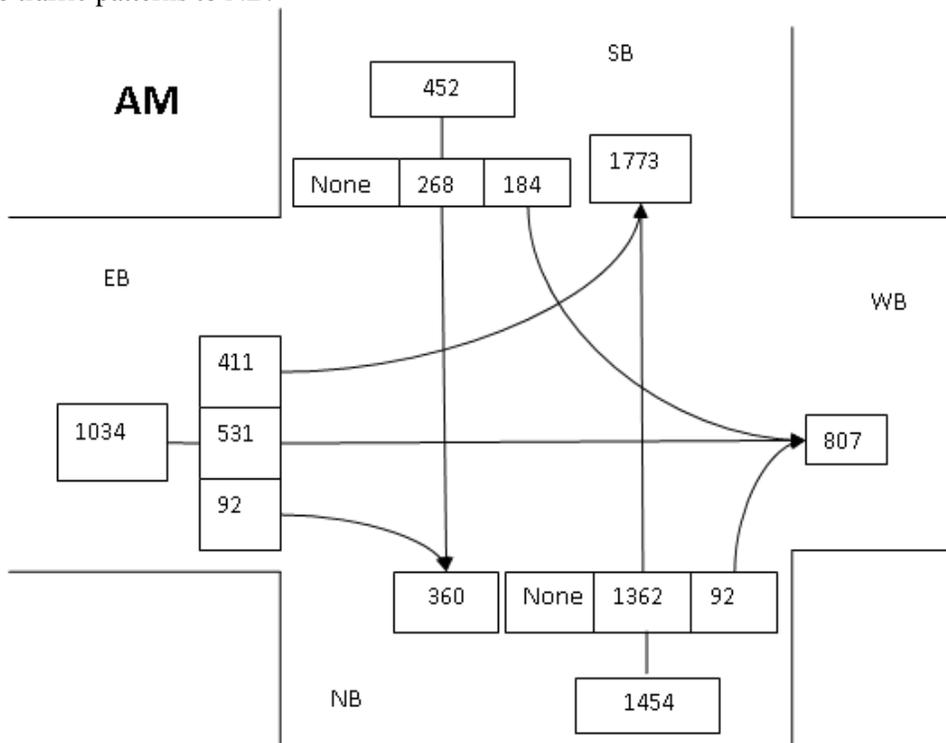


Figure 22 AM Peak Turning Movement Volume of Pascagoula St & State St

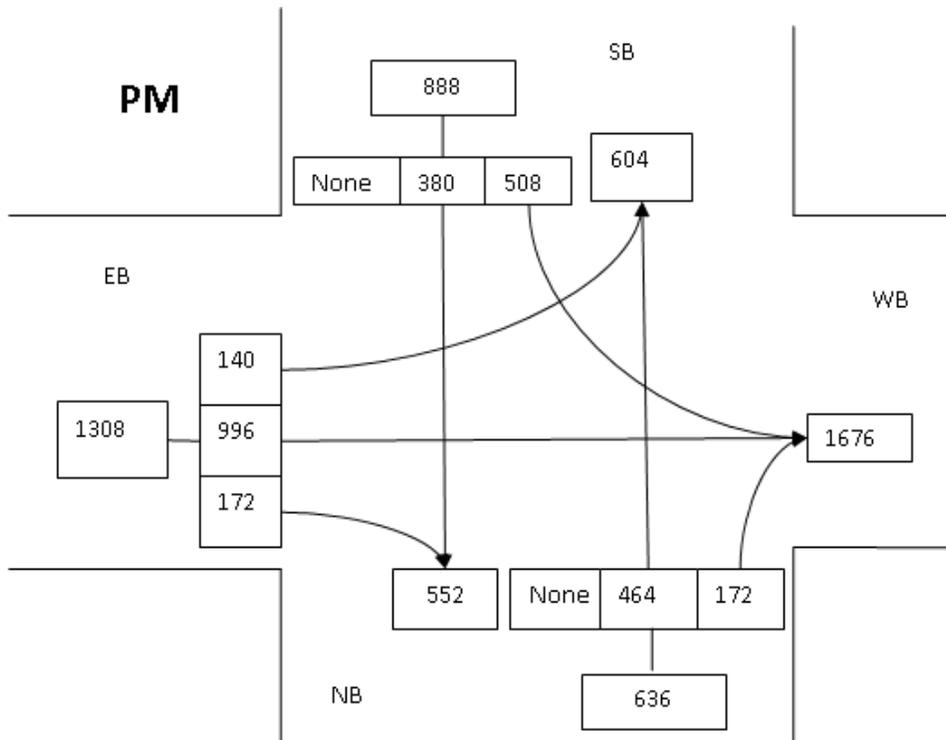


Figure 23 PM Peak Turning Movement Volume of Pascagoula St & State St

2. Pearl St & State St

Pearl St is a one-way street from west to east. It has no EB traffic. There are no left turn from the SB and no right turn from the NB. The traffic data source, AM/PM peak hour flow rates and volume of turning movements in AM/PM peak period were provided below.

Table 39 Traffic Data Source

Pearl St & State St			
Approach	WB	NB	SB
Device	Manual Count	Radar	Manual Count

The AM peak hour flow rate was calculated by the peak 15-minute traffic volume from 8:10 am to 8:25 am. Traffic volume in 5:03 pm - 5:18 pm was the peak PM 15-minute traffic volume, which was chose to calculate PM peak hour flow rate.

“Downtown factor” was used to estimate the unavailable PM peak hour flow rates at this intersection. The unavailable PM flow rates of inbound movements were estimated by corresponding AM peak hour flow rates of the movement divided by the inbound factor. The unavailable PM flow rates of outbound movements were computed by corresponding AM peak hour flow rates of the movement divided by the outbound factor. The procedures to estimate the inbound and outbound PM peak hour flow rates were shown below.

(Inbound) SB-R: $204 / 2.935 = 70$
 (Outbound) SB-T: $328 / 0.5335 = 615$

Table 40 AM Peak Hour Volume of Pearl St & State St

Pearl St (AM)	N (In)			S			W		
	L	T	R (None)	L (None)	T (Out)	R (In)	L (Out)	T (In)	R (Out)
# of Lanes	2	2			2	1	1 TL	2	1
Critical Vol. (VPH)	152	344			328	204	108	1040	432

Table 41 PM Peak Hour Volume of Pearl St & State St

Pearl St (PM)	N (In)			S			W		
	L	T	R (None)	L (None)	T (Out)	R (In)	L (Out)	T (In)	R (Out)
# of Lanes	2	2			2	1	1 TL	2	1
Critical Vol. (VPH)	436	536			615	70	202	354	147

Figure 24 and Figure 25 are described the traffic volume entering or leaving in each approach of the intersection. From the two figures, it can be seen that NB and SB are outbound as a whole and WB is inbound.

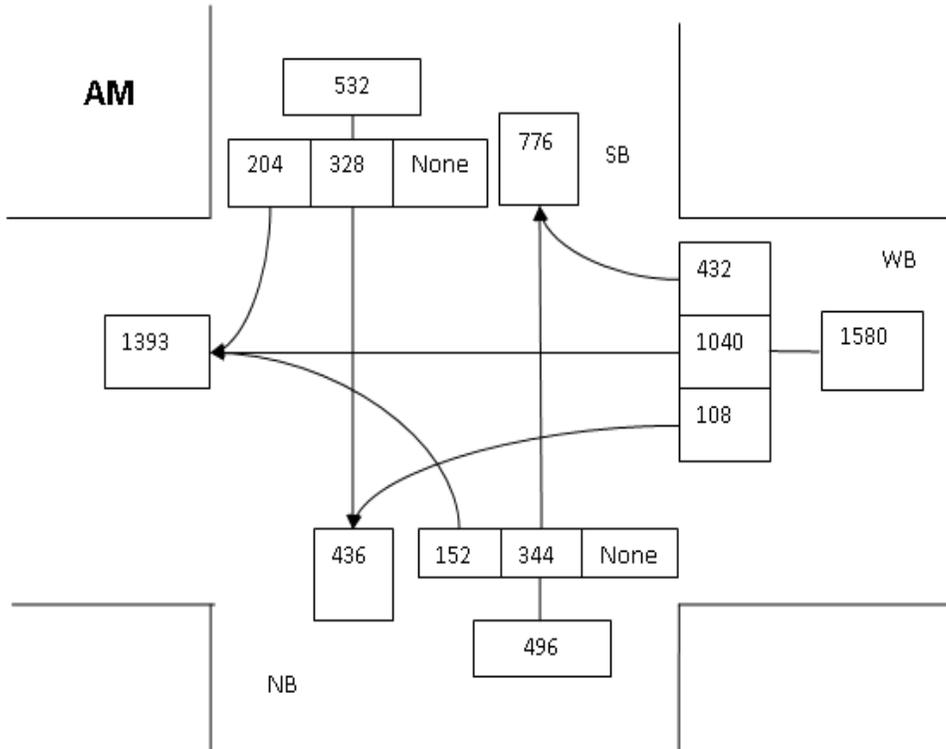


Figure 24 AM Peak Turning Movement Volume of Pearl St & State St

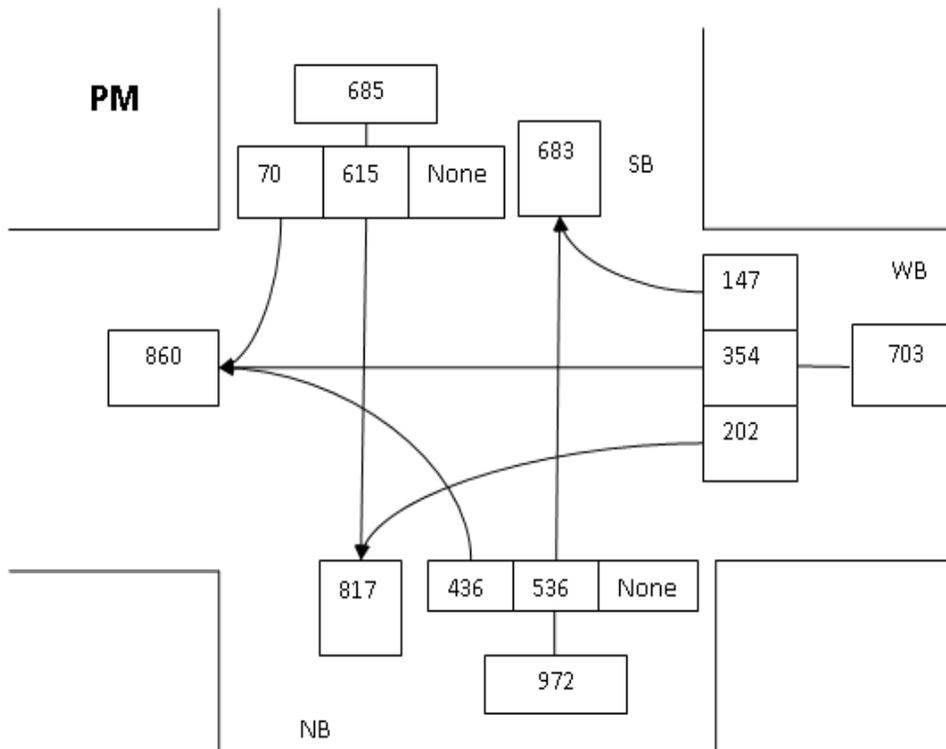


Figure 25 PM Peak Turning Movement Volume of Pearl St & State St

3. Capitol St & State St

Capitol St is a one-way street from west to east. It has no WB traffic. There is only through movement on State St and no through movement in EB. The traffic data source, AM/PM peak hour flow rate and data of turning movements during AM/PM peak periods were provided below.

Table 42 Traffic Data Source

Capitol St & State St			
Approach	EB	NB	SB
Device	Manual Count	Radar	Manual Count

AM and PM peak hour flow rates were provided in Table 43 and Table 44. Traffic data in the interval 7: 47 am - 8:02 am was the peak 15-minute volume and used to calculate the AM peak hour flow rate. PM peak hour flow rate was computed based on the traffic data during 4:30 pm - 4:45 pm. AM peak hour traffic volume was estimated by the PM peak hour flow rate multiplying the downtown factor. An example of the calculation procedures was provided below.

(In) EB-L: 332 X 2.935 = 974

(Out) SB-T: 1076 X 0.5335 = 574

Table 43 AM Peak Hour Volume of Capitol St & State St

Capitol St (AM)	N (In)			S (Out)			E (3 Lanes)		
	L (None)	T	R (None)	L (None)	T	R (none)	L (In)	T (none)	R (Out)
# of Lanes		2			2		1 and 1LR		1 and 1LR
Critical Vol. (VPH)		1268			574		974		196

Table 44 PM Peak Hour Volume of Capitol St & State St

Capitol St (AM)	N (In)			S (Out)			E (3 Lanes)		
	L (None)	T	R (None)	L (None)	T	R (none)	L (In)	T (none)	R (Out)
# of Lanes		2			2		1 and 1LR		1 and 1LR
Critical Vol. (VPH)		432			1076		332		368

Figure 26 and Figure 27 showed the turning movements data in AM and PM peak period. There were more traffic through State St which went to the north in the morning, while more traffic headed to the south in the evening.

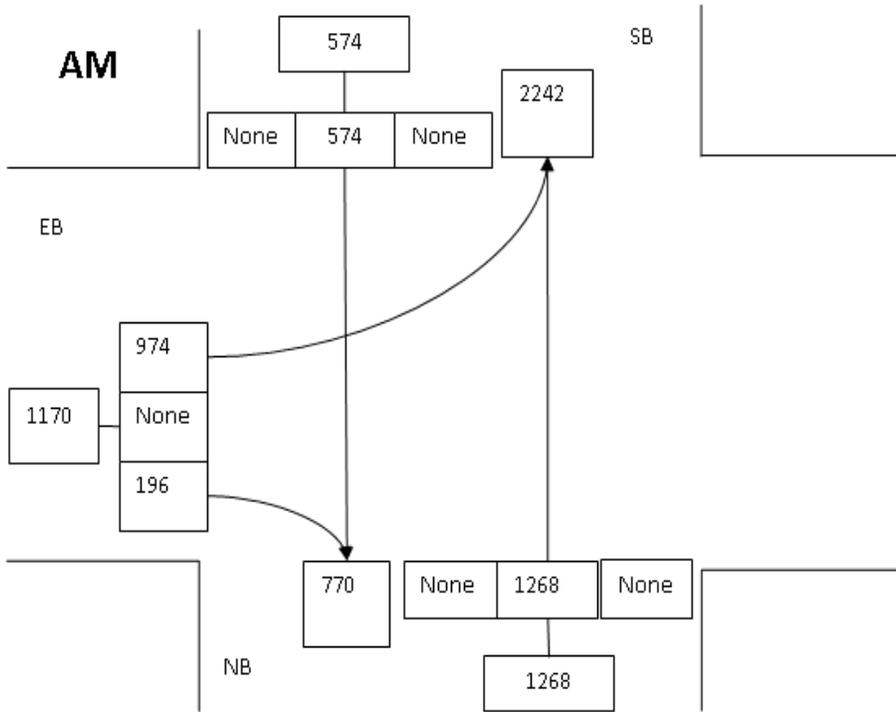


Figure 26 AM Peak Turning Movement Volume of Capitol St & State St

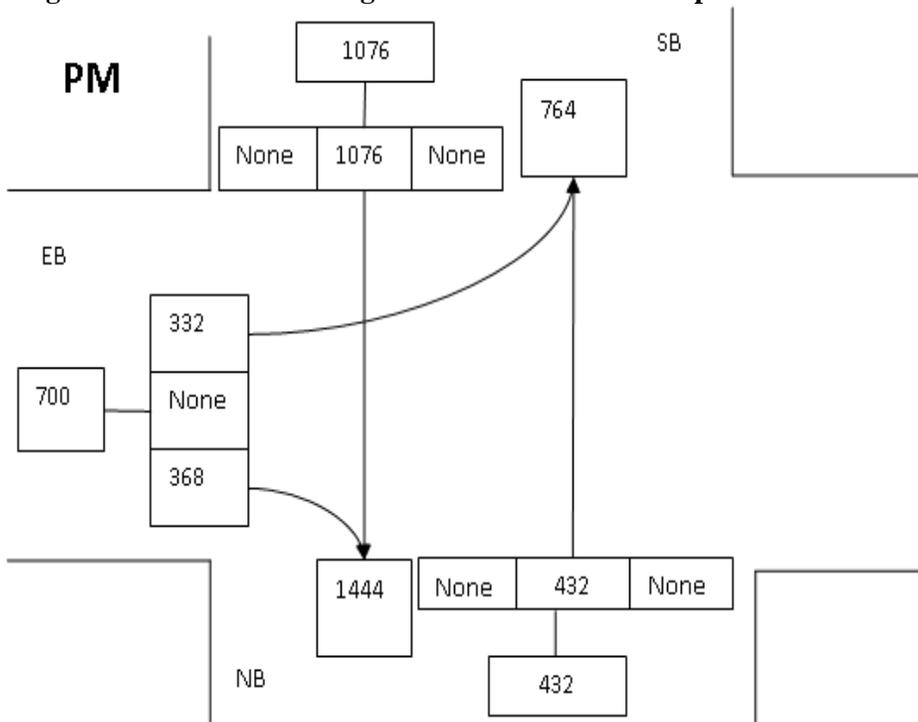


Figure 27 PM Peak Turning Movement Volume of Capitol St & State St

4. Amite St & State St

Amite St west of State St is a one-way street from east to west. It has no EB traffic. However, Amite St east of State St is a two-way street. SB and NB approaches of the intersection have through, left and right turn traffic.

Table 45 Traffic Data Source

Amite St & State St			
Approach	WB	NB	SB
Device	Manual Count	Manual Count	Manual Count

AM and PM peak hour flow rates were provided in Table 46 and Table 47. Traffic data from 7:40 am to 7:55 am was the peak 15-minute volume and used to calculate the AM peak hour flow rate. PM peak hour flow rate was estimated by AM peak hour flow rate divided by the downtown factor.

The example below showed the calculations using down factor. Inbound AM volumes were divided by inbound downtown factor and outbound AM volumes were divided by outbound downtown factor.

(Inbound) NB-T: $812 / 2.935 = 277$

(Outbound) SB-T: $360 / 0.5335 = 675$

Table 46 AM Peak Hour Volume of Amite St & State St

Amite St (AM)	N (In)			S			W		
	L	T	R	L (In)	T (Out)	R (In)	L (Out)	T (In)	R (In)
Turning Movement (In or Out)									
# of Lanes	1	1	1 TR	1	1	1 TR	1 TL	1	1
Critical Vol. (VPH)	440	812	68	16	360	124	40	164	68

Table 47 PM Peak Hour Volume of Amite St & State St

Amite St (PM)	N (In)			S			W		
	L	T	R	L (In)	T (Out)	R (In)	L (Out)	T (In)	R (In)
Turning Movement (In or Out)									
# of Lanes	1	1	1 TR	1	1	1 TR	1 TL	1	1

Critical Vol. (VPH)	150	277	23	6	675	42	75	56	23
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Figure 28 and Figure 29 showed the turning movements data in AM and PM peak period. There were more traffic through State St which went to the north in the morning, while more traffic headed to the south in the evening.

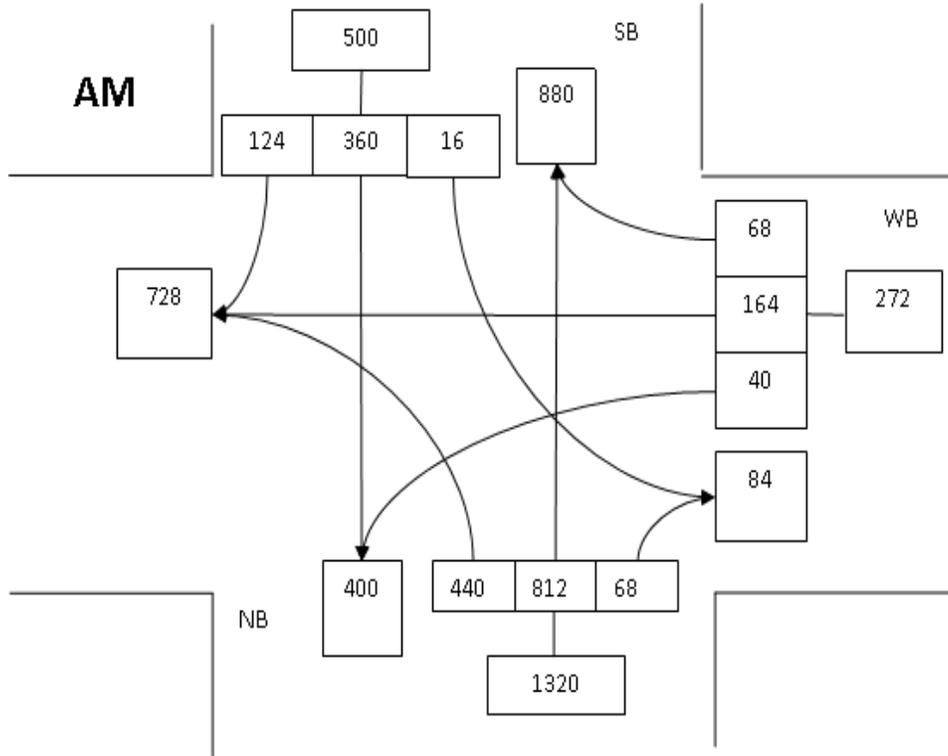


Figure 28 AM Peak Turning Movement Volume of Amite St & State St

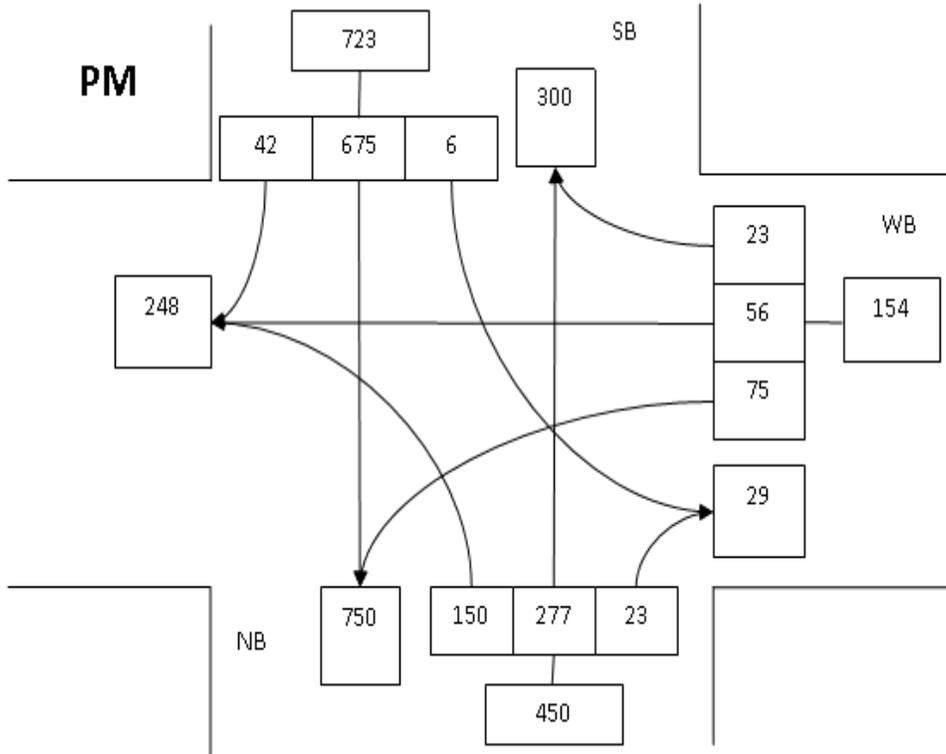


Figure 29 PM Peak Turning Movement Volume of Amite St & State St

5. High St & State St

High Street is a two-way street in the downtown area of Jackson. Its traffic data source, AM/PM peak hour flow rate and AM/PM peak hour turning movement data were provided below.

Table 48 Traffic Data Source

High St & State St				
Approach	NB	SB	EB	WB
Device	Manual Count	Manual Count	Manual Count	Manual Count

AM and PM peak hour flow rates are listed in Table 49 and Table 50. PM peak hour flow rate was collected by manual count. The peak fifteen minute volume occurred in the interval of 5:10 pm - 5:25 pm. AM peak hour flow rate was estimated by PM peak hour flow rate multiplying the downtown factor. For example,

(Inbound) NB-T: $632 \times 2.935 = 1855$
 (Outbound) SB-L: $156 \times 0.5335 = 83$

Table 49 AM Peak Hour Flow Rate of High St & State St

High St (AM)	N			S			W (In)			E		
	L (In)	T (In)	R (Out)	L (Out)	T (In)	R (In)	L	T	R	L (In)	T (Out)	R (In)
Turning Movement												

# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	188	1855	115	83	2043	211	481	822	434	693	235	223

Table 50 PM Peak Hour Flow Rate of High St & State St

High St (PM)	N			S			W (In)			E		
Turning Movement	L (In)	T (In)	R (Out)	L (Out)	T (In)	R (In)	L	T	R	L (In)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	64	632	216	156	696	72	164	280	148	236	440	76

The volumes of turning movement in AM/PM peaks were displayed respectively in Figure 30 and Figure 31. All four approaches are inbound because all of them carry more traffic in the morning than the evening.

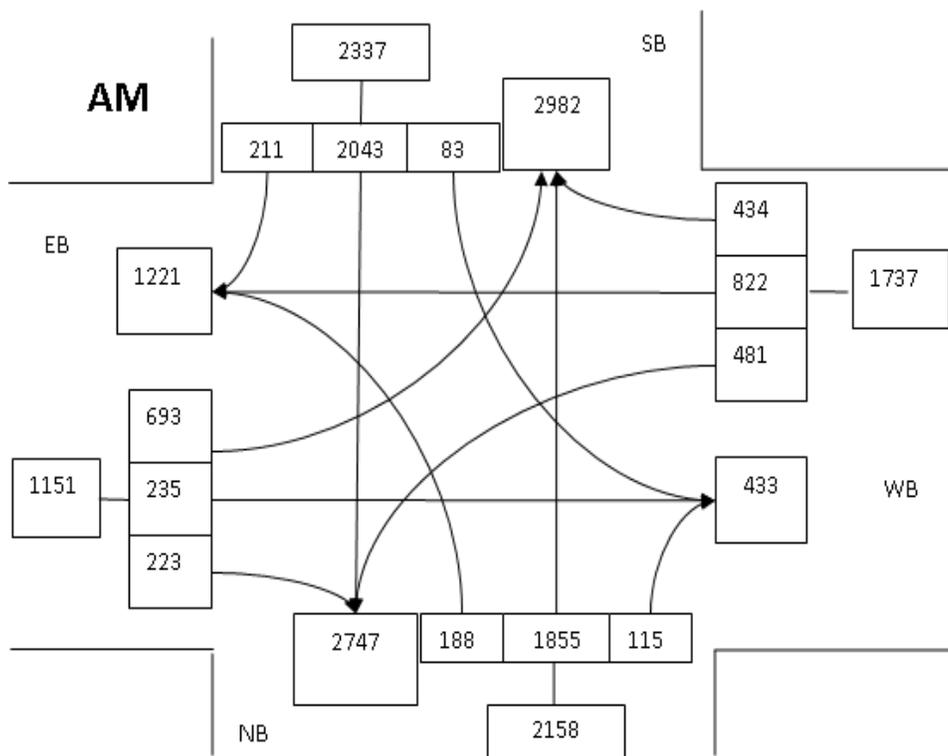


Figure 30 AM Peak Turning Movement Volume of High St & State St

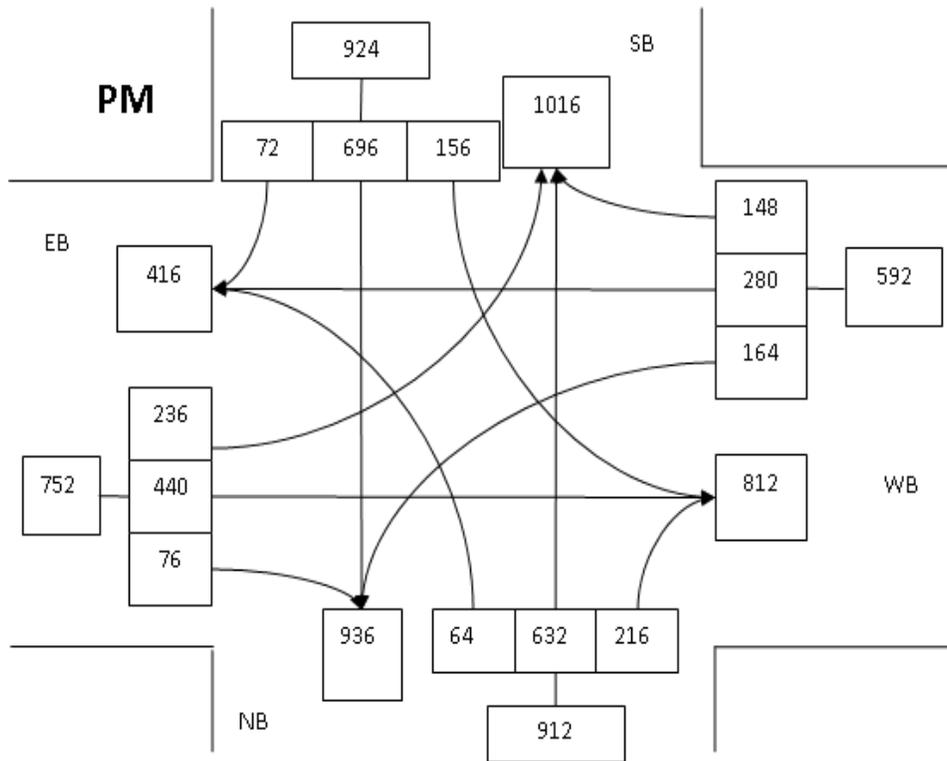


Figure 31 PM Peak Turning Movement Volume of High St & State St

6. Fortification St & State St

Fortification St is a two-way street. It is a major street that connects downtown area and I-55. The data about traffic data source, AM/PM peak hour flow rates and AM/PM peak turning movements were provided below.

Table 51 Traffic Data Source

Fortification St & State St				
Approach	NB	SB	EB	WB
Device	Manual Count	Manual Count	Manual Count	Manual Count

The volume from 3:40 pm to 3:55 pm was the peak 15-minute volume. The PM Peak hour flow rate was calculated based on the peak 15-minute volume. The AM peak hour flow rate was estimated by PM peak hour flow rate multiplied by the downtown factor. AM/PM peak hour flow rates were presented in Table 52 and Table 53.

The calculations below showed how to estimate inbound and outbound approach's AM peak hour flow rate using downtown factor. Inbound approach AM peak hour flow rate was calculated by the PM peak hour flow rate multiplying the inbound downtown factor and outbound approach AM peak hour flow rate was calculated by the PM peak hour flow rate multiplying the outbound downtown factor.

(Inbound) NB-L: $168 \times 2.935 = 493$
 (Outbound) NB-R: $168 \times 0.5335 = 90$

Table 52 AM Peak Hour Volume of Fortification St & State St

Fortification St (AM)	N			S			W (In)			E		
	L (In)	T (In)	R (Out)	L (Out)	T (In)	R (In)	L	T	R	L (In)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	493	1972	90	109	2336	364	387	564	423	728	344	282

Table 53 PM Peak Hour Volume of Fortification St & State St

Fortification St (PM)	N			S			W (In)			E		
	L (In)	T (In)	R (Out)	L (Out)	T (In)	R (In)	L	T	R	L (In)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	168	672	200	204	796	124	132	192	144	248	644	96

Figure 32 and Figure 33 showed the volumes of turning movements in each approach. For the EB approach, there was more traffic turning left and right onto State St in the morning; while, it had more through traffic leaving the downtown area in the evening. All four approaches are inbound because all of them carry more traffic in the morning than in the evening.

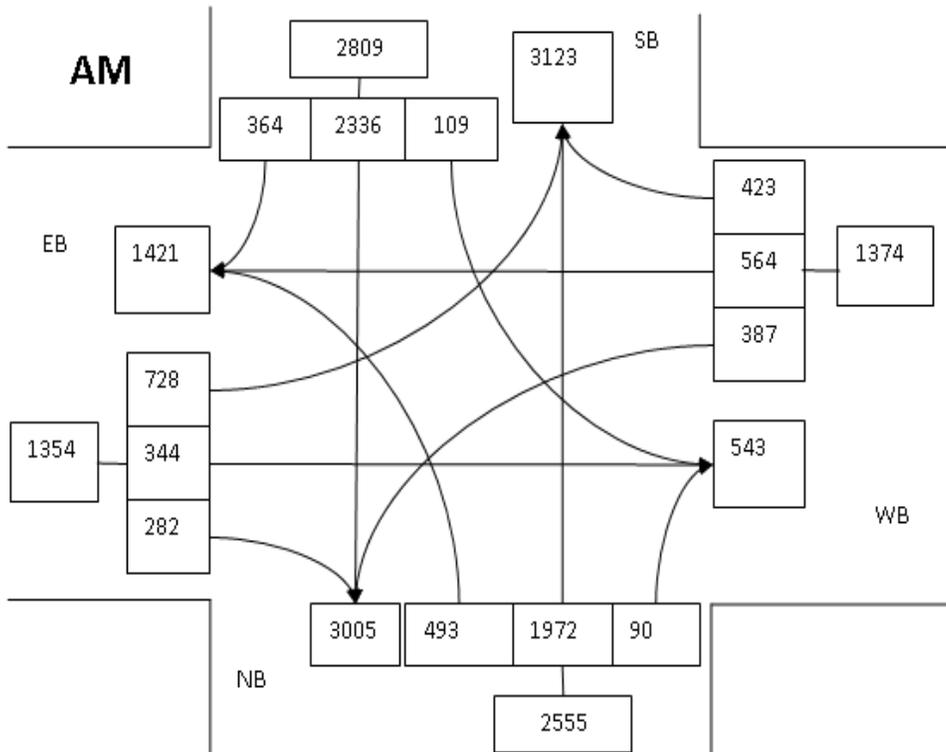


Figure 32 AM Peak Turning Movement Volume of Fortification St & State St

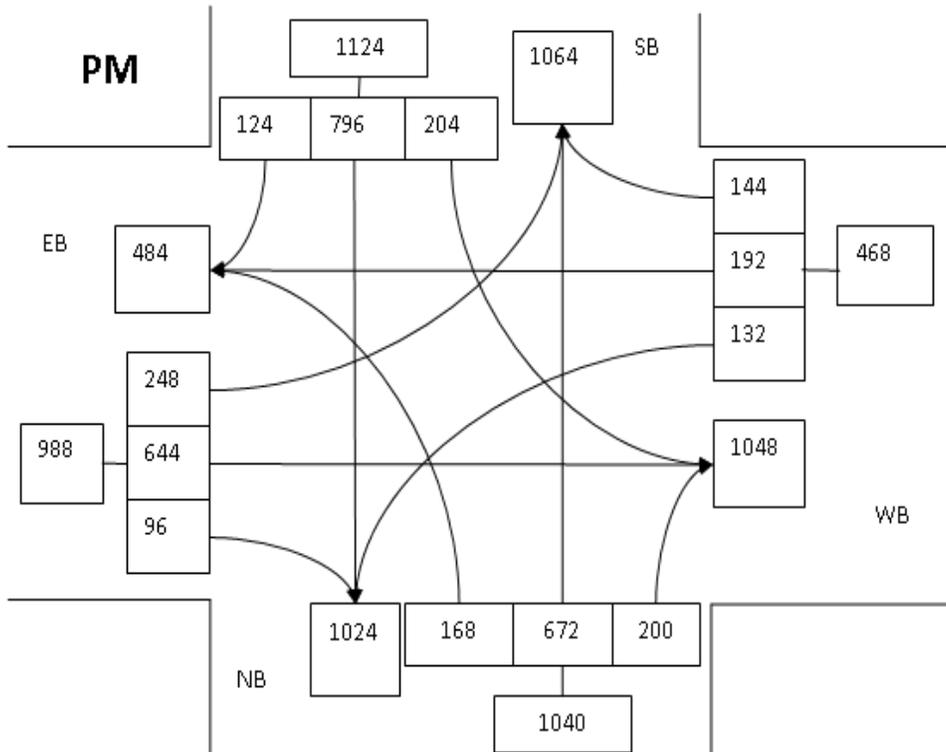


Figure 33 PM Peak Turning Movement Volume of Fortification St & State St

7. Woodrow Wilson Ave & State St

Woodrow Wilson Ave, which is similar to Fortification St, is a major arterial connecting the downtown area and I-55. It is also considered as a boundary of the downtown area. The south of Woodrow Wilson Ave is the downtown area. And, intersections north of Woodrow Wilson are located out of the downtown area. Woodrow Wilson Ave & State St intersection's traffic data source, AM/PM peak hour flow rates and volumes of turning movement during AM/PM peak periods were presented below.

Table 54 Traffic Data Source

Woodrow Wilson Ave & State St				
Approach	NB	SB	EB	WB
Device	Manual Count	Manual Count	Manual Count	Manual Count

PM peak 15-minute volume occurred between 4:45 pm and 5:00 pm, which was used to calculate the PM peak hour flow rate. The field data of AM peak period was not available. The AM peak hour flow rate was estimated by PM peak hour flow rate multiplying the downtown factor. The AM/PM peak hour volumes were provided in Table 55 and Table 56 respectively. There was no data for NB right turn.

(Inbound) NB-L: $144 \times 2.935 = 423$

(Outbound) NB-T: $644 \times 0.5335 = 344$

Table 55 AM Peak Hour Flow Rate of Woodrow Wilson Ave & State St

Woodrow Wilson Ave (AM)	N			S			W			E		
	L (In)	T (Out)	R (In)	L (Out)	T (In)	R (In)	L (In)	T (In)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	3	1	1	2	1	2	2	1	2	2	1
Critical Vol. (VPH)	423	344	N/A	169	2160	1197	1820	2371	154	164	467	564

Table 56 PM Peak Hour Flow Rate of Woodrow Wilson Ave & State St

Woodrow Wilson Ave (PM)	N			S			W			E		
	L (In)	T (Out)	R (In)	L (Out)	T (In)	R (In)	L (In)	T (In)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	3	1	1	2	1	2	2	1	2	2	1
Critical Vol. (VPH)	144	644	N/A	316	736	408	620	808	288	308	876	192

Figure 34 and Figure 35 showed the volumes of turning movements in each approach. SB and WB approaches are inbound. For the NB right turn, the volume was highlighted with red color because traffic volume of the NB right turn was not available.

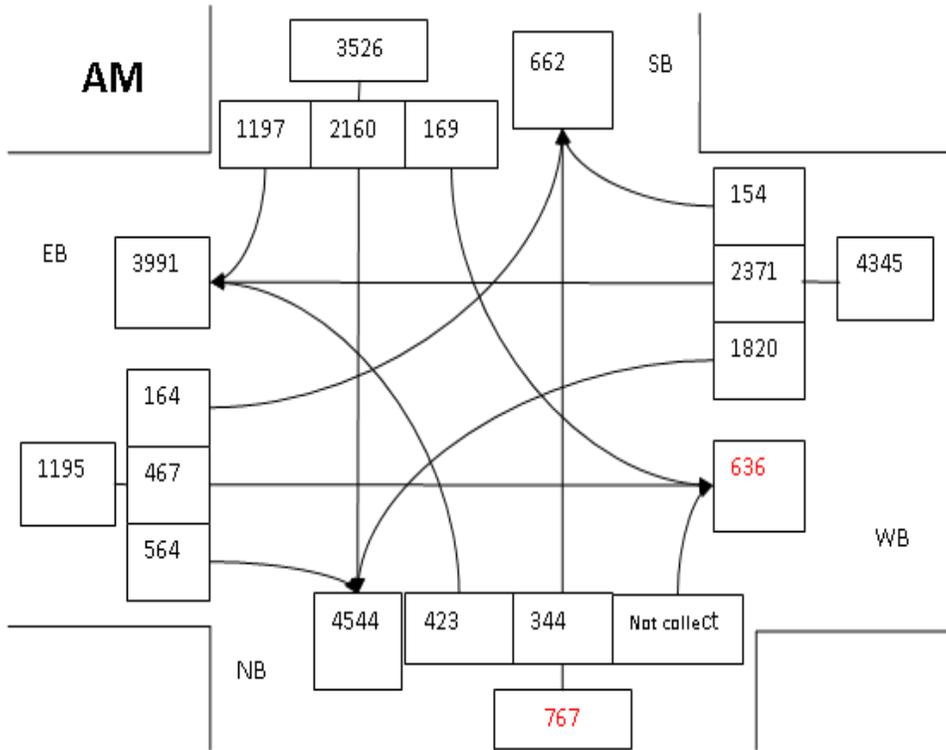


Figure 34 AM Peak Turning Movement Volume of Woodrow Wilson Ave & State St

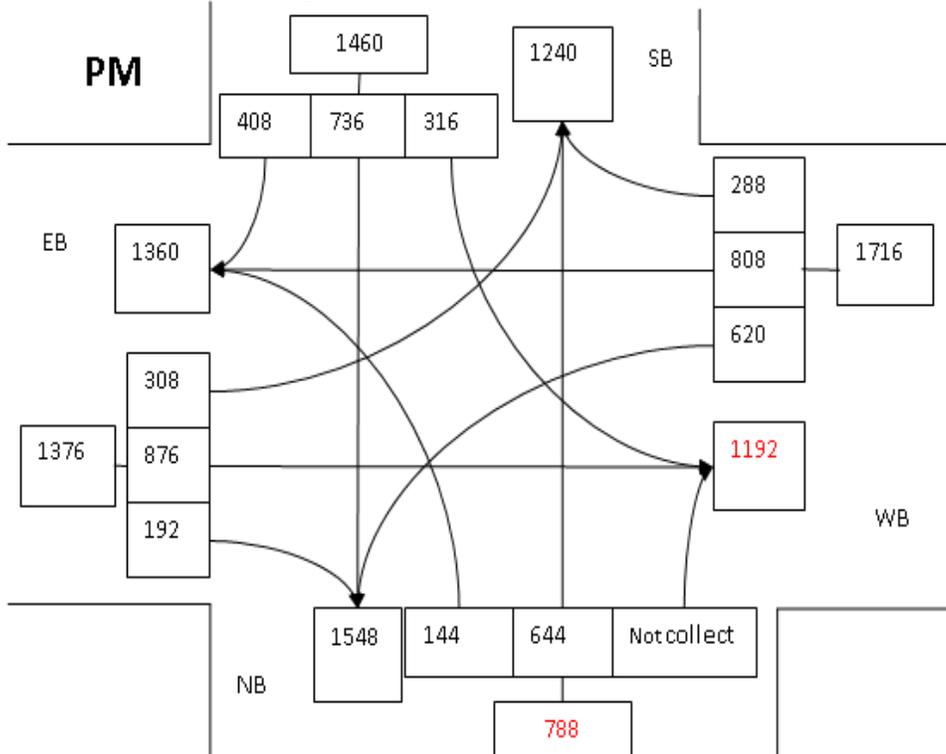


Figure 35 PM Peak Turning Movement Volume of Woodrow Wilson Ave & State St

8. Old Canton Rd & State St

Old Canton Rd & State St is a three-leg intersection. The traffic source data, AM/PM peak hour flow rates, and volumes of turning movements in AM/PM peak period were provided below.

Table 57 Traffic Data Source

Old Canton Rd & State St				
Approach	NB	SB	EB	WB
Lane	Through	Right	Through	Left
Device	Radar	NC 200	NC 200	Radar

Peak 15-minute volume in AM was during 7:39 am - 7:54 am, while 5:15 pm - 5:30 pm was the peak 15 minutes in the PM peak period. AM/PM peak hour flow rates were computed based on the peak 15-minute volume and the results were listed in Table 58 and Table 59.

Table 58 AM Peak Hour Volume of Old Canton Rd & State St

Old Canton Rd (AM)	NB (Out)		SB (In)	WB (In)
Turning Movement	T	R	T	L
# of lanes	2	2	1	2
Critical Vol. (VPH)	280	560	572	916

Table 59 PM Peak Hour Volume of Old Canton Rd & State St

Old Canton Rd (PM)	NB (Out)		SB (In)	WB (In)
Turning Movement	T	R	T	L
# of lanes	2	2	1	2
Critical Vol. (VPH)	676	1104	300	820

Figure 36 and Figure 37 showed the volumes of turning movements in each approach. SB and WB are inbound and NB is outbound.

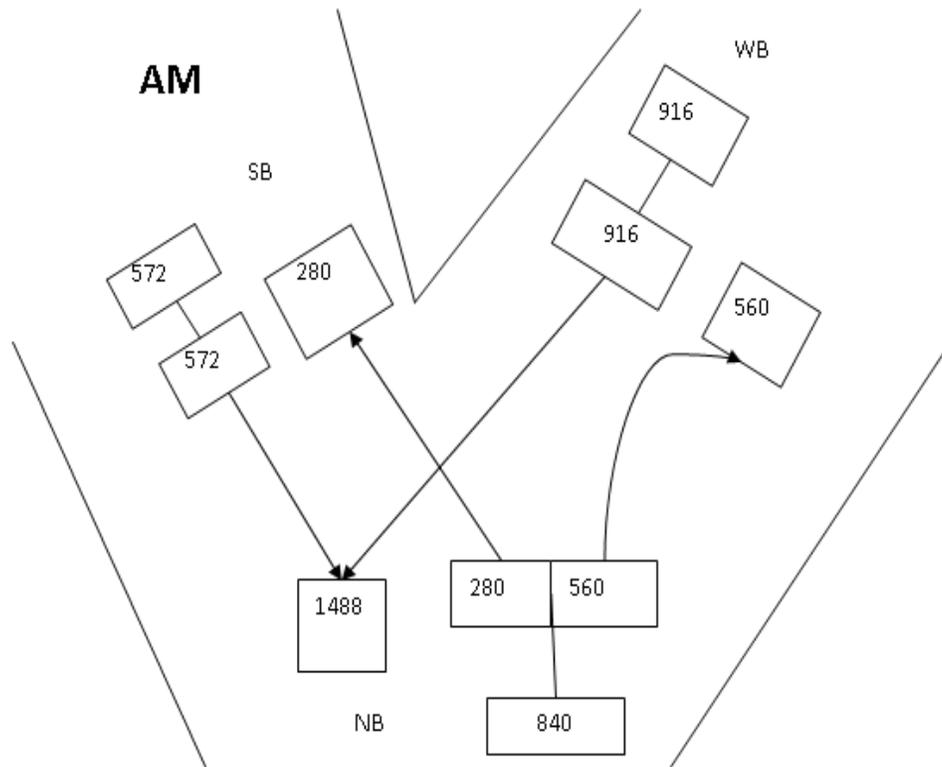


Figure 36 AM Peak Turning Movement Volume of Old Canton Rd & State St

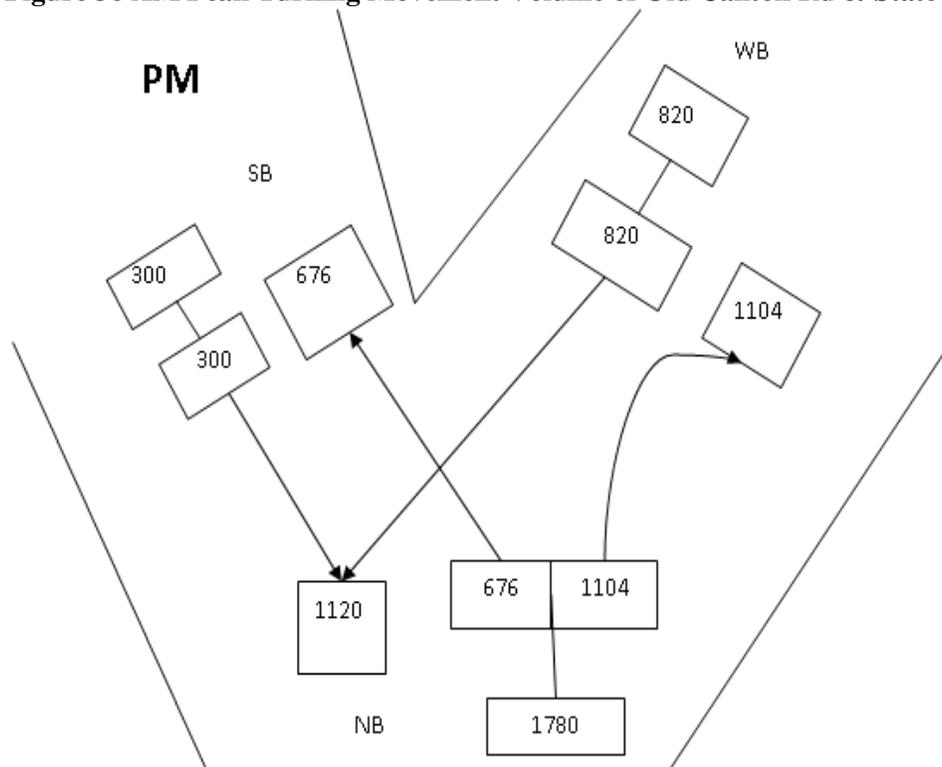


Figure 37 PM Peak Turning Movement Volume of Old Canton Rd & State St

9. Meadowbrook Rd & State St

Meadowbrook Rd is a two-way street. The traffic data source, AM/PM peak hour flow rates, and volume of turning movements in AM/PM peak period were presented below.

Table 60 Traffic Data Source

Meadowbrook Rd & State St				
Approach	NB	SB	EB	WB
Device	Manual Count	Manual Count	Manual Count	Manual Count

Manual count was used to collect traffic data in the AM peak period. The peak 15-minute volume occurred at 7:45 am - 8:00 am. The AM peak hour flow rate was computed based on the peak 15-minute volume. The PM peak hour flow rate was estimated using AM peak hour flow rate divided by briarwood factor. An example about using briarwood factor to estimate unavailable traffic data was shown below. For estimating outbound PM traffic, the corresponding AM traffic volume was divided by the outbound briarwood factor. Same procedure was applied to estimate inbound PM traffic using inbound briarwood factor.

(Inbound) SB-T: $444 / 1.34 = 331$

(Outbound) NB-L: $28 / 0.668 = 42$

The AM/PM peak hour flow rates were shown in Table 61 and Table 62.

Table 61 AM Peak Hour Volume of Meadowbrook St & State St

Meadowbrook St (AM)	N (Out)			S			W			E		
	L	T	R	L (Out)	T (In)	R (In)	L (In)	T (In)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	28	196	68	116	444	120	124	204	48	96	244	96

Table 62 PM Peak Hour Volume of Meadowbrook St & State St

Meadowbrook St (PM)	N (Out)			S			W			E		
	L	T	R	L (Out)	T (In)	R (In)	L (In)	T (In)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	42	293	102	174	331	90	93	152	72	144	365	72

The volumes of turning movements in AM/PM peaks were showed in Figure 38 and Figure 39. SB and WB are inbound, while NB and EB are outbound.

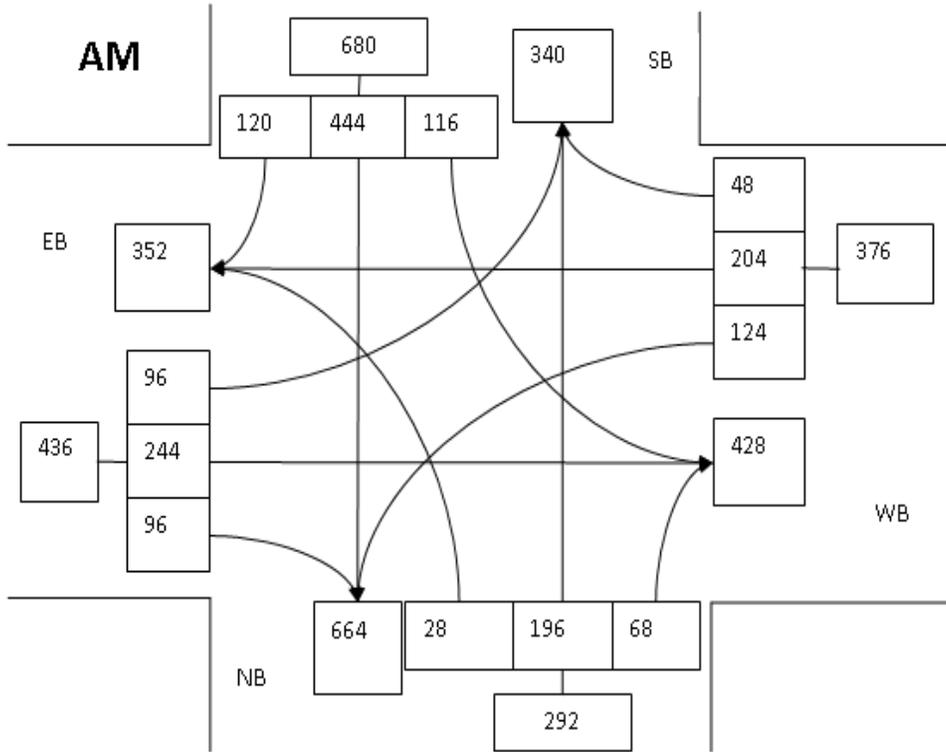


Figure 38 AM Peak Turning Movement Volume of Meadowbrook St & State St

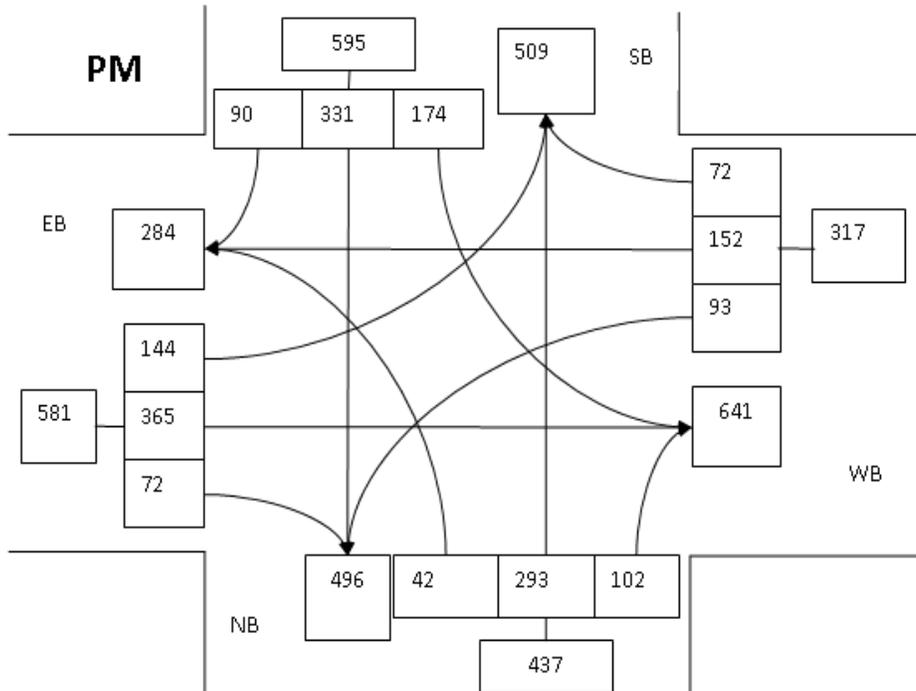


Figure 39 PM Peak Turning Movement Volume of Meadowbrook St & State St

10. Northside Dr & State St

Northside Drive is a two-way street. It is a major street connecting I-55 and State St. This intersection is a four-leg intersection. The traffic data source, AM/PM peak hour flow rates, and volumes of turning movements in AM/PM periods were presented below.

Table 63 Traffic Data Source

Northside Dr& State St				
Approach	NB	SB	EB	WB
Device	Manual Count	Manual Count	Manual Count	Manual Count

Manual count was used to collect traffic data during the AM peak period. 8:30 am - 8:45 am had the peak 15-minute traffic volume. The AM peak hour flow rate was computed based on the peak 15-minute volume. The PM peak hour flow rate was estimated by AM peak hour flow rate divided by the briarwood factor. The AM/PM peak hour flow rates were listed in Table 64 and Table 65.

(Inbound) SB-T: $404 / 1.34 = 301$
 (Outbound) NB-T: $132 / 0.668 = 198$

Table 64 AM Peak Hour Volume of Northside Drive & State St

Northside Dr (AM)	N (Out)			S			W			E		
	L	T	R	L (Out)	T (In)	R (Out)	L (In)	T (Out)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	16	132	48	32	404	348	140	292	8	168	408	16

Table 65 PM Peak Hour Volume of Northside Drive & State St

Northside Dr (PM)	N (Out)			S			W			E		
	L	T	R	L (Out)	T (In)	R (Out)	L (In)	T (Out)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1 TR	1	1	1 TR
Critical Vol. (VPH)	24	198	72	48	301	521	104	437	12	251	611	12

The AM/PM volumes of turning movements in each approach were shown in Figure 40 and Figure 41. All four approaches are outbound approach which had more traffic in the morning than the evening.

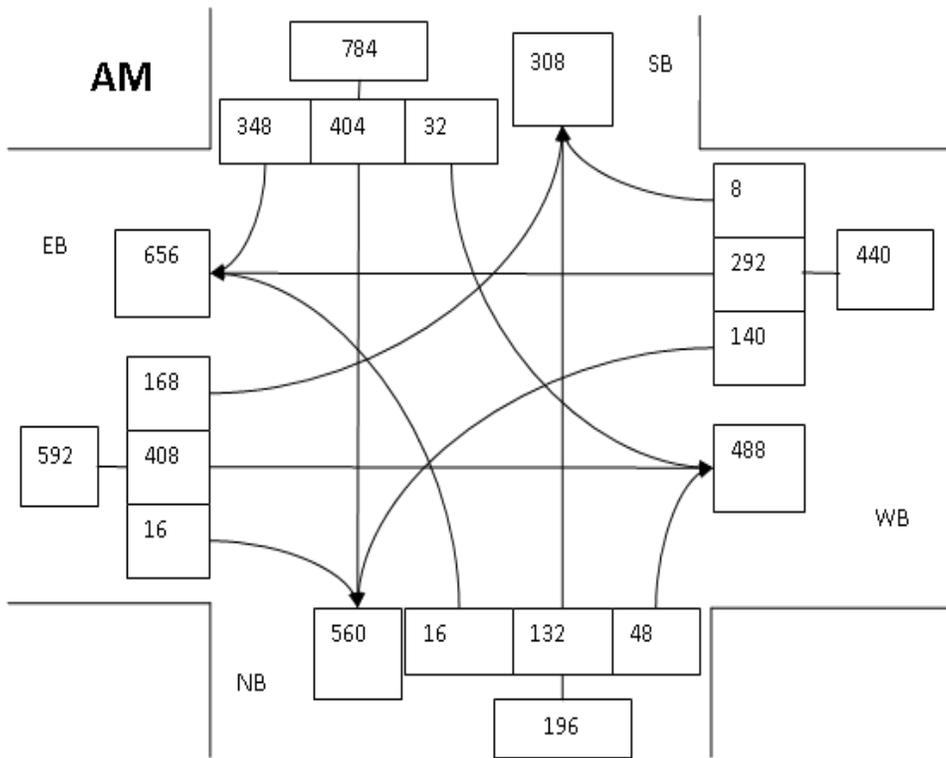


Figure 40 AM Peak Turning Movement Volume of Northside Dr & State St

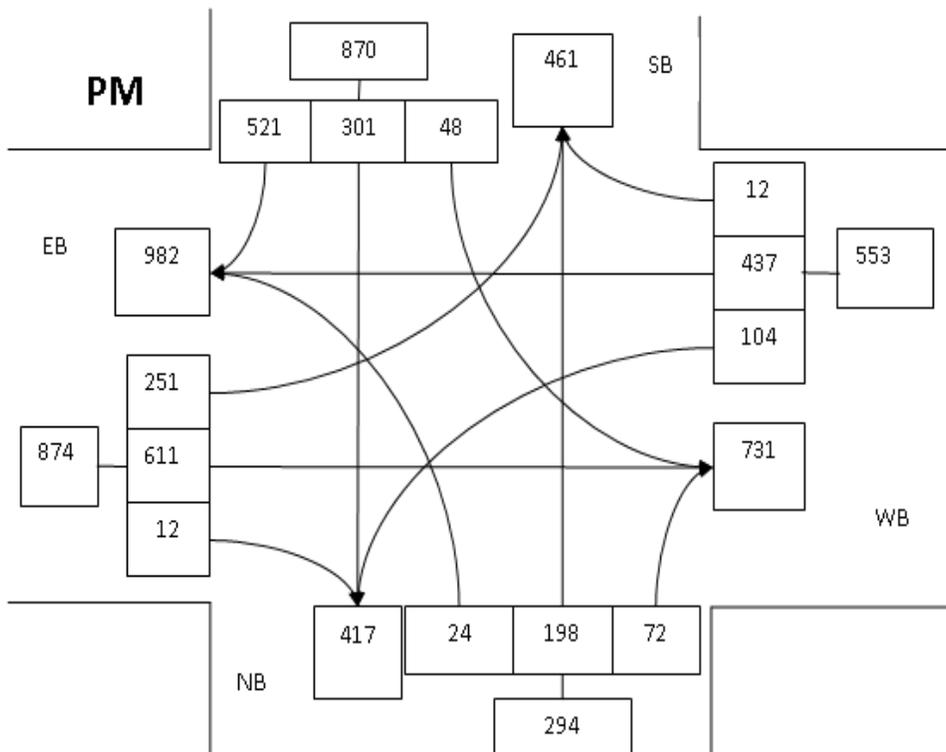


Figure 41 PM Peak Turning Movement Volume of Northside Dr & State St

11. Briarwood Dr & State St

Briarwood Dr & State St is a three-leg intersection. The data including traffic data source, AM/PM Peak hour flow rates and volumes of turning movements in AM/PM peak period were shown below.

Table 66 Traffic Data Source

Briarwood Dr& State St				
Approach	WB		NB	SB
Turning Movement	Left	Right		
Device	Manual Count	NC 200	Radar	NC 200

The AM traffic data of WB left turn was collected for 20 minutes by manual count. These data were used to estimate the AM peak hour volume of WB left turn. The PM peak hour flow rate of WB left turn was estimated by AM peak hour flow rate divided by the briarwood inbound factor. More specifically, the briarwood outbound factor was calculated by AM SB through traffic flow divided by PM SB through traffic flow. The briarwood inbound factor was calculated by AM NB traffic flow divided by PM NB traffic flow. The peak 15-minute volume occurred in 7:45 am - 8:00 am in the morning and 5:15 pm - 5:30 pm in the evening. AM/PM peak hour flow rates were calculated using corresponding peak 15-minute traffic volume. The AM/PM peak hour flow rates were listed in Table 67 and Table 68.

Table 67 AM Peak Hour Flow Rate of Briarwood Dr& State St

Briarwood Dr (AM)	N (Out)			S (In)			W		
	L (None)	T	R	L	T	R (None)	L (In)	T (None)	R (Out)
# of Lanes		2	1	1	2		2		1
Critical Vol. (VPH)		476	152	112	564		153		92

Table 68 PM Peak Hour Flow Rate of Briarwood Dr & State St

Briarwood Dr (PM)	N (Out)			S (In)			W		
	L (None)	T	R	L	T	R (None)	L (In)	T (None)	R (Out)
# of Lanes		2	1	1	2		2		1
Critical Vol. (VPH)		700	240	84	420		114		112

Figure 42 and Figure 43 showed the volumes of turning movement in AM/PM peak period. SB and WB are inbound approaches, while NB is an outbound approach.

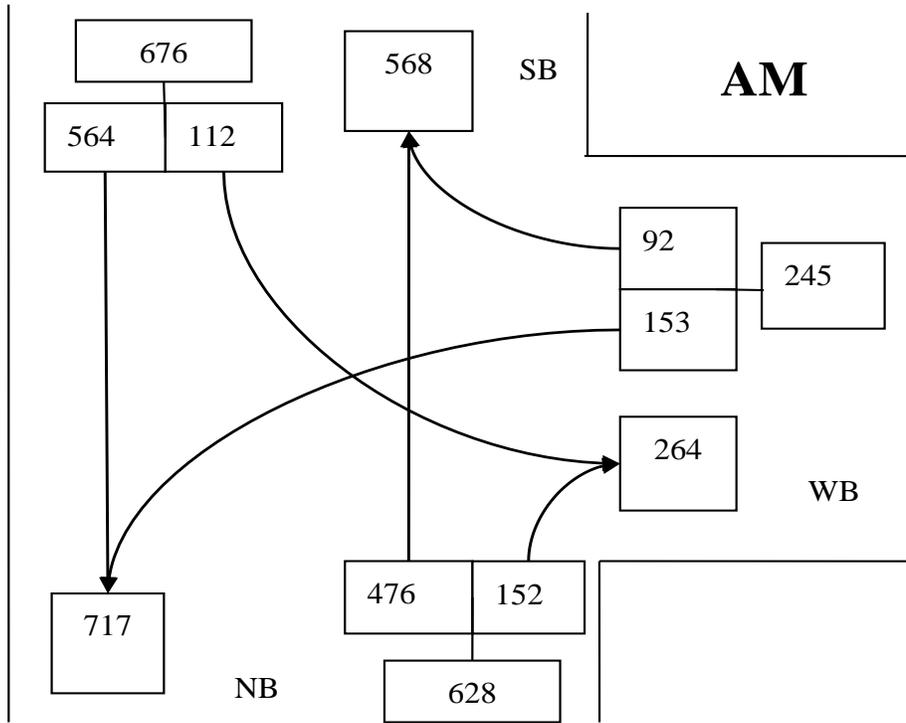


Figure 42 AM Peak Turning Movement Volume of Briarwood Dr& State St

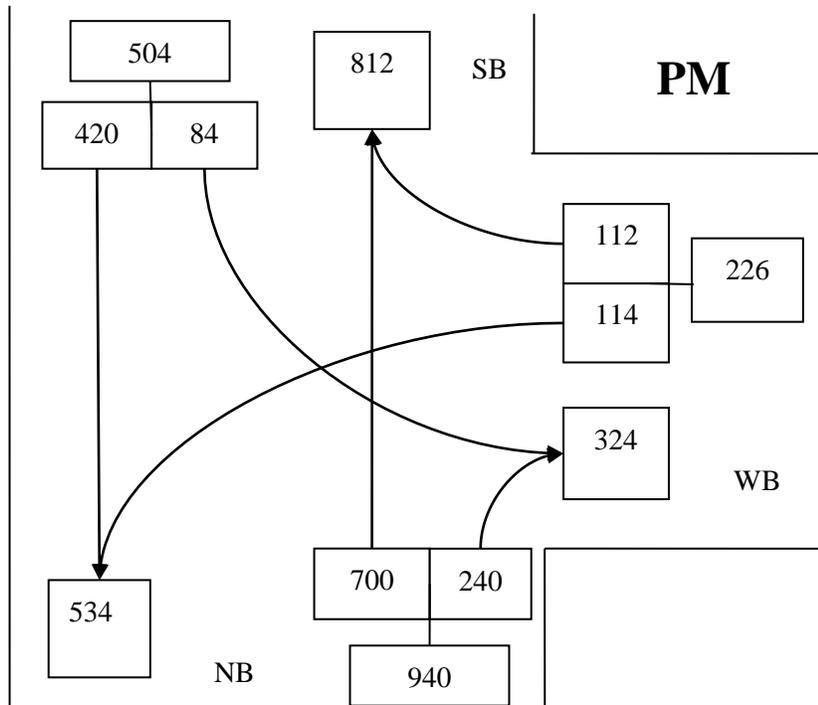


Figure 43 PM Peak Turning Movement Volume of Briarwood Dr& State St

12. Beasley Rd & State St

Beasley Rd & State St is a four-leg intersection. The intersection’s traffic data source, AM/PM peak hour flow rates, and AM/PM turning movements were provided below.

Table 69 Traffic Data Source

Beasley Rd & State St				
Approach	NB	SB	EB	WB
Device	Manual Count	Manual Count	Manual Count	Manual Count

Manual count was used to collect traffic data at Beasley Rd & State St during the PM period. The PM peak 15-minute volume occurred at 5:00 pm - 5:15 pm. The PM peak hour flow rate was calculated based on the peak 15-minute volume. The AM peak hour flow rate was estimated by PM peak hour flow rate multiplying the briarwood factor. The AM/PM peak hour flow rates were listed in Table 70 and Table 71, respectively.

Table 70 AM Peak Hour Volume of Beasley Rd & State St

Beasley Rd (AM)	N (Out)			S			W			E		
	L	T	R	L (Out)	T (In)	R (Out)	L (In)	T (Out)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1	1	1	1
Critical Vol. (VPH)	80	403	104	43	466	102	188	190	69	53	179	161

Table 71 PM Peak Hour Volume of Beasley Rd & State St

Beasley Rd (PM)	N (Out)			S			W			E		
	L	T	R	L (Out)	T (In)	R (Out)	L (In)	T (Out)	R (Out)	L (Out)	T (Out)	R (In)
# of Lanes	1	1	1 TR	1	1	1 TR	1	1	1	1	1	1
Critical Vol. (VPH)	120	604	156	64	348	152	140	284	104	80	268	120

Figure 44 and Figure 45 showed the volumes of turning movement in AM/PM peak period. SB is inbound approach, while WB, EB, and NB are outbound approaches.

- **Variations**

Field data variations were studied in this section. Traffic data recorded by Radar and NC 200 were used to analyze variations because these two devices are able to collect traffic data over a long period (several hours or days). Radars and NC 200s were implemented at 6 intersections to collect data. They are Amite St & State St (NB), capital St & State St (NB), Pascagoula St & State St (SB), Pearl St & State St (NB), Briarwood Drive & State St (all approaches), and Old Canton Rd & State St (all approaches). Variations in terms of time of day and peaks at these intersections were studied, respectively. The results were discussed as following.

1. Amite St & State St

Amite St & State St NB traffic data was collected by NC 200 for 18 hours from 3: 15 pm Oct 12, 2010 to 9:15 am Oct 13, 2010. The hourly volumes and time of day factors were shown in Table 72.

Table 72 Hourly Volumes and Time of Day of Amite St & State St NB

Date	Time	Volume	Time of day (%)	Date	Time	Volume	Time of day (%)
Oct 12 th 2010	16:00-16:59	724	13.7	Oct 13 th 2010	01:00-01:59	31	0.6
	17:00-17:59	831	15.7		02:00-02:59	18	0.3
	18:00-18:59	518	9.8		03:00-03:59	10	0.2
	19:00-19:59	363	6.9		04:00-04:59	21	0.4
	20:00-20:59	195	3.7		05:00-05:59	52	1.0
	21:00-21:59	182	3.4		06:00-06:59	255	4.8
	22:00-22:59	136	2.6		07:00-07:59	1012	19.2
	23:00-23:59	75	1.4		08:00-08:59	816	15.5
Oct 13 th 2010	0:00-0:59	40	0.8	Total	17 hours	5279	100

Figure 46 showed variation of NB left turn and through movement in 15 minutes interval. In the figure, AM peak period was 7:30 am - 8:30 am and PM peak period was round 16:15 pm - 18:00 pm which had much heavier traffic than other periods. Through traffic was considered as critical movement in NB which had more traffic than left turn. It also can be seen that PM peak period had more traffic and last longer than AM peak period.

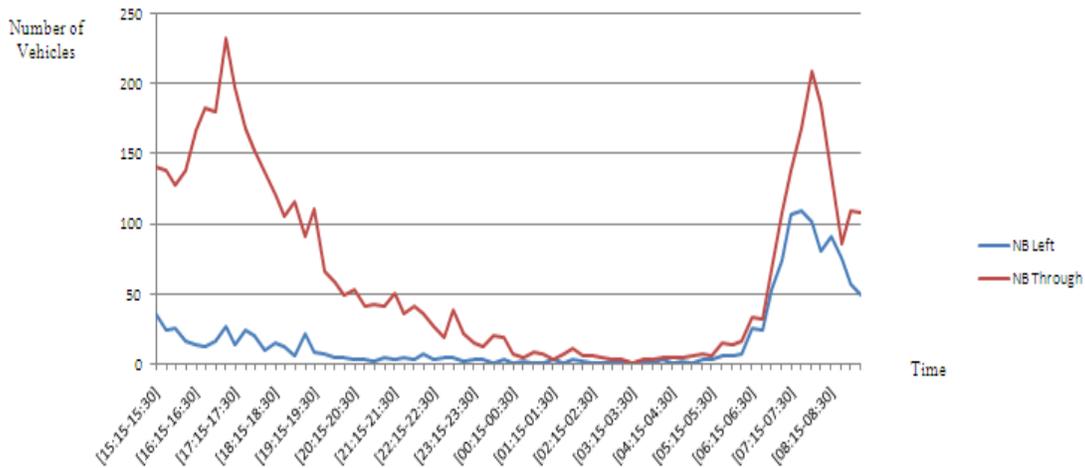


Figure 46 Variations of NB Traffic in Amite St & State St

2. Capitol St & State St

NB of Capitol St & State St intersection was monitored by Radar. Capital St is one-way street from west to east. There is no left turn and right turn traffic from NB. Hourly volumes and time of day factors of NB were shown in Table 73.

Table 73 Hourly Volumes and Time of Day of Capitol St & State St NB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 12 th 2010	16:00-17:00	496	13.5	Oct 13 th 2010	0:00-1:00	27	0.7
	17:00-18:00	623	17		1:00-2:00	21	0.6
	18:00-19:00	435	11.9		2:00-3:00	13	0.3
	19:00-20:00	315	8.6		3:00-4:00	6	0.2
	20:00-21:00	128	3.5		4:00-5:00	15	0.4
	21:00-22:00	134	3.7		5:00-6:00	53	1.4
	22:00-23:00	95	2.6		6:00-7:00	249	6.8
	23:00-24:00	50	1.4		7:00-8:00	1004	27.4
Total	Time			Volume		Time of Day (%)	
	16 hours			3664		100	

Variations of 15-minute traffic of NB were shown in Figure 47. AM peak period was around 6:45 am - 8:45 am and PM peak period was between 4:00 pm and 6:45 pm. From the figure, it can be

found that AM peak period had more traffic than PM peak period which indicated that NB is inbound approach.

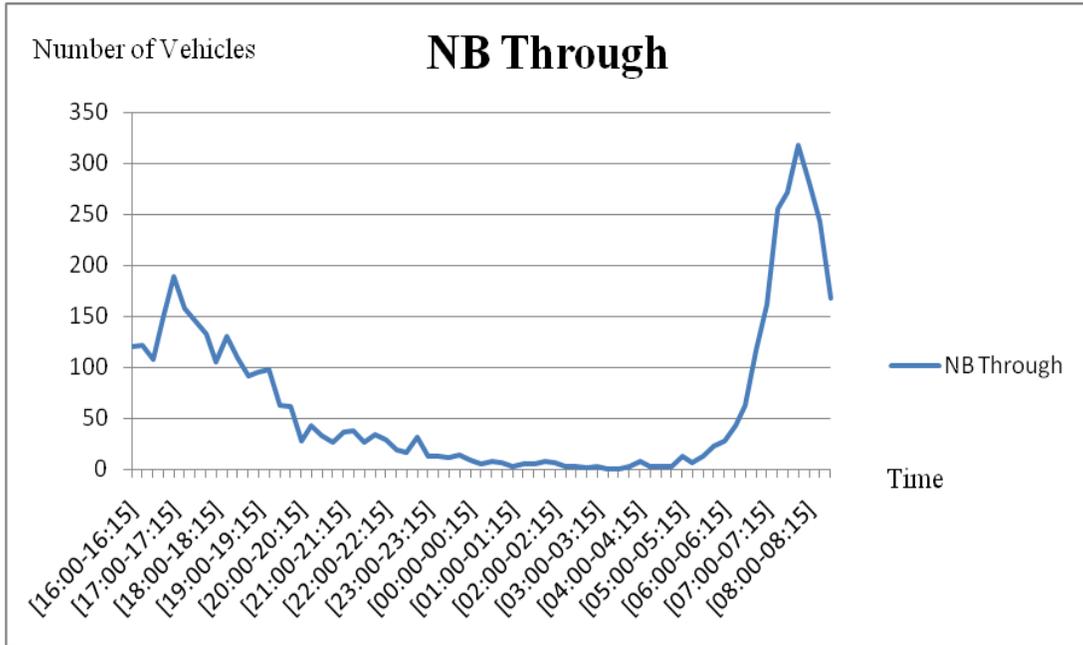


Figure 47 Variations of 15-Minute Traffic of Capitol St & State Street NB

3. Pascagoula St & State St

Pascagoula St & State St SB data were collected by Radar. Hourly volumes and time of day factors of SB were shown in Table 74.

Table 74 Hourly Volumes and Time of Day of Pascagoula St & State St SB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 12 th 2010	17:00-18:00	832	34.7	Oct 13 th 2010	1:00-2:00	15	0.6
	18:00-19:00	340	14.2		2:00-3:00	6	0.3
	19:00-20:00	229	9.6		3:00-4:00	14	0.6
	20:00-21:00	176	7.3		4:00-5:00	9	0.4
	21:00-22:00	161	6.7		5:00-6:00	54	2.3
	22:00-23:00	148	6.2		6:00-7:00	84	3.5
	23:00-24:00	62	2.6		7:00-8:00	238	9.9
Oct 13 th 2010	0:00-1:00	26	1.1	Total	15 hours	2394	100

Figure 48 showed variations of 15-minute traffic volume of Pascagoula St & State St SB. In the figure, PM peak period had much heavier traffic than AM peak period. The AM peak period was around 7:15 am - 8:30 am and PM peak period was around 4:45 pm - 6:45 pm. It could be found that through traffic was much heavier than left turns in AM peak period but turned to be less than left turns in PM peak period.

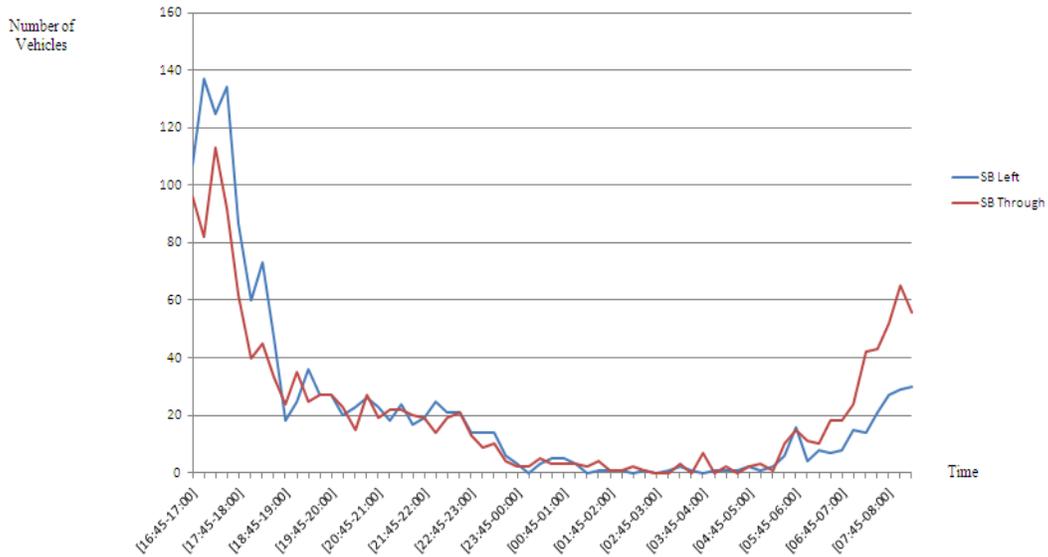


Figure 48 Variations of SB Traffic of Pascagoula St & State St

4. Pearl St & State St

Pearl St & State St NB is monitored by Radar. Hourly volumes and time of day factors of NB were provided in Table 75.

Table 75 Hourly Volumes and Time of Day of NB in Pearl St & State St Intersection

Data	Time	Volume	Time of Day (%)	Data	Time	Volume	Time of Day (%)
Oct 12 th 2010	12:00-13:00	579	12	Oct 12 th 2010	22:00-23:00	107	2.2
	13:00-14:00	541	11.2		23:00-24:00	59	1.2
	14:00-15:00	459	9.5	Oct 13 th 2010	0:00-1:00	34	0.7
	15:00-16:00	406	8.4		1:00-2:00	18	0.4
	16:00-17:00	697	14.4		2:00-3:00	12	0.2
	17:00-18:00	762	15.7		3:00-4:00	11	0.2
	18:00-19:00	419	8.6		4:00-5:00	11	0.2
19:00-	256	5.3	5:00-6:00	37	0.8		

	20:00						
	20:00-21:00	162	3.3		6:00-7:00	111	2.3
	21:00-22:00	164	3.4	Total	19 hours	4845	100

NB left and through traffic variations by 15-minute were shown in Figure 49. There were three peak periods (AM, Noon and PM) for NB through traffic. The AM peak began around 7:15 am. The end time of AM peak was not available because of no sufficient data. Our traffic counts ended at 7:45 am and no data after 7:45 am. The Noon peak was around 11:45 am - 3:00 pm. The PM peak was around 3:30 pm - 6:30 pm. Left traffic had more traffic in PM peak period than the other periods. NB traffic was heavy in most time of a day expect for the period from 8:00 pm to 6:45 am.

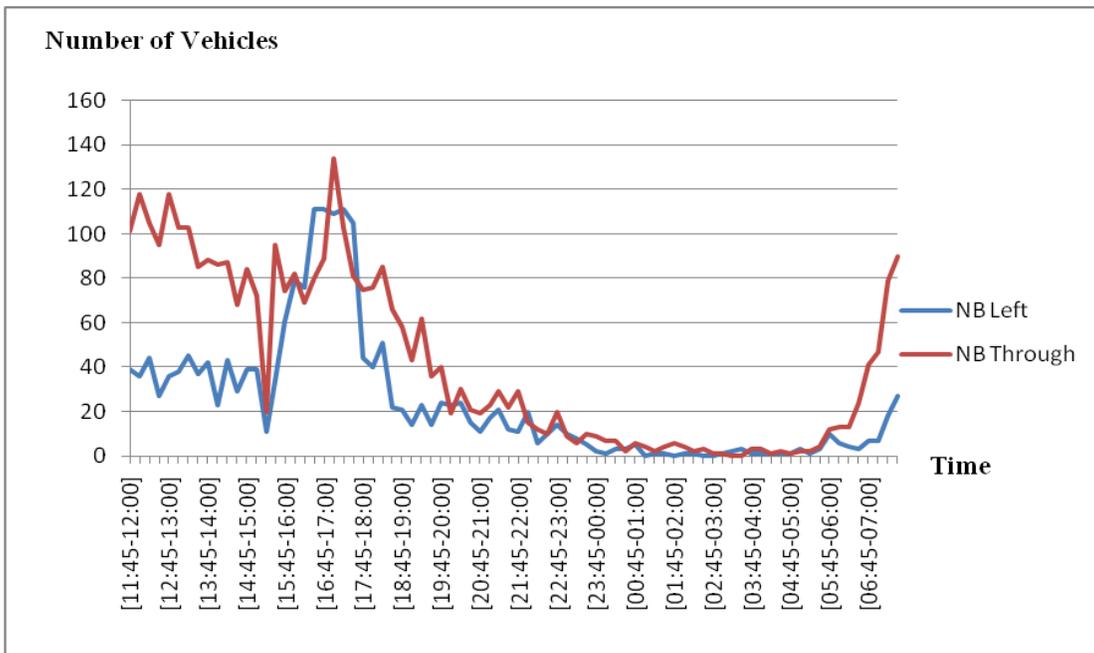


Figure 49 Variations of NB Left and Through Traffic in Pearl St & State St

5. Briarwood Dr & State St

Traffic data at Briarwood Dr & State St is collected by both Radar and NC 200 for all approaches. Information about hourly volumes and time of day for each approaches are available.

a. Northbound

NB through traffic data was collected by Radar and right turn traffic data was collected by NC 200. The hourly volumes and time of day was shown in Table 76.

Table 76 Hourly Volumes and Time of Day of Briarwood Drive & State St NB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 11 th 2010	14:00-15:00	568	10.7	Oct 12 th 2010	0-1	72	1.4
	15:00-16:00	611	11.5		1--2	28	0.5
	16:00-17:00	699	13.2		2--3	24	0.5
	17:00-18:00	788	14.9		3--4	26	0.5
	18:00-19:00	428	8.1		4--5	27	0.5
	19:00-20:00	353	6.7		5--6	39	0.7
	20:00-21:00	208	3.9		6--7	146	2.8
	21:00-22:00	144	2.7		7--8	420	7.9
	22:00-23:00	151	2.9		8--9	483	9
	23:00-24:00	83	1.6	Total	19 hours	5298	100

Figure 50 showed 15-minute traffic variations of Briarwood Drive & State St NB. The PM peak period had more traffic and lasted longer than AM peak period. AM peak period was round 7:30 am - 8:45 am and PM peak period was around 1:30 pm and 6:30 pm.

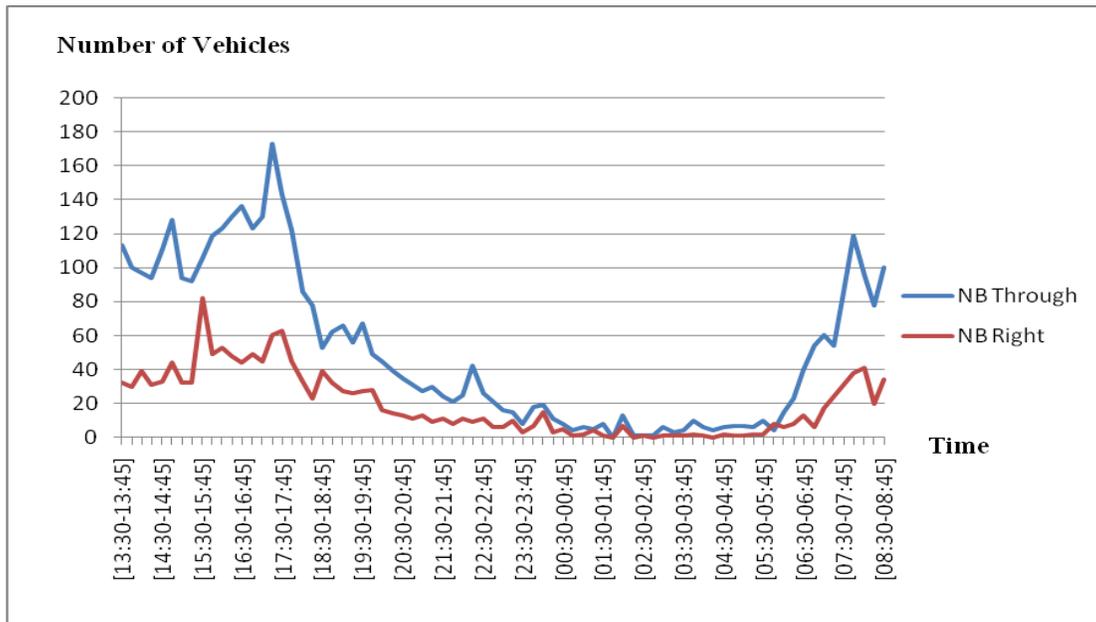


Figure 50 Variations of Through and Right Traffic of Briarwood Drive & State St NB

b. Southbound

SB traffic data (left and through) were collected by NC 200. Because of a hardware failure, SB left turn traffic data from 9:45 am to 11:00 am were lost. The missing data were estimated by the research team using extrapolate when calculated hourly volumes of 9-10 and 10-11. Hourly volumes and time of day factors were shown in Table 77.

Table 77 Hourly Volumes and Time of Day of Briarwood Drive & State St SB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 11 th 2010	11--12	364	5.9	Oct 12 th 2010	0-1	45	0.7
	12--13	444	7.2		1--2	20	0.3
	13--14	450	7.3		2--3	9	0.2
	14-15	441	7.2		3--4	9	0.2
	15-16	489	7.9		4--5	18	0.3
	16-17	457	7.4		5--6	47	0.8
	17-18	479	7.8		6--7	210	3.4
	18-19	364	5.9		7--8	512	8.3
	19-20	279	4.5		8--9	395	6.4
	20-21	192	3.1		9--10	406	6.6
	21-22	160	2.6		10--11	173	2.8
	22-23	114	1.9		Total	24 hours	6155

SB left and through traffic data variations by 15-minute interval were shown in Figure 51. SB traffic could be separated by two periods: peak period from 6:15 am to 21:45 pm and non-peak period from 21:45 pm to 6:15 am. From Figure 51, we found that AM peak was around 7:15 am - 8:30 am and PM peak was around 11:15 am - 18:30 pm.

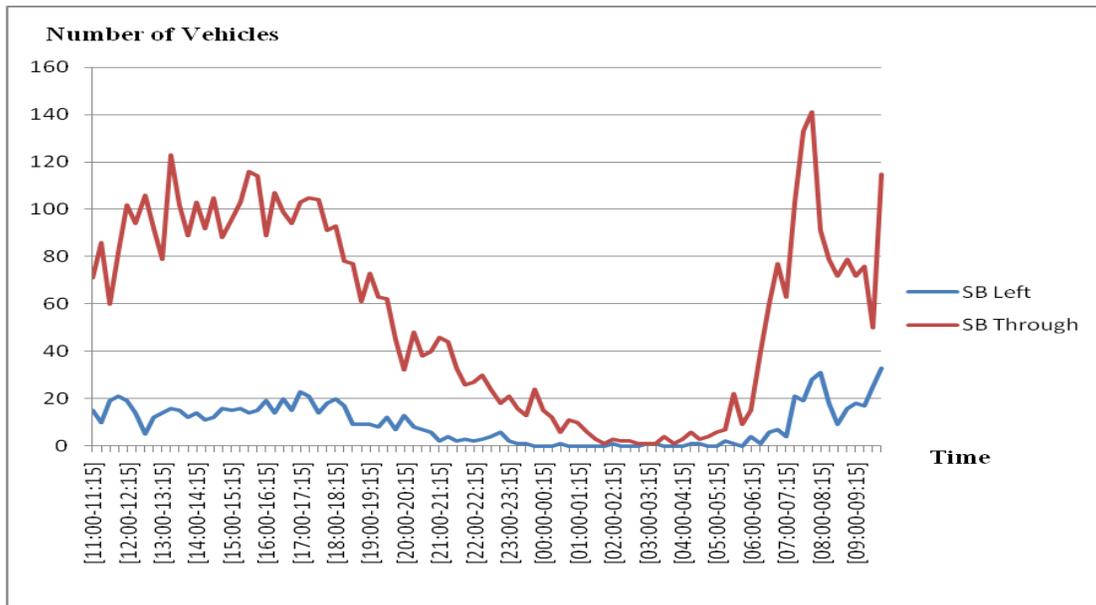


Figure 51 Variations of Left and Through Traffic of Briarwood Drive & State St SB

c. Westbound

WB left turn traffic data was collected by Radar. But due to a hardware failure, the radar didn't recorded data. WB left turn traffic data were manually counted by 20 minutes which were used to estimate AM/PM peak hour volume. However, there was no continuous WB left turn traffic data for variations study. WB right turn traffic data was collected by NC 200 which was available for variation analysis. Hourly volumes and time of day factors of WB right turn were provided below.

Table 78 Hourly Volumes and Time of Day of Right Turn Traffic of Briarwood Drive & State St WB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 11 th 2010	11:00-12:00	64	5.7	Oct 12 th 2010	0:00-1:00	13	1.2
	12:00-13:00	71	6.3		1:00-2:00	4	0.4
	13:00-14:00	81	7.2		2:00-3:00	1	0.1
	14:00-15:00	76	6.8		3:00-4:00	4	0.4
	15:00-16:00	87	7.7		4:00-5:00	3	0.3
	16:00-17:00	101	9		5:00-6:00	1	0.1
	17:00-18:00	118	10.5		6:00-7:00	18	1.6
	18:00-19:00	85	7.6		7:00-8:00	52	4.5
	19:00-20:00	49	4.3		8:00-9:00	57	5.1
	20:00-21:00	30	2.7		9:00-10:00	86	7.7
	21:00-22:00	35	3		10:00-11:00	55	4.9
	22:00-23:00	28	2.5		Total	24 hours	1124

WB right turn traffic data variations were displayed in Figure 52. It can be found that the PM peak period lasted longer than AM peak period. AM peak period was around 9:30 am - 10:30 am and PM peak period was roughly between 1:00 pm and 6:45 pm.

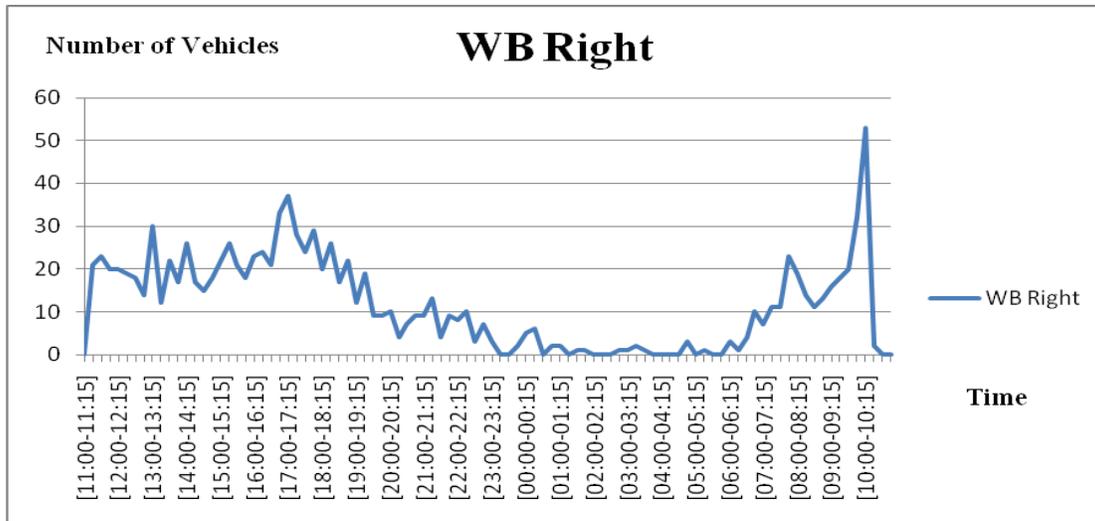


Figure 52 Variations of Right Turn Traffic of Briarwood Drive & State St WB

d. Whole Intersection

Figure 53 displayed variations of all turning movements in the intersection. It showed that NB/SB through traffics were critical for the intersection which had more traffic than other turning movements. The intersection's AM peak period was around 7:15 am - 8:30 am and its PM peak period was between 15:15 pm and 18:15 pm.

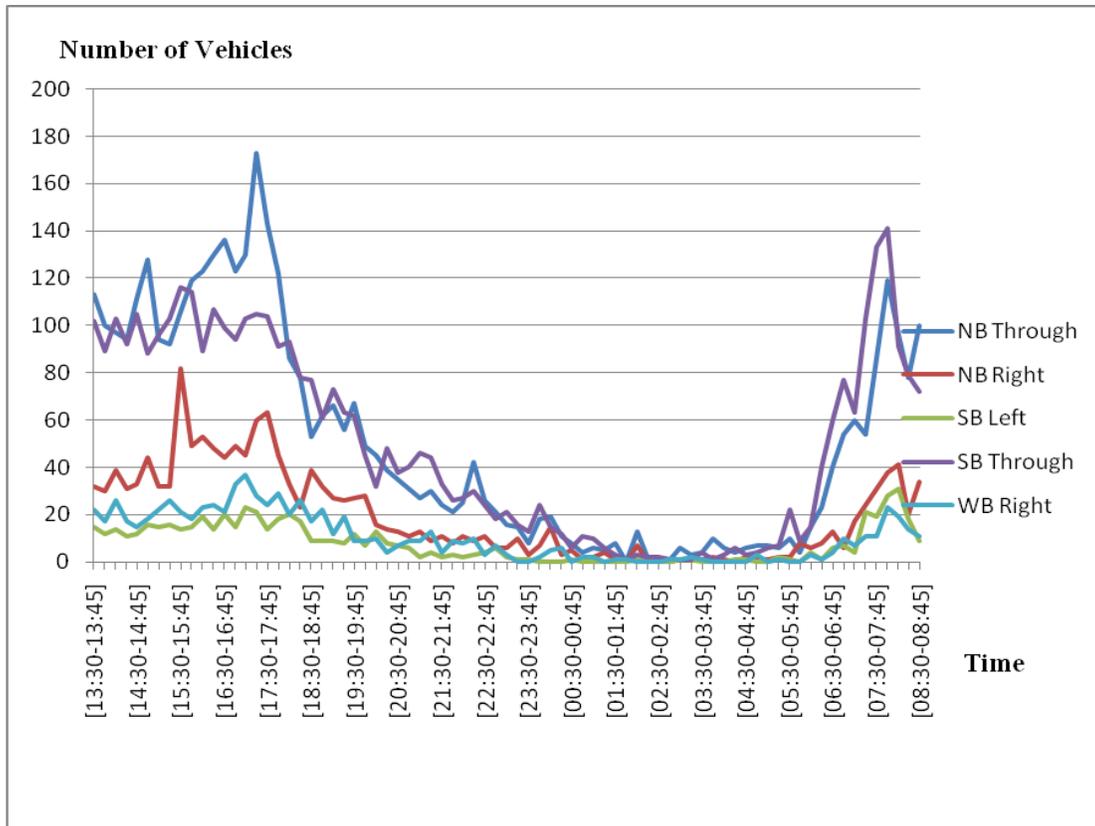


Figure 53 Variations of All Turning Movement of Briarwood Drive & State St

6. Old Canton Rd & State St

Same as Briarwood Dr & State St, Radar and NC 200 were used to collect traffic data at Old Canton Rd & State St. Traffic data from Oct 13th 2010 to Oct 15th 2010 were recorded. But Oct 14th is the only day that had 24 hours data. For Oct 13th and 15th, traffic data were just recorded for several hours. So, traffic data of Oct 14th were used for hourly volumes and time of day analysis for all approaches. 15-minute traffic volume data for three days were used for variations study for all approaches which would be better to show the volumes change than one day's data.

a. Northbound

NB through traffic data was collected by Radar and NB right turn traffic data was collected by NC 200. Hourly volumes and time of day factors of NB was shown in Table 79.

Table 79 Hourly Volumes and Time of day of Old Canton Rd & State St NB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 14 th 2010	00:00-01:00	74	0.5	Oct 14 th 2010	12:00-13:00	1076	7.5
	01:00-02:00	32	0.2		13:00-14:00	969	6.7

	02:00-03:00	27	0.2		14:00-15:00	1044	7.3
	03:00-04:00	14	0.1		15:00-16:00	1223	8.5
	04:00-05:00	36	0.3		16:00-17:00	1417	9.9
	05:00-06:00	109	0.8		17:00-18:00	1291	9
	06:00-07:00	254	1.8		18:00-19:00	815	5.7
	07:00-08:00	701	4.9		19:00-20:00	652	4.5
	08:00-09:00	741	5.2		20:00-21:00	369	2.6
	09:00-10:00	706	4.9		21:00-22:00	406	2.8
	10:00-11:00	823	5.7		22:00-23:00	316	2.1
	11:00-12:00	1097	7.6		23:00-24:00	185	1.2
Daily Total Volume	14377			Total Time of Day (%)	100		

Figure 54 showed NB traffic data variations in the three days. From the figure, it can be seen that there were three peak periods (AM, Noon and PM) per day at this intersection. The AM, Noon, and PM peak periods were around 7:00 am to 9:30 am, 10:45 am to 1:45 pm, and 2:45 pm to 5:30 pm respectively.

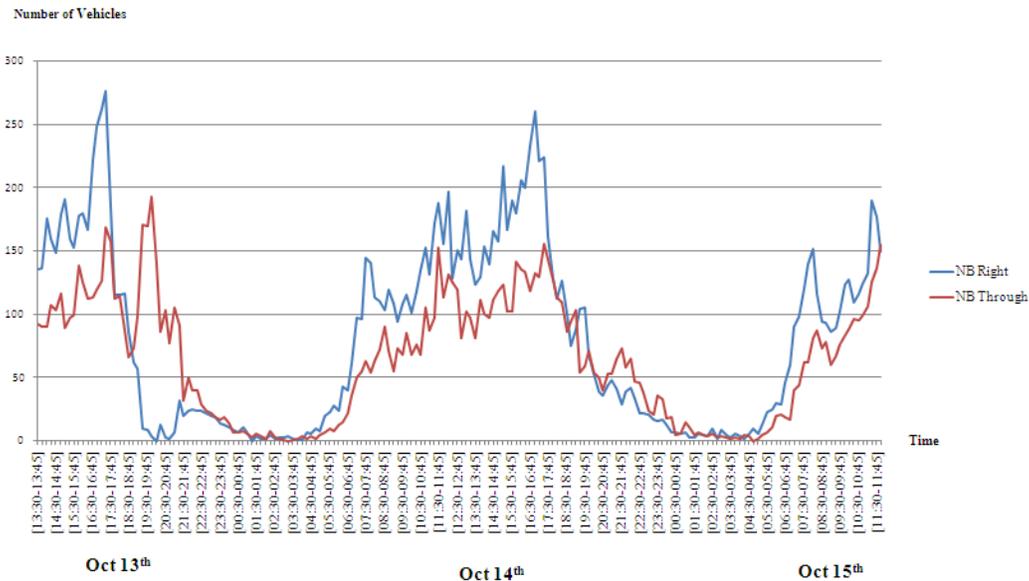


Figure 54 Variations of Through and Right Traffic of Old Canton Rd & State St NB

b. Southbound

SB traffic data was collected by NC 200. Traffic data from Oct 14th was used for hourly volumes and time of day factors analysis.

Table 80 Hourly Volumes and Time of Day of Old Canton Rd & State St SB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 14 th 2010	00:00-01:00	17	0.4	Oct 14 th 2010	12:00-13:00	435	9
	01:00-02:00	6	0.1		13:00-14:00	377	7.8
	02:00-03:00	12	0.2		14:00-15:00	328	6.8
	03:00-04:00	7	0.1		15:00-16:00	323	6.7
	04:00-05:00	9	0.2		16:00-17:00	312	6.5
	05:00-06:00	47	1		17:00-18:00	420	8.7
	06:00-07:00	146	3		18:00-19:00	387	8
	07:00-08:00	407	8.4		19:00-20:00	174	3.6
	08:00-09:00	302	6.3		20:00-21:00	133	2.8
	09:00-10:00	228	4.7		21:00-22:00	94	1.9
	10:00-11:00	223	4.6		22:00-23:00	87	1.8
	11:00-12:00	293	6.1		23:00-24:00	56	1.3
Daily Total Volume		4823		Daily Time of Day (%)		100	

Same as NB, three days' traffic data of SB were used for SB traffic data variations analysis. Figure 55 showed the traffic data variations of three days. SB has three peak periods which is similar to NB. The AM, Noon and PM periods appeared around 7:00 am - 8:45 am, 11:30 am - 2:00 pm, and 4:00 pm - 7:15 pm respectively.

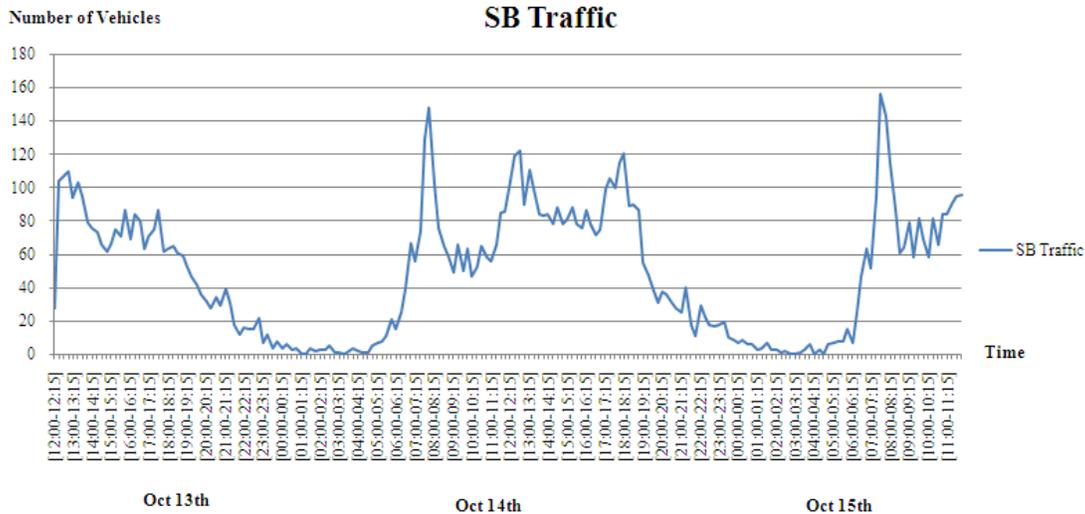


Figure 55 Traffic Data Variations of Old Canton Rd & State St SB

c. Westbound

WB data were collected by Radar. Traffic data from Oct 14th was used to analyze hourly volumes and time of day factors. The results were shown in Table 81.

Table 81 Hourly Volumes and Time of Day of Old Canton Rd & State St WB

Date	Time	Volume	Time of Day (%)	Date	Time	Volume	Time of Day (%)
Oct 14 th 2010	00:00-01:00	35	0.4	Oct 14 th 2010	12:00-13:00	749	8.2
	01:00-02:00	16	0.2		13:00-14:00	608	6.6
	02:00-03:00	8	0.1		14:00-15:00	597	6.5
	03:00-04:00	7	0.1		15:00-16:00	595	6.5
	04:00-05:00	22	0.2		16:00-17:00	643	7
	05:00-06:00	74	0.8		17:00-18:00	906	9.9
	06:00-07:00	338	3.7		18:00-19:00	685	7.5
	07:00-08:00	672	7.3		19:00-20:00	343	3.7
	08:00-09:00	647	7		20:00-21:00	219	2.4
	09:00-10:00	517	5.6		21:00-22:00	155	1.7
	10:00-11:00	501	5.5		22:00-23:00	133	1.4
11:00-12:00	636	6.9	23:00-24:00	74	0.8		
Daily Total Volume		9180		Daily Time of Day (%)		100	

WB data variations were analyzed for three days from Oct 13th to Oct 15th, 2010. Figure 56 showed the data variation for WB. It is easy to find that WB also had three peak periods per day which is similar to NB and SB. AM, Noon, and PM peak periods happened around 6:45 am - 9:15 am, 10:30 am - 1:45 pm, and 4:15 pm - 6:45 pm respectively.

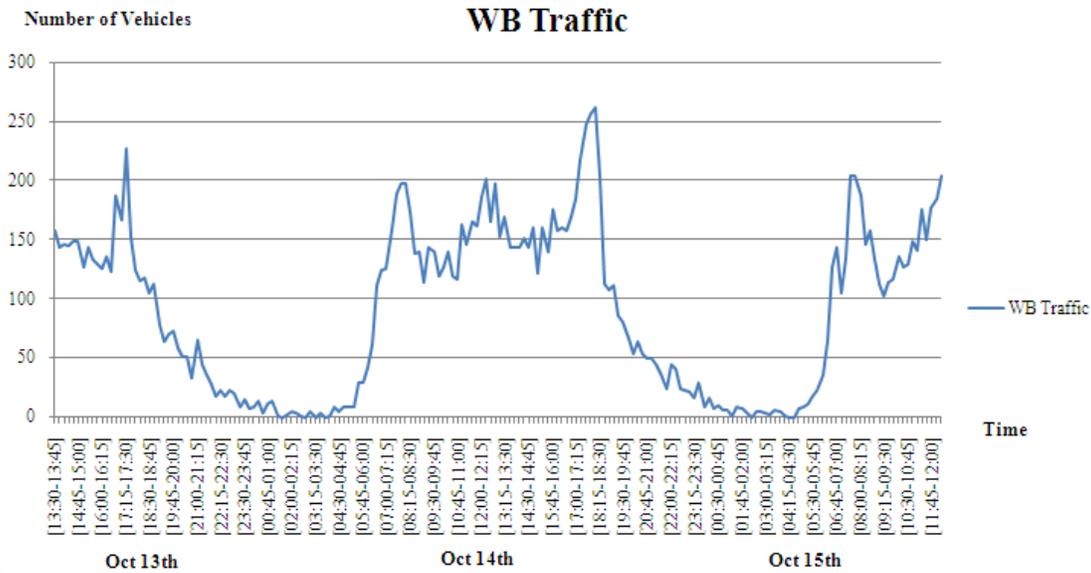


Figure 56 Traffic Data Variations of Old Canton Rd & State St WB

d. Whole Intersection

Figure 57 showed all turning movements data variations of all approaches at Old Canton Rd & State St. It can be seen that all turning movements' traffic data had AM, Noon and PM peak periods. AM peak was around 6:45 am - 9:15 am; Noon peak was around 10:45 am - 1:45 pm; PM peak was around 3:45 pm - 6:45 pm. PM peak period had the highest traffic volume. It also can be found that SB had the lowest traffic volume within all approaches.

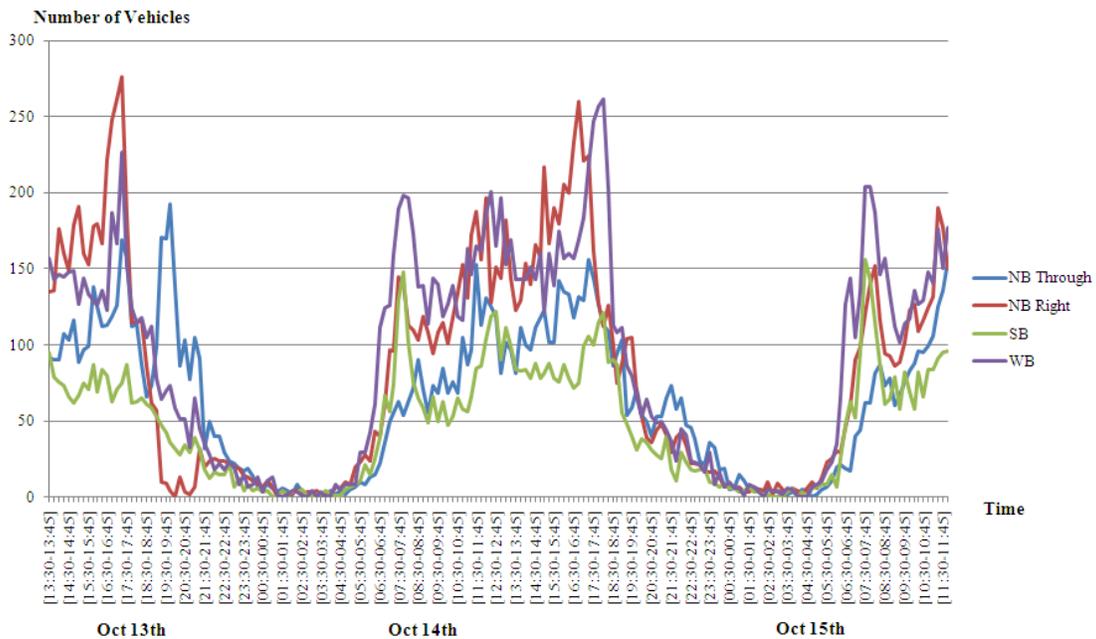


Figure 57 Variations of All Turning Movements of Old Canton Rd & State St All Approaches

Based on the discussion above, Table 82 summarized AM/PM peak periods of all six intersections on State St. It can be seen that AM peak was around 7:30 am - 8:30 am and PM peak was around 4:45 pm - 5:30 pm for all six intersections on State St.

Table 82 Traffic Patterns on State St (U.S. 51)

Intersection Name	Approach	AM Peak Period	Noon Peak Period	PM Peak Period
Amite St & State St	NB	7:30 am - 8:30 am	-----	4:15 pm - 6:00 pm
Capitol St & State St	NB	6:45 am - 8:45 am	-----	4:00 pm - 6:45 pm
Pascagoula St & State St	SB	7:15 am - 8:30 am	-----	4:45 pm - 6:45 pm
Pearl St & State St	NB	Begin 7:15 am	11:45 am - 3:00 pm	3:30 pm - 6:30 pm
Briarwood Dr & State St	NB	7:30 am - 8:45 am	-----	1:30 pm - 6:30 pm
	SB	7:15 am - 8:30 am	-----	11:15 am - 6:30 pm
	WB ¹	9:30 am - 10:30 am	-----	1:00 pm - 6:45 pm
	ALL	7:15 am - 8:30 am	-----	3:15 pm - 6:15 pm
Old Canton Rd & State St	NB	7:00 am - 9:30 am	10:45 am - 1:45 pm	2:45 pm - 5:30 pm
	SB	7:00 — 8:45	11:30 am - 2:00 pm	16:00—19:15
	WB	6: 45 — 9:15	10:30 am - 1:45 pm	16:15—18:45
	ALL	6:45—9:15	10:45 am - 1:45 pm	15:45—18:45

- **Travel Time / Average Speed Study**

Global Position System (GPS) was utilized to collect travel time data from intersection to intersection among twelve intersections. The travel time data between two intersections were analyzed in terms of directions and AM/PM. Average speed traversing two successive intersections was calculated using the travel time data. Variations of travel time and average speed between two neighboring intersections, in terms of directions, AM/PM change, were studied. Standard deviation of travel time and average speed were computed as well. In the calculation procedure, leg lengths (travel distances) between two intersections in different experiments are slightly different because it was hard to keep identical start and end points for the vehicle traversing between two intersections in different experiments.

1. Pascagoula St to Pearl St

Pascagoula St and Pearl St locate in the downtown area. Table 83 and Table 84 showed the travel time and average speed between these two intersections. In Table 83, AM travel time was slightly longer than PM travel time, which meant that travel speed in the morning was lower than the afternoon. And, there were more traffics traveling from Pascagoula St to Pearl St in the morning than the afternoon.

Table 83 Travel Time and Average Speed from Pascagoula St to Pearl St

Pascagoula St to Pearl St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:45	342	10	23	1	16:48	350	8	30
2	8:27	340	9	26	2	17:57	341	8	29
3	7:30	353	10	24					

MEAN	9.7	24.3	MEAN	8	29.5
STANDARD DEVIATION	0.58	1.25	STANDARD DEVIATION	0.00	0.54

In Table 84, the traffic pattern from north to south is opposite to the traffic pattern from south to north. More traffic traveled from north to south in the afternoon than in the morning. The travel time in the afternoon was longer than in the morning. And the travel speed in the afternoon was much lower than in the morning. This trend was more obvious than south-north direction. It can be concluded that State St from Pascagoula St to Pearl St carried inbound traffic, while it carried outbound traffic in the opposite direction.

Table 84 Travel Time and Average Speed from Pearl St to Pascagoula St

Pearl St to Pascagoula St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:43	347	8	30	1	16:44	321	16	14
2	8:25	364	8	31	2	17:55	345	12	20
MEAN			8	30.5	MEAN			14	17
STANDARD DEVIATION			0.00	1.02	STANDARD DEVIATION			2.83	4.19

2. Pearl St to Capitol St

In Table 85, travel time from Pearl St to Capitol St in morning was a little longer than in the afternoon. But average travel speed in the afternoon was higher than in the morning which indicated the traffic conditions were better in the afternoon than in the morning. It is known that traffic volume has inverse trend with speed which means speed would increase along with decrease of traffic volume, and vice versa. It seems that more traffic traveled from Pearl St to Capitol St in the morning than in the evening.

Table 85 Travel Time and Average Speed from Pearl St to Capitol St

Pearl St to Capitol St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:45	364	9	28	1	16:48	408	8	35
2	8:28	339	8	29					
3	7:30	331	8	28	2	17:58	343	7	33
4	7:59	316	8	27					
MEAN			8.3	28	MEAN			7.5	34
STANDARD DEVIATION			0.5	0.84	STANDARD DEVIATION			0.71	0.96

In Table 86, travel time in the afternoon was obviously longer than in the morning in the north-south direction. Travel speed in the afternoon was less than half of the speed in the morning. It seems that this section of State St in north-south direction carried outbound traffic, and carried inbound traffic in the opposite direction.

Table 86 Travel Time and Average Speed from Capitol St to Pearl St

Capitol St to Pearl St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:42	340	7	33	1	16:44	344	33	7
2	8:25	362	8	31	2	17:55	351	12	20
MEAN			7.5	32	MEAN			22.5	13.5
STANDARD DEVIATION			0.71	1.60	STANDARD DEVIATION			14.85	9.08

3. Capitol St to Amite St

In Table 87, the mean travel time from Capitol St to Amite St was almost twice the mean travel time in the afternoon which means the mean travel speed in the morning was half of the mean travel speed in the afternoon. It is clear that more traffic traveled from Capitol St to Amite St in the morning than in the afternoon.

Table 87 Travel Time and Average Speed from Capitol St to Amite St

Capitol St to Amite St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:45	330	13	17	1	16:48	353	7	34
2	8:28	345	10	24					
3	7:31	353	12	20	2	17:58	365	7	36
4	8:00	313	23	9					
MEAN			14.5	17.5	MEAN			7	35
STANDARD DEVIATION			5.80	6.07	STANDARD DEVIATION			0	0.83

From Table 88, the traffic pattern in north-south direction was inverse to the south-north direction. More traffic traversed north-south direction in the afternoon than in the morning, which means, in the afternoon, travel time was longer and travel speed was lower than in the morning. It indicates that the section of State St between Capitol St to Amite St carried inbound traffic in the south-north direction and carried outbound traffic in the north-south direction.

Table 88 Travel Time and Average Speed from Amite St to Capitol St

Amite to Capitol									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:42	376	8	32	1	16:43	321	38	6
2	8:25	355	8	30					
3	7:58	328	7	32	2	17:55	332	16	14
4	8:19	341	18	13					
MEAN			10.3	26.8	MEAN			27	10
STANDARD DEVIATION			5.19	9.29	STANDARD DEVIATION			15.56	5.93

4. Amite St to High St

Table 89 showed that mean travel time from Amite St to High St in the morning was longer than in the afternoon and that the mean travel speed had opposite trend. It means that traffic volume was higher in the morning than in the evening.

Table 89 Travel Time and Average Speed from Amite St to High St

Amite St to High St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:46	1414	39	25	1	16:49	1365	26	36
2	8:29	1437	41	24					
3	7:31	1494	40	25	2	17:59	1583	36	30
4	8:00	1642	34	33					
MEAN			38.5	26.8	MEAN			31	33
STANDARD DEVIATION			3.11	4.17	STANDARD DEVIATION			7.07	4.11

From High St to Amite St, the mean travel time and travel speed was nearly the same in the morning and afternoon. Variations of travel time and travel speed in the morning and afternoon was not significant. South-north direction of State St between Amite St and High St carried inbound traffic. But traffic pattern in north-south direction of this section is not clear.

Table 90 Travel Time and Average Speed from High St to Amite St

High St to Amite St									
Morning					Afternoon				
Trip	Time	Leg Length	Travel Time	Speed (mph)	Trip	Time	Leg Length	Travel Time	Speed (mph)

		(ft)	(seconds)			(ft)	(seconds)		
1	7:42	1419	24	40	1	16:42	1343	36	25
2	8:25	1451	36	27					
3	7:58	1578	26	41	2	17:54	1391	19	50
4	8:19	1350	22	42					
MEAN			27	37.5	MEAN			27.5	37.5
STANDARD DEVIATION			6.22	7	STANDARD DEVIATION			12.02	17.31

5. High St to Fortification St

In Table 91, the mean travel speed from high St to Fortification St was almost equal in the morning and afternoon. Although the mean travel time in the morning was a little longer than in the afternoon, it might be caused by the travel distance in the morning which was slightly longer than in the afternoon. It indicates that traffic patterns in the morning and afternoon were similar.

Table 91 Travel Time and Average Speed from High St to Fortification St

High St to Fortification St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:48	2704	45	41	1	16:50	2712	47	39
2	8:30	2760	44	43					
3	7:33	2721	54	34					
4	8:02	2956	51	40					
MEAN			48.5	39.5	MEAN			47	39
STANDARD DEVIATION			4.80	3.62	STANDARD DEVIATION			~	~

Table 92 shows the similar situation with Table 91 that mean travel speed in the morning and afternoon was nearly the same. Travel time in the afternoon had an increase of 5.3 % than in the morning which was relatively small. It indicates traffic patterns are also similar in the morning and afternoon. Based on these results, we can conclude that the variations of travel time and travel speed were minor in both directions.

Table 92 Travel Time and Average Speed from Fortification St to High St

Fortification to High St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:41	2855	47	41	1	16:37	2769	58	33
2	8:23	2765	50	38					

3	7:57	2707	52	35	2	17:53	2809	42	46
4	8:19	2793	41	46					
MEAN			47.5	40	MEAN			50	39.5
STANDARD DEVIATION			4.80	4.79	STANDARD DEVIATION			11.31	9.23

6. Fortification St to Woodrow Wilson Ave

In Table 93, the mean travel time from Fortification St to Woodrow Wilson Ave in the afternoon was longer than in the morning which means that the mean travel speed was lower in the afternoon than in the morning. It indicates that traffic volume in the afternoon was higher than in the morning.

Table 93 Travel Time and Average Speed from Fortification St to Woodrow Wilson Ave

Fortification St to Woodrow Wilson Ave									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:50	5529	113	33	1	16:53	5422	118	31
2	8:33	5363	115	32					
3	7:35	5417	120	31	2	17:34	5443	136	27
4	8:06	5476	137	27					
MEAN			121.3	30.8	MEAN			127	29
STANDARD DEVIATION			10.90	2.59	STANDARD DEVIATION			12.73	2.86

In Table 94, more traffic traveled from Woodrow Wilson Ave to Fortification St in the morning than in the afternoon. The mean travel time in the morning was significantly larger than in the afternoon. The mean travel speed in the afternoon was obviously higher than in the morning. It shows that, in this section of State St, the south-north direction is outbound, while, the north-south direction is inbound.

Compared the traffic patterns between this section of State St and other the five sections discussed above and we found that south-north direction of State St between Fortification St and Woodrow Wilson Ave is outbound, and north-south direction of this section is inbound which is opposite to the above five sections whose south-north direction is inbound and north-south direction is outbound.

Table 94 Travel Time and Average Speed from Woodrow Wilson Ave to Fortification St

Woodrow Wilson Ave to Fortification St									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)

1	7:41	5564	146	26	1	17:52	5654	96	40
2	8:21	5442	104	36					
3	7:55	5413	140	26					
4	8:17	5421	141	26					
MEAN			132.8	28.5	MEAN			96	40
STANDARD DEVIATION			19.35	5.00	STANDARD DEVIATION			~	~

7. Woodrow Wilson Ave to Old Canton Rd

In Table 95, the mean travel time from Woodrow Wilson Ave to Old Canton Rd in the afternoon was nearly 10 seconds longer than in the morning. Mean travel speed in the afternoon was also lower than in the morning. Therefore, it indicated that traffic volume in the afternoon was higher than in the morning.

Table 95 Travel Time and Average Speed from Woodrow Wilson Ave to Old Canton Rd

Woodrow Wilson Ave to Old Canton Rd									
Morning					Afternoon				
Trip	Time	Leg Length (Ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:52	1707	58	20	1	16:55	1947	70	19
2	8:34	1773	60	20					
3	7:37	1940	63	21					
4	8:07	1938	46	29	2	17:37	1741	63	21
MEAN			56.8	22.5					
STANDARD DEVIATION			7.46	4.18	STANDARD DEVIATION			4.95	1.44

Table 96 showed that the traffic pattern from Old Canton Rd to Woodrow Wilson Ave were changed compared to the opposite direction. Mean travel time in the morning was longer than in the afternoon. Mean travel speed in the morning was lower than in the afternoon. These two results indicates that more traffic traveled from Old Canton Rd to Woodrow Wilson Ave in the morning than in the afternoon. Therefore, south-north direction of State St was outbound, while north-south direction of State St was inbound. Compared Table 95 with Table 96, and we found that the mean travel time in the morning in both directions was almost equal. Difference of mean travel speed in the morning in two directions was minor. It is reasonable to conclude that the traffic volumes from two directions in the morning were close to each other. South-north direction of State St was considered as outbound and the opposite direction was inbound which means traffic volume in the morning was less than in the afternoon.

Table 96 Travel Time and Average Speed from Old Canton Rd to Woodrow Wilson Ave

Old Canton Rd to Woodrow Wilson Ave									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:37	1702	61	19	1	17:48	1941	50	26
2	8:18	1923	53	25					
3	7:52	1956	45	30					
4	8:14	2083	66	22					
MEAN			56.3	24	MEAN			50	26
STANDARD DEVIATION			9.22	4.58	STANDARD DEVIATION			~	~

8. Old Canton Rd to Meadowbrook Rd

Table 97 showed that mean travel time from Old Canton Rd to Meadowbrook Rd in the morning was longer than the afternoon which means more traffic traveled from south to north in the morning. It seems that south-north of State St between Old Canton Rd and Meadowbrook Rd was inbound.

Table 97 Travel Time and Average Speed from Old Canton Rd to Meadowbrook Rd

Old Canton Rd to Meadowbrook Rd									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:55	7303	187	27	1	17:41	6740	169	27
2	8:38	6561	186	24					
MEAN			186.5	25.5	MEAN			169	27
STANDARD DEVIATION			0.71	1.82	STANDARD DEVIATION			~	~

There are only travel time and travel speed data from Meadowbrook Ave to Old Canton Rd in Table 98. Variations of travel time and travel speed between AM/PM are not available. The mean AM/PM travel time of south-north direction was larger than the opposite direction which indicates the south-north direction carried more traffic than the north-south direction.

Table 98 Travel Time and Average Speed from Meadowbrook Rd to Old Canton Rd

Meadowbrook Rd to Old Canton Rd				
Morning				
Trip	Time	Leg Length (ft)	Travel Time(seconds)	Speed (mph)
1	8:16	6770	136	34
2	8:13	6775	131	35

MEAN	133.5	34.5
STANDARD DEVIATION	3.54	0.93

9. Meadowbrook Rd to Northside Dr

In Table 99, the mean travel time from Meadowbrook Rd to Northside Drive in the morning was longer than the afternoon. Travel speed in the morning was slower than the afternoon. However, there were just two sample data in the morning and one sample data in the afternoon. The travel time and speed data in the afternoon is close to the average of the travel time and speed data in the morning. However, we cannot conclude that whether travel time in the morning is longer or shorter than the afternoon.

Table 99 Travel Time and Average Speed from Meadowbrook Rd to Northside Drive

Meadowbrook Rd to Northside Dr									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	7:57	2074	43	33	1	17:42	1997	50	27
2	8:46	2027	63	22					
MEAN			53	22.5	MEAN			50	27
STANDARD DEVIATION			14.14	7.74	STANDARD DEVIATION			~	~

Only PM travel time and travel speed data were collected for the north-south direction. Compared Table 99 with Table 100 and we found that AM travel time from Northside Dr to Meadowbrook Rd was longer than the opposite direction. It indicates more traffic traversed from Northside Dr to Meadowbrook Rd than the opposite direction.

Table 100 Travel Time and Average Speed from Northside Drive to Meadowbrook Rd

Northside Dr to Meadowbrook				
Morning				
Trip	Time	Leg Length (ft)	Travel Time(seconds)	Speed (mph)
1	7:31	2003	71	19
2	8:14	2146	62	24
MEAN			66.5	21.5
STANDARD DEVIATION			6.36	3.06

10. Northside Dr to Briarwood Dr

In Table 101, travel time from Northside Dr to Briarwood Dr in the afternoon was longer than the morning. Travel speed in the afternoon was slower than the morning. It indicates that the traffic volume from Northside Dr to Briarwood Dr in the afternoon was heavier than the morning. And, north-south State St carried the outbound traffic.

Table 101 Travel Time and Average Speed from Northside Dr to Briarwood Dr

Northside Dr to Briarwood Dr									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	8:01	9456	216	30	1	17:47	9723	253	26
2	8:51	9771	229	29					
MEAN			222.5	29.5	MEAN			253	26
STANDARD DEVIATION			9.19	0.53	STANDARD DEVIATION			~	~

In Table 102, the mean travel time from Briarwood Dr to Northside Dr in the morning was shorter than the opposite direction. Travel speed was also higher than the opposite direction. It indicates that less traffic traverse in State St north-south direction than south-north direction.

Table 102 Travel Time and Average Speed from Briarwood Dr to Northside Drive

Briarwood Dr to Northside Dr				
Morning				
Trip	Time	Leg Length (ft)	Travel Time(seconds)	Speed (mph)
1	7:28	9476	212	30
2	8:13	9962	190	36
MEAN			201	33
STANDARD DEVIATION			15.56	3.73

11. Briarwood Drive to Beasley Rd

The travel times from Briarwood Dr to Beasley Rd in the morning and afternoon were almost the same. The difference of travel speed in AM/PM was not significant either. It indicates that traffic conditions in the morning and afternoon were similar. Compared Table 103 to Table 104, we found that travel time in the morning from Beasley Rd to Briarwood Dr was longer than the opposite direction, which means that State St from Beasley Rd to Briarwood Dr carried more traffic than the opposite direction.

Table 103 Travel Time and Average Speed from Briarwood Dr to Beasley Rd

Briarwood Dr to Beasley Rd									
Morning					Afternoon				
Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)	Trip	Time	Leg Length (ft)	Travel Time (seconds)	Speed (mph)
1	8:02	2188	32	47	1	17:47	2103	33	43
MEAN			32	47	MEAN			33	43
STANDARD DEVIATION			~	~	STANDARD DEVIATION			~	~

Table 104 Travel Time and Average Speed from Beasley Rd to Briarwood Dr

Beasley Rd to Briarwood Dr				
Morning				
Trip	Time	Leg Length (ft)	Travel Time(seconds)	Speed (mph)
1	8:08	2173	43	36
MEAN			43	36
STANDARD DEVIATION			~	~

12. Beasley Road to County Line Road

Table 105 summarized the travel time and travel speed data in the afternoon from Beasley Rd to County Line Rd.

Table 105 Travel Time and Average Speed from Beasley Rd to County Line Rd

Beasley Rd to County Line				
Trip	Time	Leg Length (ft)	Travel Time(seconds)	Speed (mph)
1	17:50	5902	126	32
MEAN			126	32
STANDARD DEVIATION			~	~

13. Summary

After analyzing the travel time and travel speed data from 12 segments on State St, we found that south-to-north direction carried inbound traffic from Pascagoula St to High St and north-to-south direction carried outbound traffic (except High St to Amite St). For the State St section between High St and Fortification St, variations of two directions' AM/PM traffic were minor. State St from Fortification St to Old Canton Rd was outbound and the opposite direction was inbound. Furthermore, it can be found that people travel from Pascagoula to High St and from Old Canton Rd to Fortification St in the morning to enter the downtown area for work. In the afternoon, they travel in the opposite direction from the downtown area back to their home. It is a common traffic pattern of major city.

- **Other Parameters**

Discharge Headway

The discharge headway was found by watching the traffic by video streaming along North State Street. Video cameras on MDOT website, MDOTTRAFFIC (previous named MStTraffic) (<http://www.mdottraffic.com/>) were used to watch the traffic. Due to lack of sight distance, the only intersection that under surveillance along North State Street was Fortification Street & State St. The northbound through approach and southbound through and through/right approach were observed during the morning peak (7:30-8:10 a.m.) and the afternoon peak (4:30-5:15 p.m.) over three days. The data collected were analyzed and graphed. Average discharge headway was

calculated by averaging twenty groups' data. Since discharge headway turned to be stable after discharged five vehicles, the first fifth samples in each group were dropped. Each group's average discharge headway was calculated by using the headways from 6th car to the end of the queue. An average discharge headway of 2.346 seconds was found along Fortification Street (NB and SB). The averages of each group's data were shown in Table 106 as well as the average of all groups' data, which was used as the discharge headway for the intersection.

Table 106 Discharge Headway of Fortification St & State St

Fortification Street Discharge Headway Averages					
NB T		SB T		SB T/R	
Group Index	Average Headway (S)	Group Index	Average Headway (S)	Group Index	Average Headway (S)
1	3.170	1	2.050	1	2.680
2	2.583	2	2.560	2	2.166
3	2.000	3	2.509	3	1.938
4	2.850	4	2.088	4	2.090
5	2.077	5	1.962		
6	3.142	6	2.274		
7	2.420	7	2.146		
8	1.957				
9	2.256				
TOTAL AVERAGE (S)			2.346		

Free Flow Speed (FFS)

Speed data collected by Radar and NC 200 were used to calculate Free Flow Speed (FFS). FFS of an intersection was considered as 85th percentile of speed data collected in the field. The research team calculated FFS for 6 intersections, including Amite St & State St (NB), Capital St & State St (NB), Pascagoula St & State St (SB), Pearl St & State St (NB), Briarwood Drive & State St, and Old Canton Rd & State St. The results were presented as following.

1. Amite St & State St (NB)

Speed data at Amite St & State St was collected by NC 200. Three units of NC 200 were placed on the three lanes of NB. One unit was used for left turn lane and the other two units were implemented for two through lanes. Highway Data Management (HDM) is the software developed specially for NC 200. HDM can generate 85th percentile speed based on collected data. The generated 85th percentile speed of NB at Amite St & State St were shown in Table 107.

Table 107 85th Percentile Speed of each Lane of NB at Amite St & State St

	Left Lane	Through Lane 1	Through Lane 2
85 th percentile speed (mph)	23.79	35.52	33.77

In Table 107, left turn vehicles' speed is less than through vehicles' speed which are consist with field data. Left turn vehicle speed cannot represent FFS because left turn drivers need to slow

down and make a left turn even if there is almost no traffic at the intersection. The average of 85th percentile speed in two through lanes, 35 mph, was used as FFS of this approach.

2. Capitol St & State St (NB)

Capitol St & State St NB traffic data was collected by Radar. Based on the field speed data, cumulative percentage of speed distribution was shown in Figure 58. The 85th percentile speed is around 28 mph, which is close to FFS.

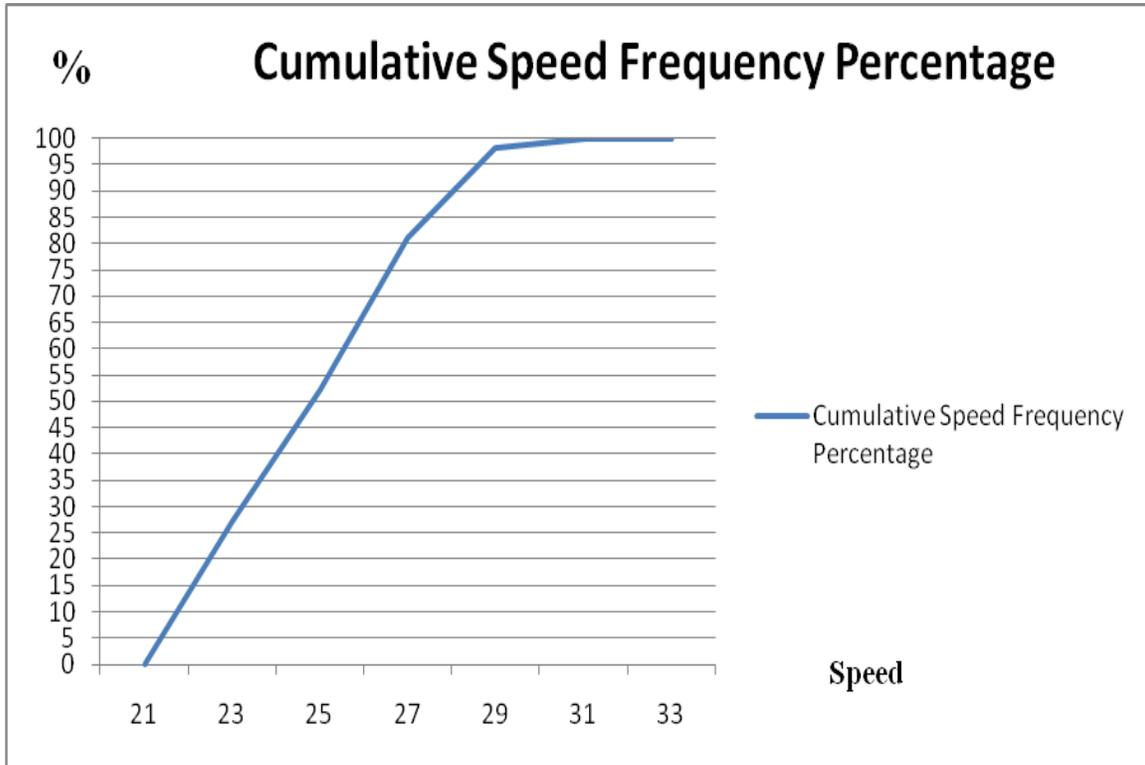


Figure 58 Cumulative Frequency of Speed Distribution of NB of Capitol St & State St

3. Pascagoula St & State St (SB)

The through traffic speed data were used to determine the FFS. The 85th percentile speed of through traffic was about 37 mph which was used as FFS. The cumulative speed frequency was shown in Figure 59.

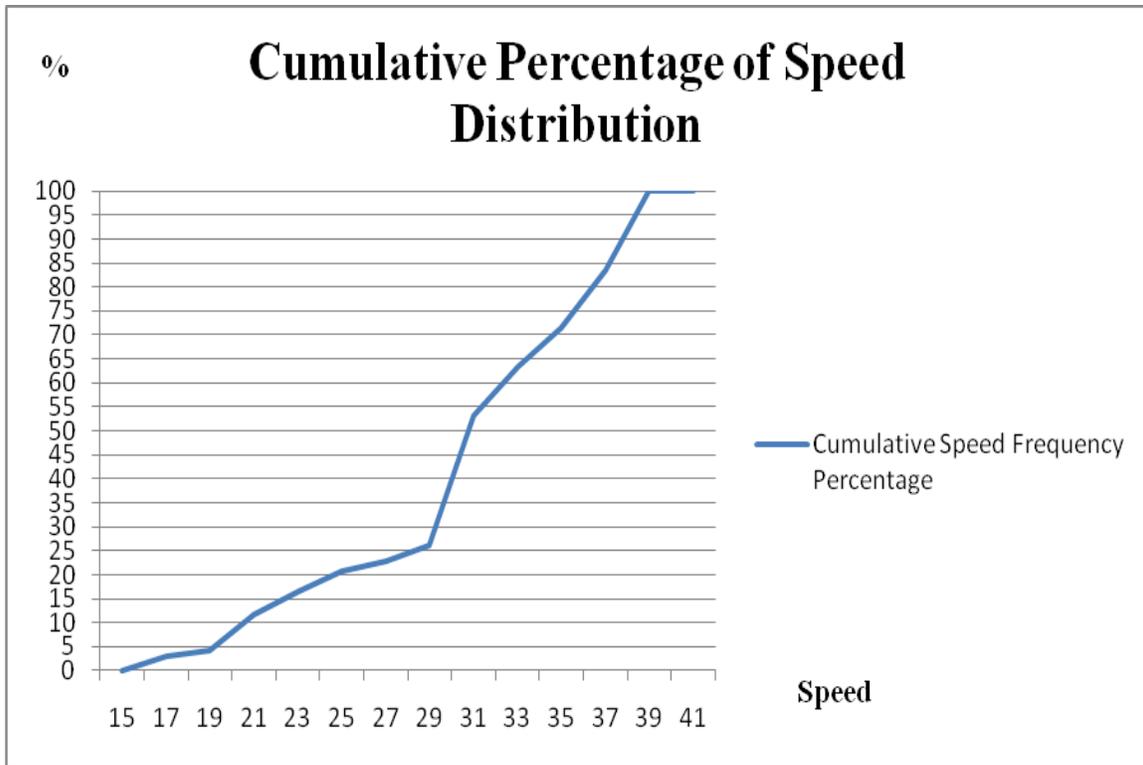


Figure 59 Cumulative Speed Frequency Distribution of SB of Capitol St & State St

4. Pearl St & State St (NB)

Traffic data at this intersection were collected by Radar. Through traffic speed data was used to estimate 85th percentile speed. Figure 60 showed cumulative percentage of NB speed distribution. 85th percentile speed was rounded to 30 mph and used as FFS of this approach.

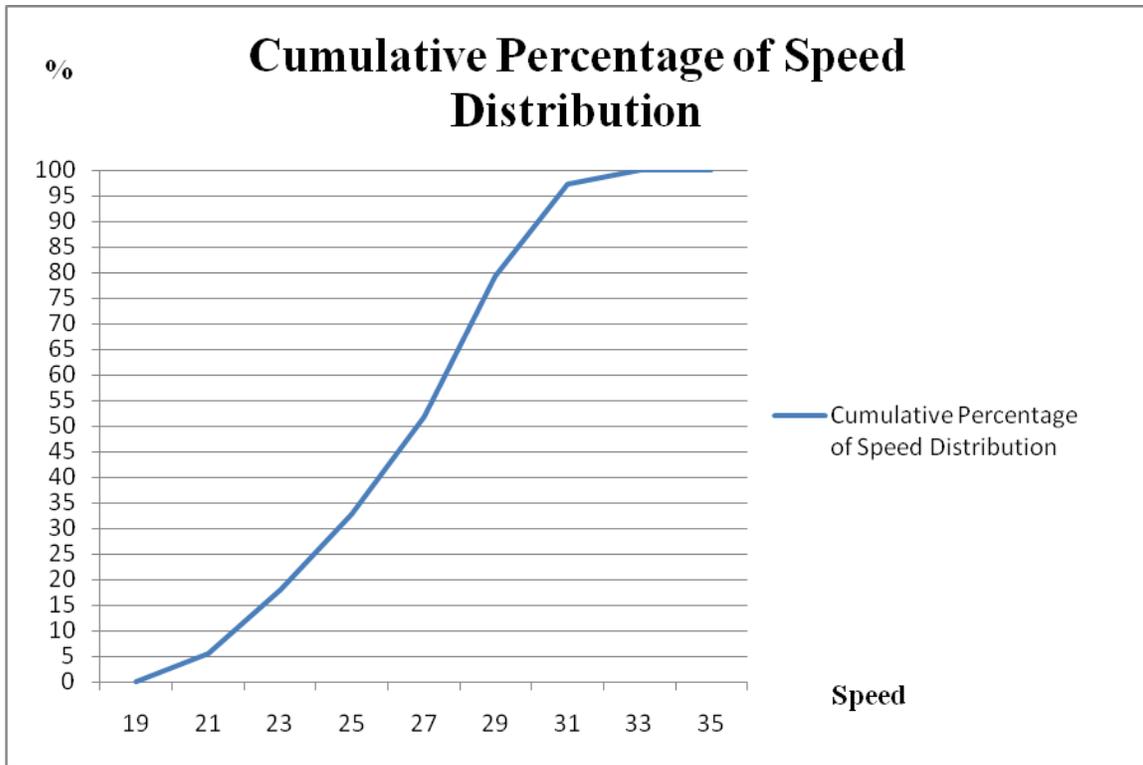


Figure 60 Cumulative Percentage of Speed Distribution of NB of Pearl St & State St

5. Briarwood Dr& State St

Traffic data at Briarwood Dr& State St intersection were collected by Radar and NC 200 for all approaches. Speed data of all approaches were available. Traffic speed data from each approach were analyzed for FFS.

a. NB

Through traffic speed data were used to estimate FFS. The NB through traffic was recorded by Radar. Figure 61 showed cumulative percentage of speed distributions. The NB 85th percentile speed of through traffic was about 28 mph. However, compared to FFS of SB, NB's 85th percentile speed was much lower. In reality, FFS of NB and SB of an arterial section is almost the same. Then, Google Maps Street View was used to obtain the speed limit of State St of Briarwood Dr intersection. Speed limits of State St close to State St & Cedars of Lebanon Rd and Beasley Rd & State St are 40 mph. Briarwood Drive & State St intersection is located between these two intersections. A reasonable assumption, that FFS of State St near Briarwood Drive & State St is also 40 mph, was made. 40 mph was used as FFS of NB instead of using 85th percentile speed of NB.

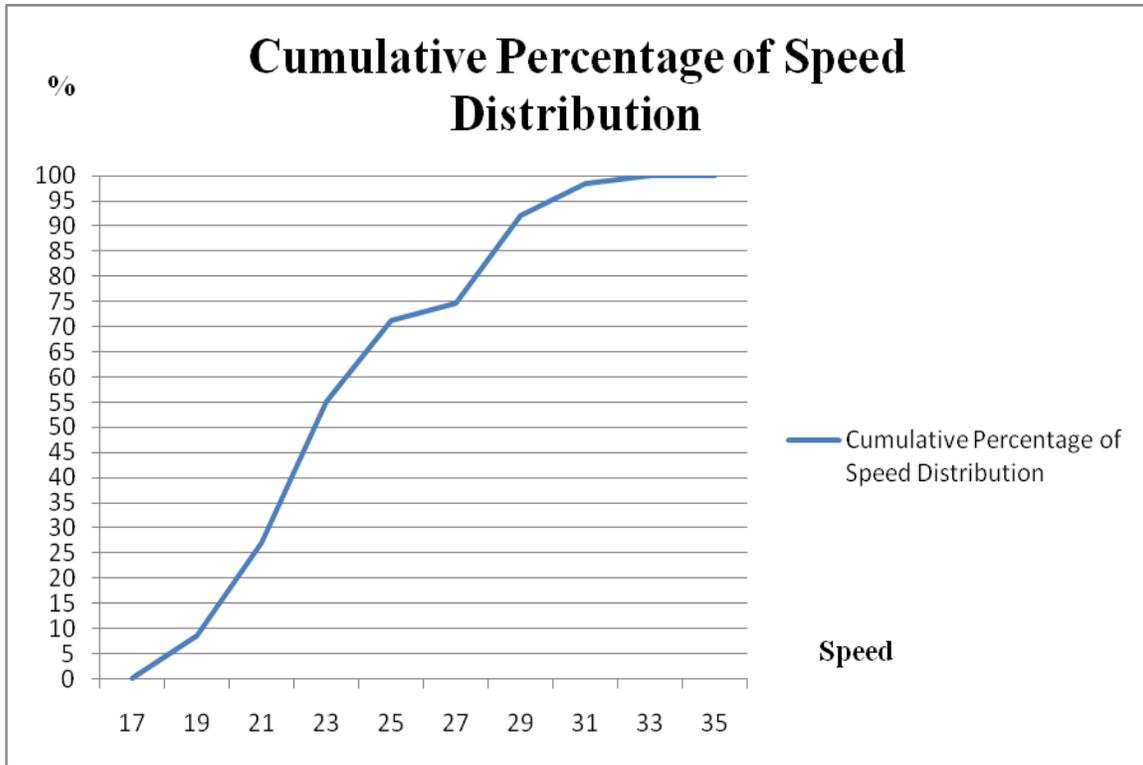


Figure 61 Cumulative Percentage of Speed Distribution of NB of Briarwood Dr& State St

b. SB

SB speed data were recorded by NC 200. The 85th percentile speed of each lane in SB can be generated by HDM. As discussed before, only through lanes' speed data were used to determine the 85th percentile speed. Table 108 showed the 85th percentile speed in each through lane. An average of the 85th percentile speed is 47.51 mph. 48 mph is used as the FFS of SB.

Table 108 85th Percentile Speed of SB through Lanes of Briarwood Drive & State St

	Through lane 1	Through lane 2
85 th percentile speed (mph)	47.81	47.21

c. WB

Traffic data was only available for WB right turn. The 85th percentile speed of right turn traffic is 25.53 mph. The speed is rounded up to 26 mph. Google Map Street View (<https://maps.google.com/>) was used to find the speed limit of Briarwood Dr which is 40 mph. Therefore, the WB FFS was assumed to be 40 mph which equals to the speed limit.

6. Old Canton Rd & State St

Traffic at Old Canton Rd & State St were monitored by Radar or NC 200 for all approaches. And, speed data of all approaches are available for analyzing FFS on each approach.

a. NB

NB's through traffic was recorded by Radar, while NB right turn data was collected by NC 200. Speed data of through and right turn can be used to determine FFS. The NB layout was shown in Figure 62. It can be seen that both NB through and right turn traffic may not need to slow down or just slow down a little to pass the intersection.

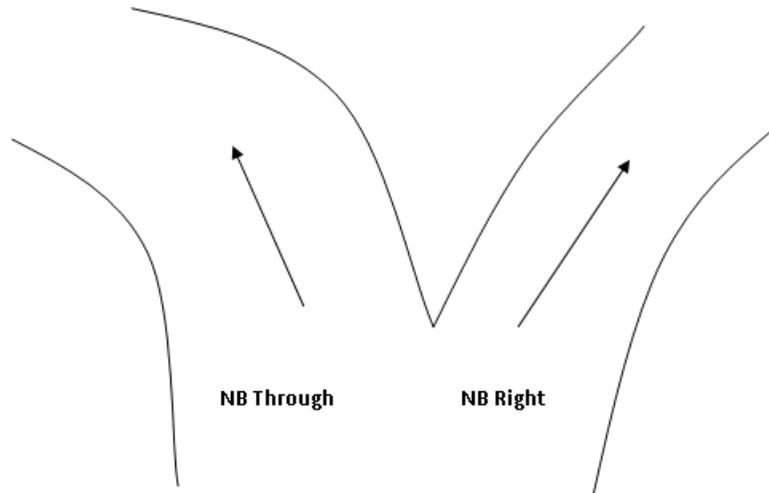


Figure 62 Layout of NB on Old Canton Rd & State St

The average 85th percentile speed of through and right turn traffic was used as the NB's FFS. NB through traffic were recorded by Radar. The 85th percentile speed of through traffic was displayed in Figure 63. 30.5 mph was the 85th percentile speed of NB Through traffic.

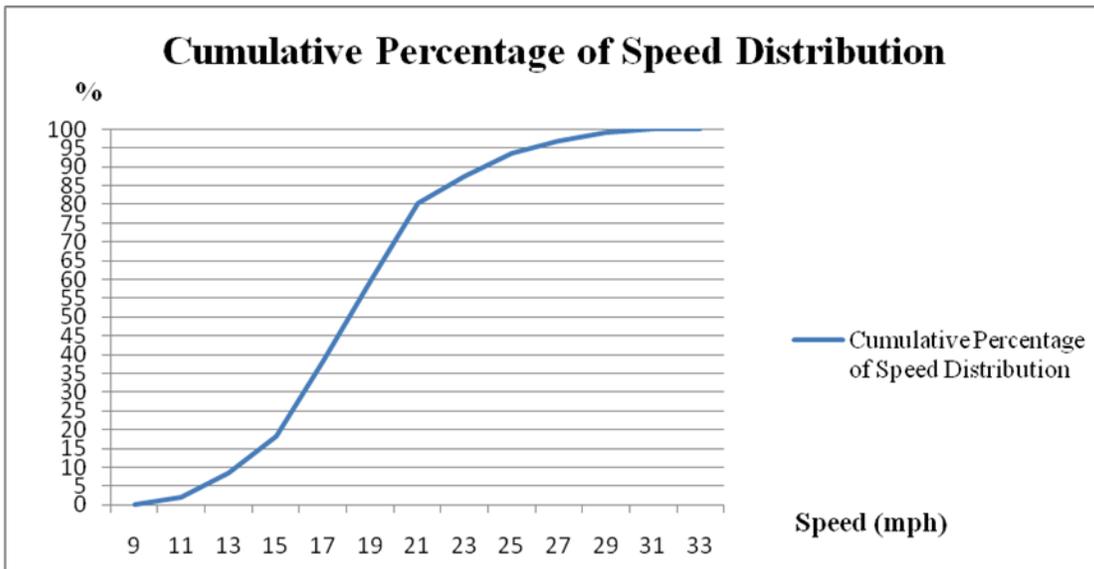


Figure 63 Cumulative Percentage of Speed Distribution of NB Through Traffic

NB right turn traffic were recorded by NC200. The 85th percentile speed of two NB right turn lanes were listed in Table 109.

Table 109 85th Percentile Speed of NB Right Turn Lanes of Old Canton Rd & State St

Lane	Right Turn Lane 1	Right Turn lane 2	Average
85 th percentile speed (mph)	32.51	34.96	33.7

The average value of the NB 85th through speed and the 85th right turn speed is 32 mph. It was used as the NB's FFS.

b. SB

SB traffic were monitored by NC 200. The 85th percentile speed of SB was calculated by HDM as 30.38 mph. 30 mph is used as SB's FFS.

c. WB

WB traffic data was collected by Radar. The cumulative percentages of speed distribution of WB was shown in Figure 64. The 85th percentile speed of WB was 24 mph which was considered as the WB's FFS.

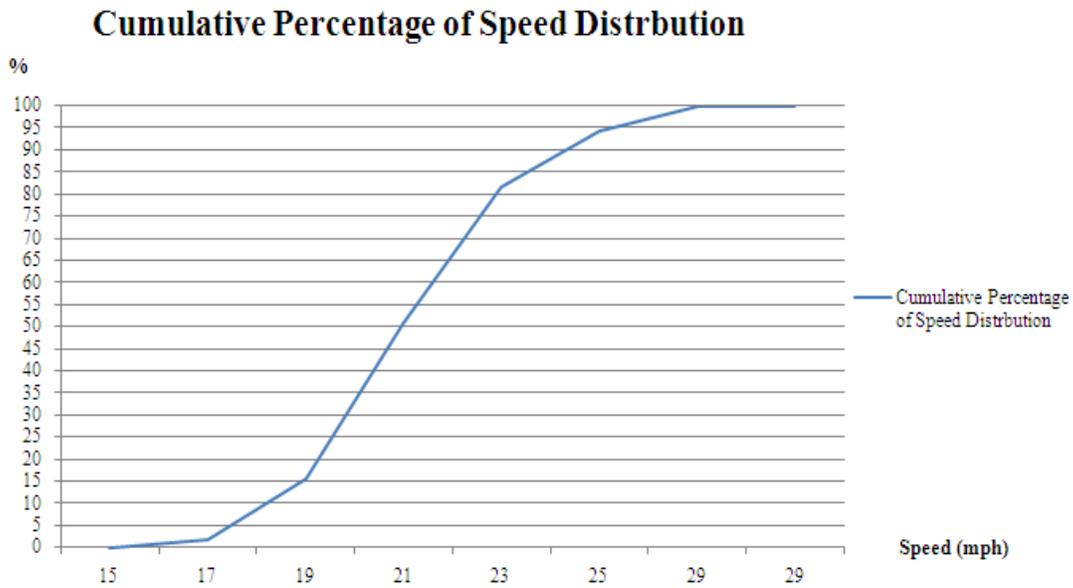


Figure 64 Cumulative Percentage of Speed Distribution of WB of Old Canton Rd & State St