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Improving Traffic Signal System Planning, Design, and Management with Big-Data-Enhanced ATSPM System

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16. Abstract

In this project, an ATSPM-in-the-loop simulation framework is developed to introduce the data-driven ATSPM framework into traffic signal planning and design. Literature review on ATSPM and Best practice survey are conducted first. Then the research team explores and documents how to set up two real-world open-source ATSPM systems: Utah-DOT ATSPM system and big-data-empowered UTA-In-Motion, an add-on module for ATSPM systems. Multiple integrating software tools are developed to parse simulation outputs into real-world ATSPM signal logs and WGS84 trajectories as the data feeds of ATSPM systems. Two case studies are conducted to demonstrate how this new framework can be used for traffic signal design projects. Last but not least, the research team presents their research outcome during state and national conference and webinars as technology transfer activities.

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IMPROVING TRAFFIC SIGNAL SYSTEM PLANNING, DESIGN, AND MANAGEMENT WITH BIG-DATA-ENHANCED ATSPM SYSTEM

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DISCLAIMER

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AADT - Annual Average Daily Traffic
AoG - Arrivals on Green
AoR - Arrivals on Red
AOC - Arrival-On-Coordination
AT - Arrival Type
ATMS - Advanced Transportation Management System
ATSPM - Automated Traffic Signal Performance Measures
AVAST - *AI-empowered Video Analytics for Smart Transportation*
BA - Band Attainability
CBD - Central Business District
CCTV - Closed-Circuit Television
CP - Critical Point
CV - Connected Vehicle
DOT - Department of Transportation
EDC - Everyday Count
ETC - Electronic Toll Collection
EHV - Equivalent Hourly Volume
EULA - End-User License Agreement
FDOT - Florida Department of Transportation
FHWA - Federal Highway Administration
GOR - Green Occupancy Ratio
GT – Green Time
HCM - Highway Capacity Manual
INDOT - Indiana Department of Transportation
ITS - Intelligent Transportation Systems
LOS - Level of Service
MnDOT - Minnesota Department of Transportation
MOE - Measures of Effectiveness
MPOs - Metropolitan Planning Organizations
NB - Northbound
NCTCOG - North Central Texas Council of Government
NEMA - National Electrical Manufacturers Association
NJDOT - New Jersey Department of Transportation
OSI - Oversaturation Severity Index
PCD - Purdue Coordination Diagram
PHF - Peak Hour Factor
PMRG - Performance Measure Report Generator
POG - Percent of Green

POR - Percent on Red
PR - Platoon Ratio
RBC - Ring-Barrier controller
ROR - Red Occupancy Ratio
SB - Southbound
SILS - Software-In-The-Loop
SPMs - Signal Performance Measures
SMART-SIGNAL - Systematic Monitoring of Arterial Road Traffic Signals
SMS - Space Mean Speed
TD – Total Delay
TM – Technical Memo
TMC - Traffic Message Channel
TSD - Time-Space Diagram
TSP - Transit Signal Priority
TxDOT - Texas Department of Transportation
UDA - User-Defined Attributes
UDOT - Utah DOT
UTAIM - UTA-In-Motion
v/c - Volume-to-Capacity

EXECUTIVE SUMMARY

Automated Traffic Signal Performance Measures or ATSPM represent a state-of-the-art traffic signal management solution. After over 20 years of collaboration among federal agencies, state agencies, academia, and industry, the ATSPM system is increasingly being accepted and adopted by agencies. While the ATSPM deployment is scaling up, new challenges are surfacing. A main challenge is the increasing gap between traffic signal design and traffic signal operations. Traffic signal planning and design still follow the traditional approach instead of the new ATSPM approach due to a lack of the needed data. Once the new traffic signal timings are deployed in the field, they will receive much more extensive evaluation using the ATSPM method. Another challenge is that deploying the ATSPM system takes lots of resources even though the software may be free. This situation results in limited access to the ATSPM system among the stakeholders. Other challenges include the high demand for detectors, operational and maintenance complexities, etc. These issues will likely become hurdles for agencies to scale up their ATSPM deployment. This research aims to address the above issues to facilitate TxDOT's stage-wide ATSPM deployment.

The research team from the University of Texas at Arlington conducted multiple tasks to fulfill the research goals. A literature review and a best practice survey were first conducted. Among over 100 responses, most expressed an interest in ATSPM. Many agencies have either deployed or are considering deploying the ATSPM systems. Second, a traffic simulation model was developed to generate the travel demand for future scenarios. A popular traffic simulation software, PTV VISSIM, was adopted to serve this goal. A TxDOT freight corridor, Cooper Street in Arlington (TX) was modeled including 14 intersections. Multiple sources of data were collected, including traffic signal timing data, turning counts, and connected vehicle trajectory data. All these data have been used for traffic modeling, calibration, and validation.

The major effort is developing the software tools to transform the output of traffic simulation to the data format recognizable by the real-world ATSPM system(s). All software tools will be published to the public for free.

Using the developed VISSIM models, installed ATSPM systems, and the developed software, the research team conducted two case studies to demonstrate how to use this new solution for traffic signal projects, from standard traffic signal project to complex traffic signal project including preemptions and priorities

While developing and maturing this new solution, the research team also performed technology transfer activities. A presentation was made first during the TexITE meeting in College Station in the spring of 2024. A more comprehensive presentation was made to introduce this new solution in the ATSPM monthly webinar hosted by FHWA in June 2024. This new solution received broad interest from the audience.

CHAPTER 1: INTRODUCTION

1.1. OVERVIEW OF AUTOMATED TRAFFIC SIGNAL PERFORMANCE MEASURES (ATSPM)

Automated traffic signal performance measures or ATSPM are a set of data analytics tools and approaches that collect and convert high-resolution traffic controller data into actionable performance measures automatically. In the United States, there are over 330,000 traffic signals in operation, and highway agencies typically retime these signals every three to five years at a cost of about $4,500 per intersection (1). Citizen complaints are the primary measure of performance for many existing signals. Because most intersections do not collect performance data continuously, intersection performance is simulated using software models based on periodic and manual traffic data collection, which adds cost and time to the signal retiming process.

Traditional retiming projects require agency professionals and consultants to perform an ad-hoc comparison before and after travel-time data to demonstrate the effectiveness of optimization efforts. In most cases, there is no regular performance monitoring in traffic operations, and agencies have to rely on citizen complaints to detect maintenance or operational deficiencies. Lack of active performance management can jeopardize safety and efficiency while also contributing to traffic congestion. Furthermore, traditional traffic signal retiming programs are project-based and follow a periodic schedule. Those situations such as residents' complaints and/or reaching the retiming schedule (e.g., every 3 to 5 years) will justify and start the retiming process. After the needed budget is secured, it will move to the bidding and design phase. The designers will collect travel time samples and turning movement data and reconstruct the current signal operations within a simulation environment. If the current signal performance is not satisfied, certain optimization techniques are applied to improve. The updated signal timings will later be fine-tuned and implemented in the field through observations. Approval of the new timing plans by agencies will conclude this effort and the updated timing plans will hold for another few years.

Many traffic signal systems have recently been equipped with performance monitoring modules. Selected performance metrics are continuously generated and used to monitor the effectiveness of traffic control strategies and tactics and evaluate if they meet agencies' goals. Regulating activities can be activated due to poor performance regardless of residents' calls or retiming schedule.

1.2. ADVANTAGES OF ATSPM

The Federal Highway Administration (FHWA) has been supporting and promoting the use of ATSPM as a way to improve traditional traffic signal retiming processes through continuous performance monitoring. ATSPM deployment involves updating traffic signal controllers with high-resolution data-logging capability and using novel data analytics and visualization to identify and correct deficiencies in traffic signal maintenance and operations. This can greatly facilitate agencies to achieve their goals of safety, livability, and mobility.

ATSPM is cost-effective because it can be applied to a wide range of signalized intersections and exploit existing control infrastructure. It also has the potential to cross-validate other emerging technologies and operational strategies for urban traffic management, such as adaptive signal control and/or connected vehicle applications. Using ATSPM, signal retiming efforts can be based on actual performance data, which can significantly reduce the time, cost, and effort involved in providing effective traffic signal operations.

1.3. THE STATE OF PRACTICE ON REAL-TIME SIGNAL PERFORMANCE MEASUREMENT

Traditional traffic signal timing practices often rely on traffic signal optimization software. Modeling the current signal operations can be both costly and labor-intensive as they require extensive data collection for various input parameters such as volumes, speeds, and roadway characteristics. In addition, the timing optimization based on simulation typically does not take into account special traffic events such as traffic signal priority (TSP) and preemptions, resulting in ineffective control strategies and even safety hazards for road users.

Back in 2005, the Indiana Department of Transportation (INDOT) researched to develop novel traffic signal system performance measures using logged, time-stamped vehicle detectors and traffic signal controller events, also known as high-resolution traffic signal data (2). These new signal performance measures (SPMs) provide policymakers with an effective tool for proactively managing traffic signals and corridors with a higher degree of accuracy. Some examples of real-time signal performance metrics include:

- Capacity performance measures: These measures are used to monitor capacity utilization at signalized intersections. Examples include cycle length, volume, capacity utilization, green time and capacity allocation, green occupancy ratio, red occupancy ratio, phase termination, and degree of intersection saturation. These measures can be evaluated using either advance detectors or stop bar detectors.
- Progression performance measures: These measures describe the quality of progression at a signalized intersection in terms of delay and queue length. Examples include delay estimates from measured arrival profiles, Purdue Coordination Diagrams (PCDs), flow profiles, and maximum queue length from shockwave estimation.
- Multimodal performance measures: transportation modes other than vehicles, such as pedestrians, transit, and rail traffic have their unique performance considerations under traffic signal systems. Multimodal performance measures are developed to evaluate the performance of these modes, which are incorporated into traffic signals through pedestrian phasing, transit priority, and rail preemption.

- Maintenance performance measures: It is essential to maintain the functionality of traffic control equipment, including detection devices and communication equipment to support the signal performance system. Relevant maintenance performance measures include communication quality and detector failures.
- Advanced performance measures: These measures include those that do not fall into the other categories, such as safety performance measures that describe the safety performance of an intersection in terms of intersection-related crashes and conflict points.

1.4. OVERVIEW OF EXISTING ATSPM SYSTEMS

1.4.1. ATSPM module in Q-free's Maxview Suite

The Maxview Suite developed by Q-free Inc. is an advanced transportation management system (ATMS) and targets traffic management on a full spectrum of transportation infrastructure. It includes a suite of software tools designed for traffic control and transportation management. It helps transportation agencies and companies monitor and optimize their transportation systems, including roads, highways, bridges, tunnels, and public transport systems. The Maxview suite includes customizable and configurable modules and applications to meet the specific needs of each customer. Some key features and capabilities of Maxview include:

- *Advanced Transportation System Performance Monitoring*: This module allows transportation agencies to collect, analyze, and report data from sources such as traffic sensors, cameras, and other monitoring systems. It tracks traffic flow, identifies bottlenecks and congestion, and measures the performance of transportation systems. The ATSPM contains not only those performance measures defined by the INDOT but also a few new performance measures developed by the company.
- *Traffic Management*: Maxview includes tools for traffic management, such as traffic control centers, incident management systems, and traffic signal optimization systems. These tools manage traffic flow, reduce congestion, and improve road and highway safety.
- *Public Transport Management*: Maxview includes tools for managing public transport systems, including real-time passenger information systems, fare collection systems, and passenger counting systems. These tools improve the efficiency and reliability of public transport systems and provide passengers with accurate, up-to-date information.
- *Vehicle Detection and Classification*: Maxview includes tools for detecting and classifying vehicles, such as license plate recognition systems, vehicle classification systems, and automatic vehicle identification systems. These tools improve the efficiency and accuracy of transportation systems and support applications such as tolling and access control.
- *Road Pricing and Electronic Toll Collection*: Maxview includes tools for implementing road pricing and electronic toll collection systems, such as electronic toll collection (ETC) systems and intelligent transportation systems (ITS). These tools improve the efficiency and fairness of road pricing systems and support applications such as congestion charging and high-occupancy vehicle lanes.

1.4.2. Iteris ATSPM system

Iteris Cloud-based ATSPM software is a software application developed by Iteris Inc.® that helps transportation agencies monitor and optimize the performance of their transportation systems. It is hosted on Iteris' cloud platform and can be accessed through a web browser, allowing agencies to use the ATSPM software without installing and maintaining it on their servers. The software collects and analyzes data from various sources, such as traffic sensors and cameras, to track traffic flow and identify bottlenecks and congestion. It can also be used to optimize the operation of signalized intersections and other traffic control devices.

1.4.3. INRIX ATSPM system

The INRIX ATSPM system is a software application developed by INRIX®, a company that specializes in the development of transportation analytics and networked car services. The INRIX ATSPM system is designed to help transportation agencies and departments monitor and optimize the performance of their transportation systems based on probe vehicles' information collected by the company. Compared with other ATSPM systems, the INRIX ATSPM replies on vehicle trajectories, not on traffic high-resolution signal data. It is based on a cloud-based platform and includes various tools and functions for collecting, analyzing, and reporting data from various sources, such as traffic sensors, cameras, and other monitoring systems. It also allows agencies to track traffic flow, identify bottlenecks and congestion, and measure the system's performance. The software can also be used to monitor and optimize the operation of signalized intersections, tolling systems, and other traffic control devices. Using the INRIX ATSPM system is based on annual fees per intersection.

In addition, the INRIX ATSPM system includes a range of visualization and reporting tools that allow agencies to present their data in an easily understandable format, and to share the data with stakeholders and the public. Overall, the INRIX ATSPM system is designed to provide a comprehensive and powerful solution for transportation agencies and departments to monitor and optimize the performance of their transportation systems.

1.4.4. Kimley-Horn Traction Metrics System

Kimley-Horn Traction Metrics is a software package developed by an engineering consulting firm, Kimley-Horn®, to help transportation agencies monitor and optimize the performance of their transportation systems. The cloud-based Traction Metrics software includes tools for collecting, analyzing, and reporting data from various sources such as traffic sensors and cameras. It allows agencies to track traffic flow, identify bottlenecks and congestion, and measure the performance of their systems.

1.4.5. Utah DOT's ATSPM System

Utah DOT's (UDOT) ATSPM system (*3*) was developed in partnership with Purdue University, the FHWA, and the Transportation Pooled Fund Program. UDOT's ATSPM system includes a suite of data visualization reports that can be used to evaluate the quality of traffic progression along corridors and identify any unused green time that can be allocated to other intersection movements. Currently, UDOT is collecting ATSPM data at 96% of its 1,223 traffic signals, intending to connect all signals statewide to contribute data to the ATSPM system. UDOT's traffic team decided to open all the source codes of their ATSPM system to the whole traffic signal community. This decision greatly reshaped the development of the ATSPM systems and business models.

In the UDOT's ATSPM system, users are allowed to generate different charts for various performance metrics, including approach delays, approach volumes, arrivals on red, coordination diagram, Purdue split failure diagram, pedestrian delay diagram, phase termination diagram, preemption details, split monitor diagram, and turning movement counts. Since implementing its ATSPM system, UDOT's Traffic and Safety Division has allocated approximately $7 million annually for new traffic signals. The cost to implement a small ATSPM system with 50 signals is estimated at $20,000, while a larger system with 1,000 signals would cost $230,000. The investment and deployment of UDOT's ATSPM system have reduced the maintenance budget to $3,500 per signal in 2017 and resulted in a significant decrease in public complaints and requests for traffic signal retiming.

1.4.6. UTA-In-Motion (UTAIM) ATSPM system

UTA-In-Motion (UTAIM) is a light automated traffic signal performance monitoring system with unique features developed by a research team led by Dr. Taylor Li at the University of Texas at Arlington. It aims to provide future-proof arterial traffic performance monitoring. Compared with other ATSPM systems, the UTAIM system contains several unique performance measures based on both high-resolution traffic signal events and novel traffic big data. Other than the core data source, high-resolution traffic signal events, UTAIM also incorporates various internet-based traffic big data sources, such as real-time weather and air quality, real-time travel time and speed (retrieved from the APIs of Google Maps© or Waze©), commercial connected vehicle data (via in-vehicle cellular modems) and special probe vehicles (regular vehicles equipped with a special smartphone app developed by the team). Based on various data sources, UTAIM provides users with several unique diagrams, including a multimodal vehicle arrival Diagram integrated with weather data, hourly Level-of-Service of arterial traffic, pedestrian calls, preemptions, and priority event display; a time-of-day capacity and volume analysis; a tracking-based pedestrian performance diagram; and a TSD across multiple intersections for green band analysis. UTAIM is an effective tool for designing, optimizing, and evaluating arterial traffic management. It is positioned as an add-on ATSPM module to other ATSPM systems, too. UTAIM can couple with the databases of UDOT's ATSPM and Maxview's ATSPM at this time.

CHAPTER 2: LITERATURE REVIEW

The purpose of this literature review is to provide a quick but sufficient summary for the TxDOT staff. Hence, the literature is reviewed in the format of "Reference: Summary."

Lattimer, C. R., & America, A. N. (2020). Automated traffic signals performance measures (No. FWHA-HOP-20-002). United States. Federal Highway Administration (4)

This report discusses the technical support provided by the FHWA to help states implement their Everyday Count (EDC) objectives. It introduces the concept of an Advanced Transportation System Performance Monitoring system, which was used by 31 states across the country in 2018 to varying degrees, up from 11 states in 2017. The report presents user cases that demonstrate how to use a "watchdog" report to identify abnormalities within high-resolution traffic signal database logs, such as high force-off and max-out events, low detector counts, missing data records, and high stuck pedestrian counts, and shows how to use PCDs to adjust offsets and identify split failures. The ATSPM system enables agencies to transition from a traditional signal retiming project cycle based on trigger events to a modern project cycle based on agency objectives.

Jin, P. J., Zhang, T., Brennan Jr, T. M., & Jalayer, M. (2019). Real-Time Signal Performance Measurement (RT-SPM) (No. FHWA NJ-2019-002). United States. New Jersey Department of Transportation (5)

This report, sponsored by the New Jersey Department of Transportation (NJDOT), aims to use existing field data and equipment to establish SPMs for real-time monitoring and to investigate the additional data and equipment needed to generate real-time traffic signal reports automatically. The key contribution of this project is the development of a custom program to translate existing adaptive signal systems, SCATS and InSync, history records into event code-based Indiana Traffic Signal High-Resolution Data Logger Enumerations, which can later be used to generate Advanced Transportation System Performance Monitoring reports.

Chamberlin, R., & Fayyaz, K. (2019). Using ATSPM Data for Traffic Data Analytics (No. UT-19.22). Utah. Dept. of Transportation. Research Division.(6)

This report summarizes the efforts and findings of the Utah Department of Traffic's estimated annual average daily traffic (AADT) on roadways based on 6,000 short-duration traffic counts over three years, as required by the FHWA. The research data was collected using Wavetronix Advance and Matrix detectors and was mapped with hourly count data from the Advanced Transportation System Performance Monitoring system in 2017. The accuracy of the detectors was measured using linear regression with and without adjustment factors. The results showed that the

Matrix detector was more accurate (88%) in estimating AADT, hourly counts, and seasonal factors (97%).

Day, C. M., Smaglik, E. J., Bullock, D. M., & Sturdevant, J. R. (2008). Real-Time Arterial Traffic Signal Performance Measures (No. FHWA/IN/JTRP-2008/9). Indiana Department of Transportation (INDOT), Indianapolis, Indiana.(7)

This report focuses on performance measures defined by the Highway Capacity Manual (HCM) that can be obtained on a cycle-by-cycle basis using an automated traffic signal controller. The report first reviews fundamental concepts of traffic operations, such as the basic components of a traffic signal system, vehicle detection, actuated signal operation, and coordination. It then discusses how traffic data is collected and converted into performance measures and evaluates the effectiveness of these performance measures in two comparative situations: the effects of actuating a portion of the coordinated phases and traffic signal retiming on coordinated arterial intersections. The performance measures discussed in this report are divided into three categories. The first category is the state of the intersection, including cycle length, green duration, and volume. The second category is based on the concept of intersection capacity and includes service flow rate, estimated capacity, observed capacity, volume-to-capacity (v/c) ratio, number of split failures, and critical v/c ratio. The third category is a set of performance metrics that quantify the intersection's performance in coordination with vehicle progression, including the percentage of arrivals on the green, arrival type, and platoon profile.

Day, C. M., D. M. Bullock, H. Li, S. M. Remias, A. M. Hainen, R. S. Freije, A. L. Stevens, J. R. Sturdevant, and T. M. Brennan. (2014). Performance Measures for Traffic Signal Systems: An Outcome-Oriented Approach. Purdue University, West Lafayette, Indiana. (2)

This report synthesizes research on traffic SPMs based on high-resolution controller event data, presenting a methodology for evaluating the performance of traffic signal systems. It discusses methods for collecting and managing signal event data and the necessary infrastructure to do so and presents a portfolio of performance measures for both vehicle and non-vehicle modes, including measures for system maintenance asset management, and signal operations. The report discusses various types of performance measures for traffic signal systems, including location control and system control measures. Location control measures, also known as local control measures, are based on capacity performance measures such as cycle length, green time and capacity allocation, volume and capacity utilization, green occupancy ratio and red occupancy ratio, and degree of intersection saturation. System control measures, also known as progression performance measures, include delay and quality of progression, delay estimates from measured arrival profiles, PCDs, flow profiles, platoon formations and dispersion, and maximum queue length.

The methodology for evaluating performance includes three tracks: computing vehicle measures of effectiveness (MOE), computing estimated delays, and computing non-vehicle MOEs. Vehicle MOEs consist of both capacity performance measures and progression performance measures. Non-vehicle performance measures investigated in this report include pedestrian, railroad, and transit measures, such as pedestrian phasing, railroad pre-emption, and transit priority. In addition to these performance measures, the report also discusses measures for equipment maintenance and outcome assessment. Equipment maintenance measures are used to ensure the proper functioning of the signal system, while outcome assessment measures use travel time data to measure the reliability of the transportation system.

Grossman, J., & Bullock, D. M. (2013). Performance Measures for Local Agency Traffic Signals. (No. INLTAP-TR-4-2013). Purdue University, West Lafayette, Indiana. (8)

This research project develops a specification language for deploying performance measures in Indiana and West Virginia and evaluates the feasibility of implementing adaptive traffic signal control within a short timeframe of 12 to 18 months. The results show that detection placement, detection quality, and agency commitment to supporting the operations of these systems significantly affect their applicability and effectiveness. The SPMs described in this report include cycle length, equivalent hourly flow rate, green time plot, volume-to-capacity ratio, split failures, PCD, and percentage of phases with pedestrians. The report also emphasizes the importance of high-quality vehicle detection for SPMs and recommends retrofitting existing video detection systems with thermal cameras to improve detection quality.

Stephen Remias, Jonathan Waddell, Matt Klawon, Ken Yang (2018). Signal Performance Measures Pilot Implementation (No. SPR-1681). Michigan Department of Transportation, Lansing, Michigan. (9)

The Michigan Department of Transportation implemented Advanced Transportation System Performance Monitoring on two corridors to evaluate and monitor their performance. A cost-benefit analysis was conducted to assess the potential costs and benefits associated with the deployed ATSPM system. The pilot ATSPM project showed significant benefits, including reductions in travel time. The benefit-cost ratio was estimated to be 25 to 1, considering the savings in travel time, safety benefits, maintenance benefits, continuous operational benefits, and initial optimization benefits.

Day, C. M., Bullock, D. M., Li, H., Lavrenz, S. M., Smith, W. B., & Sturdevant, J. R. (2016). Integrating Traffic Signal Performance Measures into Agency Business Processes. Purdue University, West Lafayette, Indiana. (10)

This report is intended for transportation agencies to assist in the development and implementation of an active traffic management program using performance measures to manage traffic signal systems. It provides an overview of current practices and opportunities for improvement, as well as an overview of business processes in signal system management. The report also presents various performance measures for evaluating the quality of progression, local control, and communication and detection systems. These measures include those for assessing detector health and communication status in traffic signal systems, as well as measures for evaluating local signal control and system control. For single intersections, the report includes an evaluation of capacity allocation, safety performance (such as red light running), pedestrian service, and diagnostic analysis of preemption and advanced control settings. For system performance measures, the report covers evaluations of capacity allocation, pedestrian service, safety performance, and diagnostic analysis of preemption and advanced control settings.

The report also discusses the impact of detector health and communication status on traffic control systems, including communication failures, delays for drivers, vehicle progressions, and benefit/cost analysis. For single intersections, performance measures include cycle length, duration of green and reason for termination, capacity utilization metrics, estimated delay, red light running, pedestrian utilization, and special operational diagnostics. For assessing system control, the report focuses on traffic progression and includes visualizations of traffic events, as well as quantitative performance measures that can be aggregated. These measures include Arrivals on green (AoG)/percent on green (POG), Arrivals on Red (AoR)/percent on red (POR), platoon ratio, arrival type, and estimated delay.

Liu, H. X., Ma, W., Wu, X., & Hu, H. (2008). Development of a real-time arterial performance monitoring system using traffic data available from existing signal systems (No. MN/RC 2009-01). Minnesota Department of Transportation, St. Paul, Minnesota(11)

The Minnesota Department of Transportation (MnDOT) supported a project to design a system called SMART-SIGNAL (Systematic Monitoring of Arterial Road Traffic Signals), for collecting high-resolution traffic signal data and developing performance measurements. This system can collect and store event-based traffic signal data at multiple intersections in real time, allowing for the development of performance measures such as the number of stops, queue length, and travel time. At each intersection, the SMART-SIGNAL system captures both vehicle actuation events and signal phase change events. The report presents different performance measures for intersection-level and arterial-level analysis. Intersection-level measures include arrival type, cyclic volume, occupancy profile, delay, level of service (LOS), queue size, and queue length. Arterial-level measures include travel time, delay, number of stops, stop time, and vehicle probe trajectory.

Balke, K. N., Charara, H. A., & Parker, R. (2005). Development of a traffic signal performance measurement system (TSPMS) (No. FHWA/TX-05/0-4422-2). Texas Transportation Institute, Texas A & M University System.(12)

Texas Department of Transportation (TxDOT) sponsored a project to evaluate various intersection-related performance measures and develop a prototype system for automatically collecting these measures in the field. The system was tested in two locations with different operating characteristics to assess its capability to produce the desired performance measures. As part of this project, TxDOT developed a performance measure report generator (PMRG) – a log file analysis software that collects events such as phase status, phase on, ring status, and vehicle detections. The PMRG allows users to select daily log files through a graphical interface and then generates a performance measurement report based on the selected log files. The report includes metrics such as cycle time, time to service, queue service time, duration of green, yellow, all-red, and red intervals for each phase, number of vehicles entering the intersection during each interval, yellow and all-red violation rates, and phase failure rate. The PMRG was evaluated using these same performance metrics. The results of the evaluation allowed TxDOT to assess the effectiveness of the PMRG in accurately capturing and reporting intersection performance data.

Balke, K. N., & Herrick, C. (2004). Potential Measures of Assessing Signal Timing Performance Using Existing Technologies (No. FHWA/TX-04/0-4422-1). Texas Transportation Institute, Texas A&M University System.(13)

TxDOT conducted another study to evaluate potential measures for assessing signal timing performance at isolated intersections using existing detection technologies. The report presents the results of interviews and an examination of existing controllers and detection technologies for collecting SPMs. TxDOT used volume data, specifically turning movement volume counts, to develop and evaluate signal timing plans. The report assesses the capabilities of existing traffic signal and detection technologies, including the Eagle EPAC Traffic Signal Controller, the Autoscope Solo System, and the ORACLE/2 Inductive Loop System. Potential performance measures discussed in the report include measures of reliability, efficacy, and safety. The measures of efficacy are based on average cycle time, the average duration of each phase, the average time required to service a call, and the average proportion of green used to service queue.

Gordon, R. L. (2010). Traffic signal retiming practices in the United States. NCHRP Synthesis 409. Transportation Research Board, Washington, DC.(14)

This report covers the practices used by operating agencies to revise traffic signal timing. It includes the planning process for developing signal timing plans, as well as the steps involved in developing, installing, verifying, fine-tuning, and evaluating these plans. The authors conducted a literature review and surveyed two large and two small transit agencies, as well as conducted case studies with seven agencies out of the 17 solicited. The report suggests several performance measures for evaluating signal timing, including vehicle performance measures such as volume, travel time, travel time reliability, delay, stops, throughput, queue length, crashes, fuel consumption, vehicle emissions, and progression quality at intersections. Additionally, pedestrian

LOS measures based on pedestrian delay as calculated by the HCM 2000 equation are discussed. The report also covers measures related to traffic signal timing for railroad signal preemption, including the connection between the activation of protection devices and traffic signal clearances to ensure queue clearance from tracks. Finally, the report discusses measures used to assist in retiming signals, such as the average number of phase activations in a given evaluation period, the average number of vehicles served per cycle, the average number of vehicles stored per cycle (residual queue), the probability of a vehicle having to stop at an approach during a given evaluation period, and the percentage of overloaded cycles (or cycle failures) during a given evaluation period.

Shaw, T. (2003). Performance measures of operational effectiveness for highway segments and systems. NCHRP Synthesis 311. Transportation Research Board, Washington, DC.(15)

This synthesis evaluates the use of performance measures to monitor and manage highway segments and systems. It includes more than 70 performance measures, along with their strengths and weaknesses. The report does not cover all traffic SPMs but does include an assessment of the relative strengths and weaknesses of measures such as average vehicle delay at signalized and unsignalized intersections. The synthesis also summarizes the use of performance measures, reporting techniques, and data collection techniques in support of these measures. Based on the results of a survey of state departments of transportation (DOTs) and metropolitan planning organizations (MPOs), the most relevant dimensions of operational performance were found to be the quantity and quality of travel. Measures that reflect these dimensions, such as travel time, speed, and delay, were identified as the most used. Indices derived from these basic units were found to be less relevant in an operational environment, but useful for transportation planning, policy, and prioritization analysis.

Brennan Jr, T.M, Day, C., Sturdevant, J., Raamot, E., & Bullock, D. (2010). Track Clearance Performance Measures for Railroad-Preempted Intersections. Transportation Research Record: Journal of the Transportation Research Board, (2192), 64-76. (16)

This paper presents performance measures based on high-resolution, real-time traffic signal event data that can be used to assess the maximum right-of-way transfer time to track clearance green phases, as well as the synchronization of the track clearance phase with the railroad gate warning system at a crossing. The data were collected at an instrumented intersection on US-36 and Carroll Road in McCordsville, Indiana, using an Econolite ASC/3 controller with high-resolution data logging capabilities. US-36 is a signalized arterial serving commuters in the Indianapolis area, and a significant portion of the roadway runs parallel to dual railroad tracks owned by CSX, which are used by an average of 20 trains per day in both directions with a maximum speed of 60 mph. The performance evaluation focused on the track clearance phase, specifically the termination of the track clearance green interval with the railroad gate descending.

Kazenmayer, L., Ford, G., Zhang, J., Rahman, R., Cimen, F., Turgut, D., & Hasan, S. (2022, June). Traffic Volume Prediction with Automated Signal Performance Measures (ATSPM) Data. In 2022 IEEE Symposium on Computers and Communications (ISCC) (pp. 1-6). IEEE.(17)

This paper analyzes traffic patterns and predicts hourly traffic volumes at nine intersections in Seminole County, Florida, using various machine learning models and the ATSPM dataset. In addition to the ATSPM data, the authors also consider factors such as the day of the week, time of day, holidays, hurricanes, and precipitation. The results show that the Random Forest, XGBoost, and LSTM models consistently outperform other models in hourly traffic volume prediction.

Li, P., Chowdhury, F. R., Wang, P., & Imtiaz, S. M. (2020). Actuated Traffic Signal Performance Evaluation along Arterials using Wi-Fi Travel Time Samples and High-Resolution Traffic Signal Events Data. Transportation Research Record, 2674(6), 268-280. (18)

This study proposes a new framework that combines traditional arterial travel speeds with the PCD. The arterial travel speeds are collected using Wi-Fi Mac addresses and a Kalman filter method and are used to calculate hourly LOS that are added to the traditional PCD. At the single intersection level, the study introduces a new Time-of-Day Volume/Capacity Curve to reflect the relationship between hourly calculated capacity and actual arrivals, providing a new metric for identifying congestion when the volume curve is above the capacity curve. For multiple intersections, the study introduces a multi-intersection coordination diagram that plots the time plan of each intersection and compares it to actual traffic progression. Two new MOEs are also introduced: Arrival-On-Coordination (AOC), which is the number of arrivals within the coordination plan divided by the total number of arrivals during that cycle, and Band Attainability (BA), which is the effective green band length divided by the total green time in that specific cycle. The multi-intersection coordination diagram with these MOEs provides a clear tool for examining traffic progression and reveals the significant sensitivity of progression bands to changes in vehicle speed.

Brennan Jr, T.M, Day, C., Sturdevant, J., & Bullock, D. (2011). Visual Education Tools to Illustrate Coordinated System Operation. Transportation Research Record: Journal of the Transportation Research Board, (2259), 59-72. (19)

This study develops a series of graphics to visualize the operation characteristics of coordinated systems, including time-of-day schedule change time, observed cycle length, green time and split time, coordinated phase actuation, early return to green, arrivals over advance detection relative to the green indication, and progression quality characteristics related to offset. These graphics can be used as a learning tool and as a visual feedback tool to confirm that a coordinated system is operating as expected. Additional graphics were developed to visualize other aspects of coordinated system operation, such as adjacent signal synchronization, coordinated phase operation in rest, plan time changes, preemption, the impact of queuing, and longitudinal analysis of splits. The data collection for this study is based on intersections using commercial off-the-shelf controllers to log all detection phases and relevant events at a 0.1-second resolution. Advance detectors located upstream of the stop bar in the coordinated phase are logged at a 0.1-second resolution, and high-resolution data from a single traffic signal controller under semi-actuated and actuated conditions is also collected.

Brennan Jr, T.M, Remias, S. M., & Manili, L. (2015). Performance measures to characterize corridor travel time delay based on probe vehicle data. Transportation Research Record: Journal of the Transportation Research Board, (2526), 39-50. (20)

This paper develops performance measures to characterize corridor travel time delay using probe vehicle data. A series of traffic message channel (TMC) segments are aggregated, considering a review of congestion hotspots within a corridor, using a visually intuitive methodology. The developed performance measures consider speed variability caused by roadway geometry and other HCM factors that reduce speed, such as friction factors, for each TMC. The traffic performance measures include congestion hours, travel time inflation, and corridor travel time inflation. Visualization techniques are proposed as an intuitive way to convey congestion along a corridor and as a better way to archive data. The study applies an analysis of approximately 90 million speed records collected in 2013 along I-80 in northern New Jersey. Travel time inflation, the time exceeding the expected travel time at 70% of measured free-flow speed, is used to evaluate each of the 166 directional TMC segments along 70 miles of I-80. This performance measure considers speed variability caused by roadway geometry and other HCM factors that reduce speed. The data collection was conducted for the 166 targeted TMCs, resulting in approximately 90 million 1-minute speed records being analyzed for the corridor. These speed data were stored in a database table and aggregated into 15-minute space mean speed (SMS) bins, a total of 96 bins, over 24 hours of each day in 2013 and subsequently stored in a new database table. The performance measurement involves congestion hours and the corridor travel time index.

Zheng, J., Liu, H., Misgen, S., & Yu, G. (2013). Performance Diagnosis Tool for Arterial Traffic Signals. Transportation Research Record: Journal of the Transportation Research Board, (2356), 109-116. (21)

This paper develops a performance diagnosis tool for arterial traffic signals to help agencies make fine-tuned adjustments to signal plans. To evaluate the proposed tool, the authors considered three major parameters of traffic signals: cycle length, offset, and green split, using data collected at intersections along Trunk Highway 13 in Burnsville, Minnesota. A major challenge in performing data collection and analysis is the lack of data collection capacity and efficient performance monitoring tools for traffic signal systems. The diagnosis tool includes three modules: the offset, green split, and cycle-length diagnosis modules. These modules use cycle-by-cycle and phase-by-phase performance measures for analysis.

Ardestani, S. M., Jin, P. J.1, and Feeley, C. (2016). Signal Timing Detection Based on Spatial-Temporal Map Generated from CCTV Surveillance Video. Transportation Research Record: Journal of the Transportation Research Board. (22)

This paper develops signal timing detection methods based on closed-circuit television (CCTV) footage from a major arterial intersection on US-1 in New Jersey. The methods use an ST map-based algorithm to detect signal timing from regular, low-resolution CCTV cameras available at major arterial intersections. The algorithm detects vehicle trajectories by using accurate time information recorded on the ST map. By analyzing the stalling durations of static vehicles on a scanline, the algorithm efficiently detects the starting and ending times of red lights. The results of the algorithm are compared with the output of an InSync system and are found to be satisfactory. The paper proposes performance metrics, including Type I and Type II errors for signal cycle detection, and numeric errors such as mean error and mean absolute error, to evaluate the detection accuracy of starting and ending times. This method can potentially be used at locations where CCTV cameras are installed but signal timing data are not transmitted to centralized arterial management centers.

Cheng, Y., Qin, X., Jin, P. J., Ran, B., & Anderson, J. (2011). Cycle-by-cycle queue length estimation for signalized intersections using sampled trajectory data. Transportation Research Record: Journal of the Transportation Research Board, (2257), 87-94. (23)

This paper presents a measurement method for estimating queue length, a traffic signal performance measure (SPM), based on sampled GPS probe vehicle data. The method uses traffic flow and shockwave characteristics in response to traffic signals and queuing dynamics and introduces the concept of a critical point (CP) to indicate changing vehicle dynamics. A queue length estimation method is developed based on the Lighthill–Whitham–Richards shock wave theory, using CPs related to queue formation and dissipation. The method is designed for real-time applications, providing cycle-by-cycle queue length estimation and instantaneous arterial traffic performance measurements. A CP extraction algorithm is introduced to identify CPs from raw trajectories, and the performance of the method is evaluated using multiple data sets under different flow and signal timing scenarios.

Mirchandani, P. and Head, L. (2001). A real-time traffic signal control system: architecture, algorithms, and analysis. Transportation Research Part C: Emerging Technologies, 9(6), pp.415-432. (24)

This paper presents a real-time traffic-adaptive signal control system called RHODES. The architecture of RHODES is decomposed into several components, including intersection optimization, link flow prediction, network flow prediction, platoon flow prediction, and parameter and state parameter estimation. RHODES can predict short-term and medium-range fluctuations in traffic flow. The authors argue that there is a need to consider signal control performance under different loading levels by comparing effective capacity and offering loads.

The input database for RHODES consists of three types of information: 1) Dynamic data, including real-time detector information, past and planned signal control states, and traffic flow predictions; 2) Model parameters, which are either constant or change slowly over time, such as turning percentages, queue departure rates, average link travel speeds, and other signal timing constraints; and 3) Static data, consisting of infrastructure geometry and other constant information such as the number of lanes, network nodes, arterial length, and detector locations.

In this paper, the authors consider various factors for performance measurement, including types of networks, traffic demand, and statistical issues such as how to characterize traffic demand and how to support conclusions and statements with statistical evidence.

Smaglik, E., Sharma, A., Bullock, D., Sturdevant, J. and Duncan, G. (2007). Event-Based Data Collection for Generating Actuated Controller Performance Measures. Transportation Research Record: Journal of the Transportation Research Board, 2035, pp.97-106. (25)

In this paper, the authors analyzed the collected cycle-by-cycle traffic data to address the limitation of typical data collectors, which usually collect data on an hourly or 15-minute basis, making it difficult to evaluate the performance of individual phases and splits on a cycle-by-cycle basis. To achieve this, the authors described a general-purpose data collection module within a National Electrical Manufacturers Association (NEMA) actuated traffic signal controller with some hardware modifications. The performance measurements that can be extracted from controllers using this method include Equivalent Hourly Volume (EHV), Arrival Type (AT) Data, and Delay Data.

Liu, H. and Ma, W. (2009). A virtual vehicle probe model for time-dependent travel time estimation on signalized arterials. Transportation Research Part C: Emerging Technologies, 17(1), pp.11-26. (26)

Liu and Ma proposed a virtual probe vehicle model for signalized arterials as part of a real-time arterial data collection and archival system. The virtual probe vehicle can be traced to estimate time-dependent travel time along an arterial using high-resolution "event data" from a simultaneous data collection system. They also introduced a real-time arterial performance measurement system called SMART-SIGNAL which includes a data collection, storage, and analysis system. Both signal status data and vehicle detection data are used to determine the state of the virtual probe vehicle. The results from the virtual probe can be used to estimate other performance measures, such as the number of stops, control delay at intersections, LOS, and queue length in oversaturated situations.

Comert, G. (2013). Simple analytical models for estimating the queue lengths from probe vehicles at traffic signals. Transportation Research Part B: Methodological, 55, pp.59-74. (27)

Queue length is an important measurement that can be used to estimate delay and travel time at a signalized intersection. Probe vehicles (vehicles equipped with GPS and wireless communication tools) have been widely used to gather travel time and traffic speeds, while queue length and delays are more challenging to derive from probe vehicle data. In this study, a mathematical model was developed to estimate queue lengths using traffic mobile sensing data from probe vehicles. It is important to note that the queue length is estimated by using the location and time of probe vehicles.

Wu, X., Liu, H. and Gettman, D. (2010). Identification of oversaturated intersections using high-resolution traffic signal data. Transportation Research Part C: Emerging Technologies, 18(4), pp.626-638. (28)

Wu et al. developed an algorithm to identify oversaturated intersections using high-resolution signal data, which quantitatively measures the severity of oversaturation at a signalized intersection. In this paper, the authors introduced the oversaturation severity index (OSI) as a performance measurement, including temporal dimension (T-OSI) and spatial dimension (S-OSI). The focus of this OSI shifts from measuring travel demand to measuring the detrimental effects of congestion both temporally and spatially. The OSI algorithm consists of two main parts: residual queue length estimation and the detection of spill-over conditions. It is assumed that high-resolution (second-by-second or event-based) traffic data can be obtained. The authors experimented to test their method on a selected arterial.

Hao, P. and Ban, X. (2015). Long queue estimation for signalized intersections using mobile data. Transportation Research Part B: Methodological, 82, pp.54-73. (29)

Hao and Ban examined the use of mobile data to estimate queue length for signalized intersections, particularly when the queue is very long. The "long queue" problem is defined as a queue that extends beyond the area of detection. The advantage of using mobile data to address the "long queue" problem is that it can detect queues that are far from the intersection. To account for undetected acceleration or deceleration delay, car following models were used to reconstruct the queue profile. The authors divided vehicle arriving types into four categories based on three parts of intersection delay: deceleration delay, queuing delay, and acceleration delay. Using the delay-based method, the "long queue" problem is reduced to a "short queue" problem, which can be solved with existing solutions.

CHAPTER 3:
STAKEHOLDER SURVEY ON ATSPM

3.1. BACKGROUND AND OBJECTIVES

The Advanced Transportation System Performance Monitoring system is a tool that monitors the performance of transportation systems, including roads, highways, and intersections. It provides real-time data on traffic flow, congestion, and other key metrics that are essential for transportation planning and management. This survey aims to collect feedback from related professionals, such as traffic engineers, planners, and operations staff, on their current experiences and expectations of the ATSPM system. The goal of the survey is to understand how the system is being used, what challenges and successes users have encountered and achieved, and how the system can be improved to better meet the future needs of transportation professionals.

3.2. SURVEY DESIGN

The survey was open from November 15th to December 16th, 2022. The questions were divided into two sets: The first 13 questions were to query the respondents' previous experience with the ATSPM system, and the remaining 8 questions were for those with no experience in ATSPM. The questionnaire's structure is illustrated in Fig. 3-1. The details of each question can be found in Appendix A. This survey was designed using the enterprise version of QuestionPro, a professional survey software to increase the sample size.

- The research group's LinkedIn page (the announcement reached 2,603 anonymous readers);
- Email lists of TexITE and ITS Texas (640 email recipients)
- Email list of North Central Texas Council of Government (NCTCOG) (about 200 email recipients)
- Email Lists of the TRB traffic control device committee (ACP50), traffic simulation committee (ACP80), and traffic signal committee (ACP 25) (over 2,000 recipients in total)

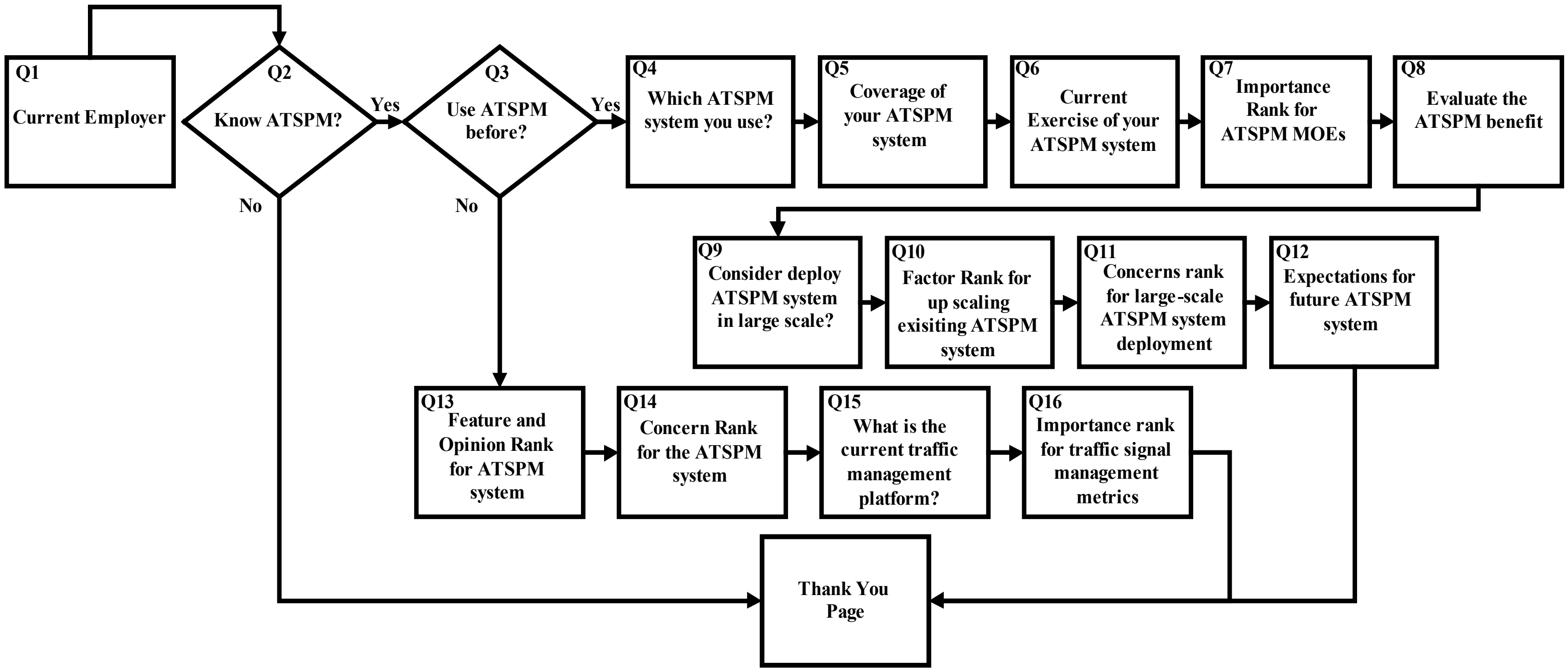


Figure 3-1: Stakeholder survey design on ATSPM.

3.3. SUMMARY OF SURVEY RESULTS

During the survey period, a total of 162 individuals participated. Of these, 90 completed the survey in its entirety, yielding a completion rate of 55.56%. The average time spent to complete the survey was 4 minutes. The majority of respondents, 85.29%, were located in the United States, while 3.7% and 2.47% were from Nepal and India, respectively.

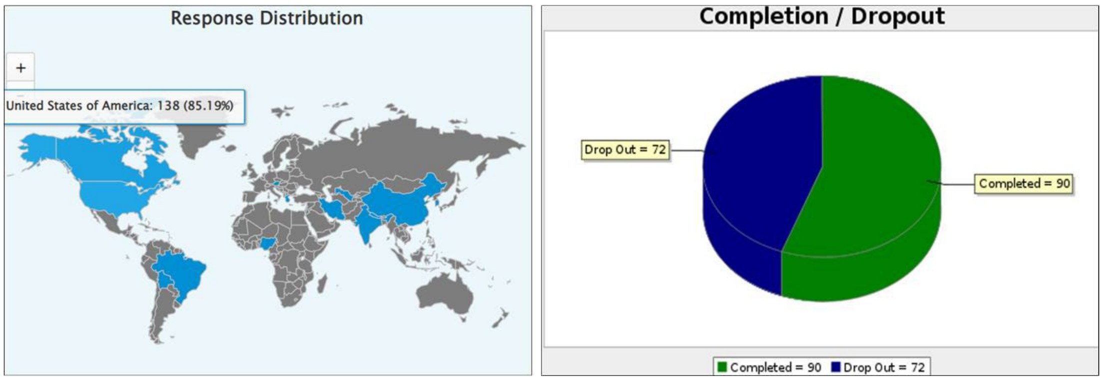


Figure 3-2: Overview of survey respondents.

In the following section, we will analyze the responses to each survey question in detail. This analysis will provide insights into the opinions and perspectives of the respondents on various topics.

- **Question 1: What's your current employer?**

For this question, we received a total of 122 responses. 32 respondents were employed by consulting firms, 27 worked for municipal traffic agencies, 25 were affiliated with academia or research institutes, and 23 were employed by state departments of transportation. The remaining responses came from a variety of sources, including federal agencies, public transit officials, and traffic control or signal software vendors. In general, we had a diverse pool of respondents.

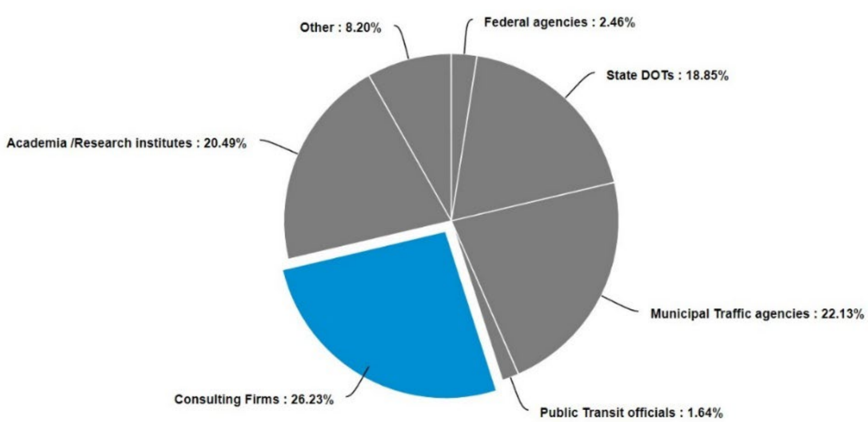


Figure 3-3: Pie chart for respondents' employment.

- **Question 2: Did you deploy, use, or consider the ATSPM system in traffic management?**

We received 119 responses for question 2. 74 respondents (62.18%) indicated that they have deployed, used, or are considering using the ATSPM system in traffic management. The remaining 45 respondents (37.82%) indicated that they have not deployed, used, or considered using the ATSPM system. If a respondent answered No, then he/she was considered an irrelevant respondent and was guided to exit the survey.

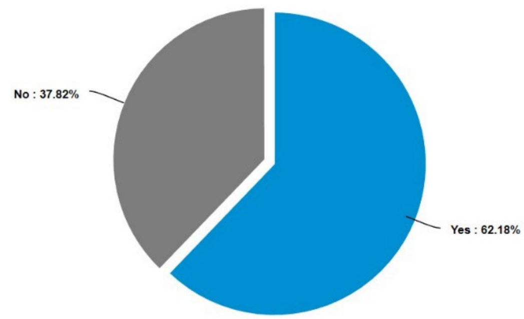


Figure 3-4: Pie chart for deploying, using, or considering ATSPM system in traffic management.

- **Question 3: Have you ever used an ATSPM system before?**

Question 3 focused on the experiences of the 74 respondents who had previously indicated that they have deployed, used, or considered using the ATSPM system in traffic management. Of these respondents, 53 (71.62%) respondents reported having prior experience using the ATSPM system, while 21 (28.38%) reported that they had never used the system (i.e., they were only interested in ATSPM). In the following questions (Q.4- Q.12), we focused on the experiences of the 53 respondents who had previously indicated that they had used the ATSPM system. These questions aimed to gather information on the respondents' perceptions and evaluations of the system. In contrast, questions 13-16 focused on the 21 respondents who had not previously used the ATSPM system and sought to gather information on their motivations and barriers preventing them from adopting.

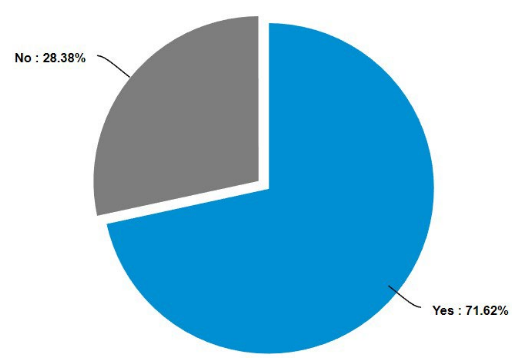


Figure 3-5: Pie chart for respondents' experience of using an ATSPM system.

▪ **Question 4: Which ATSPM system are you using or considering?**

68 respondents answered this question, indicating that the most popular ATSPM system was the open-source ATSPM system contributed by UDOT. The UDOT's ATSPM is actively used by 32.35% of respondents (22 respondents). The second most popular system was Q-free Maxview's ATSPM module, with 19.12% of respondents (13 respondents) using it. Other ATSPM systems that received responses included Iteris Cloud-based ATSPM (13.24%, 9 responses), INRIX (11.76%, 8 responses), and various versions of UDOT's ATSPM system (7.35%, 5 responses). Additionally, some respondents reported using other options such as NoTraffic's ATSPM module, Econolite Centracs, Miovision, Kimley-Horn's KIT system, and self-developed solutions such as UTAIM.

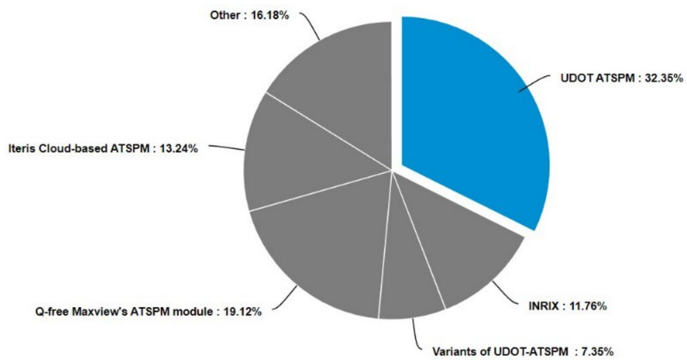


Figure 3-6: Pie chart for respondents' choice for ATSPM system.

▪ **Question 5: Coverage of your ATSPM system (deployed or considered)?**

Out of the 46 respondents, 25 reported having more than 30 intersections connected to their ATSPM system. Among these 25 respondents, the majority had a moderate scale of 150-350 intersections, while three respondents reported having more than 1,000 intersections connected. Sixteen respondents reported having a medium-sized ATSPM system deployment with 11-30 intersections, and five respondents reported having a small-scale deployment of 1-10 intersections.

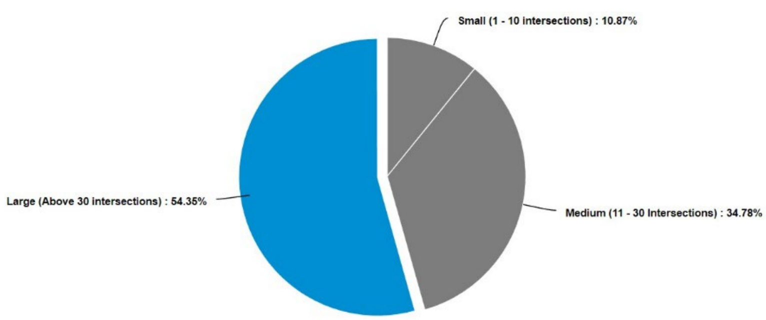


Figure 3-7: Pie chart for respondents' ATSPM system scale.

▪ **Question 6: Which best describes your current exercise with the ATSPM system?**

Out of the 44 respondents for question 6, 22 (50%) reported actively using their ATSPM system for daily traffic management. Another 14 (31.82%) respondents said they used their system occasionally, while 6 (13.64%) respondents were still trying it out. The remaining 2 (4.55%) respondents reported being in the planning stages for implementing an ATSPM system.

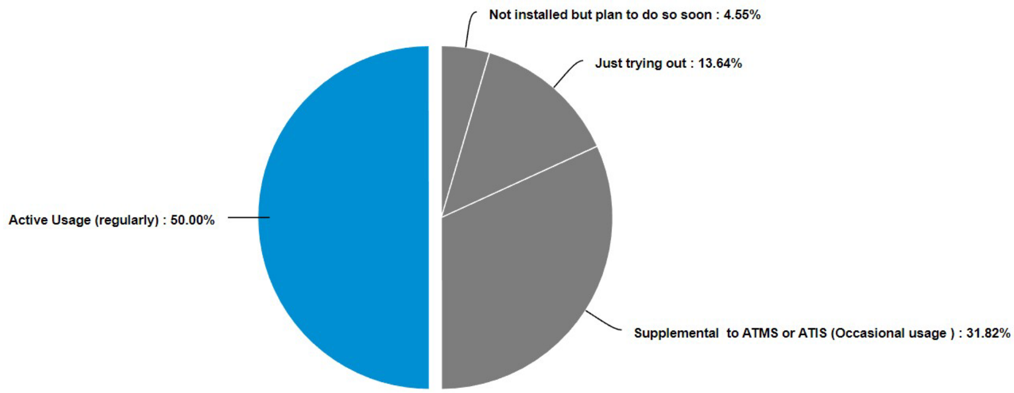


Figure 3-8: Pie chart for respondents' current exercise with the ATSPM system.

- **Question 7: On a scale of 1-5 (1. Not useful. 2. Kind of Useful; 3 It's OK; 4 Very useful; 5. Must have), what is the importance of the following commonly used ATSPM MOEs?**

Based on the respondents' rankings, the top five ATSPM modules were the Split Monitor module (4.17 points, 83.33% agreement), PCD (4.03 points, 80.56% agreement), Turning Movement Counts (3.95 points, 78.92% agreement), TSD (3.94 points, 78.79% agreement), and Approach Volume (3.92 points, 78.38% agreement). The bottom five modules were Ped Delay (3.5 points, 70% agreement), Approach Delay (3.49 points, 69.73% agreement), Preemption Delay (3.44 points, 68.89% agreement), Multimodal Chart (3.16 points, 63.12% agreement), and Approach Speed (3.14 points, 62.78% agreement).

The overall ranking is as follows:

1. Split Monitor
2. PCD
3. Turning Movement Counts
4. Time-Space Diagram
5. Approach Volume
6. Timing and Actuation
7. Purdue Split Failure
8. Arrive On Red
9. Yellow and Red Actuation
10. Time of Day Volume/Capacity Plot
11. Left-Turn Gap
12. Ped Delay

13. Wait Time
14. Approach Delay
15. Preemption Details
16. Multimodal vehicle arrival diagram
17. Approach Delay

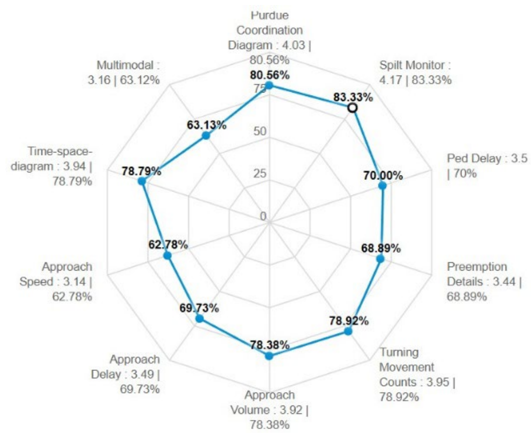


Figure 3-9: Spider chart ATSPM module ranking (top 5 and bottom 5).

- **Question 8: Please evaluate the ATSPM's benefits you perceive.**

Based on the rankings from the 33 respondents for question 8, the top-ranked ATSPM modules were Signal Health Monitoring (4.45 points, 89.09% agreement), Reduce Delay and Emissions (3.91 points, 78.18% agreement), and Causes for Congestion (3.67 points, 73.33% agreement). The lowest-ranked module was Safety Analysis (3.58 points, 71.52% agreement). Overall, the ranking order for the modules was Signal Health Monitoring, Reduce Delay and Emissions, Causes for Congestion, and Safety Analysis.

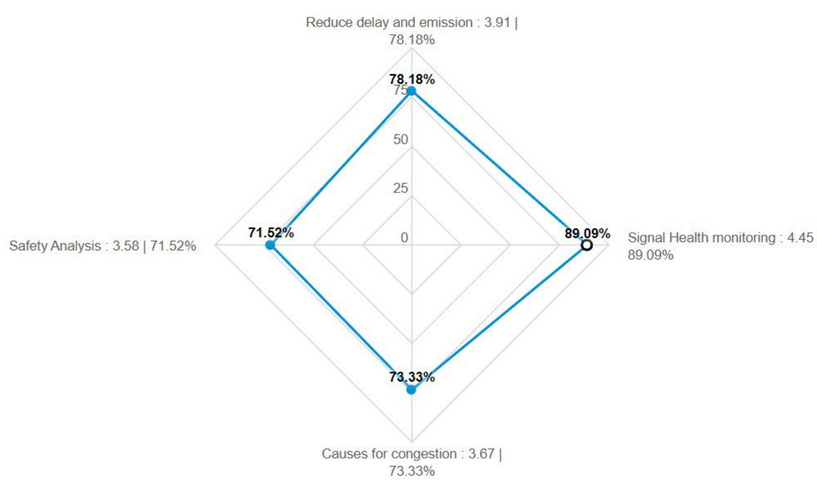


Figure 3-10: Spider chart ATSPM benefit ranking.

- **Question 9: Have you deployed ATSPM, or do you consider deploying ATSPM at a large scale (>30 intersections)?**

In response to question 9, 33 respondents (93.94%) provided answers. Of these, 31 respondents reported considering deploying ATSPM at a large scale, while only 2 respondents (6.06 %) did not consider this option.

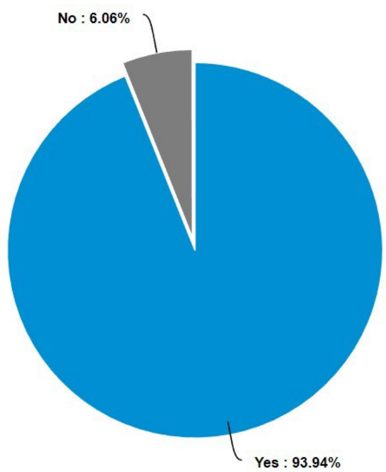


Figure 3-11: Pie chart for considering upscaling the ATSPM system.

- **Question 10: What inspires you to consider upscaling your ATSPM system? (Please rank)**

According to the 33 responses, the top reason for upscaling the ATSPM system was the desire for comprehensive MOEs (4.27 points, 85.45% agreement). This was followed by the availability of budget and affordability (3.85 points, 76.79% agreement) and ease and straightforwardness (3.42 points, 68.48% agreement). The least common reason cited was a requirement from the administration (2.64 points, 76.97% agreement).

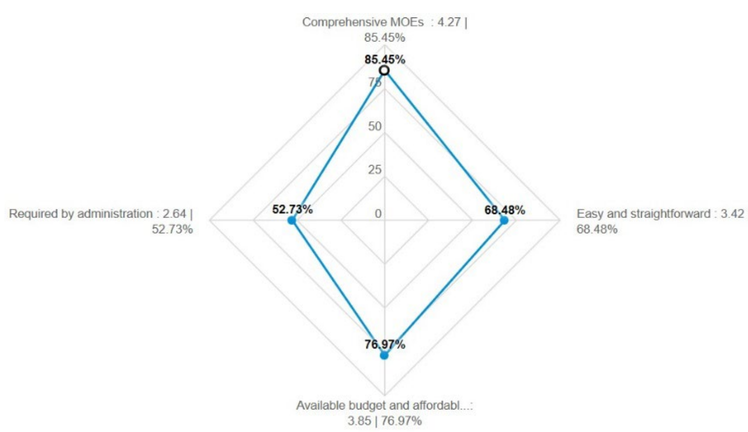


Figure 3-12: Spider chart for upscaling ATSPM system.

- **Question 11: What concerns you about the large-scale deployment of the ATSPM system? (Please rank)**

According to the 33 responses, the top concern for upscaling the ATSPM system was the cost of hardware and software (3.55 points, 70.91% agreement). This was followed by concerns about the need for frequent major software upgrades potentially leading to a lack of futureproofing (3.27 points, 65.45% agreement) and the need for a lot of training (3.21 points, 64.24% agreement). Other concerns included the potential overlap of functions with other ITS/big-data solutions (2.88 points, 57.58% agreement) and the difference between the ATSPM system and the current practice.

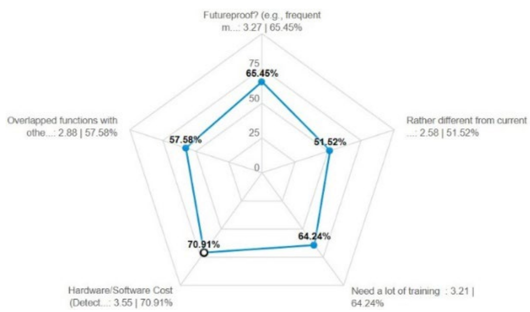


Figure 3-13: Spider chart for concerns about upscaling the ATSPM system.

- **Question 12: What are your wishes and expectations for the ATSPM system in the future?**

According to the 33 responses, the most desired module for future ATSPM systems was corridor-level traffic signal performance metrics (4.48 points, 89.70% agreement). This was followed by the introduction of more safety-related MOEs (4.03 points, 80.61% agreement), the incorporation of emerging big data such as connected vehicle and crowdsourced traffic data to create innovative MOEs (4 points, 80% agreement), AI-empowered automated problem identification and decision support for traffic signal management (3.79 points, 75.76% agreement), and the integration of inclusive sensing data such as pedestrian and bicyclist data (3.76 points, 75.15% agreement). The least desired module was multimodal traffic (3.67 points, 73.33% agreement).

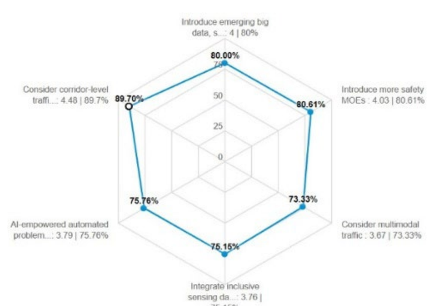


Figure 3-14: Spider chart for expectations of future ATSPM system.

Please be aware that questions 13 to 16 were intended to be answered by respondents who have not used an ATSPM system before.

- **Question 13: Based on your understanding, please rank your opinions about ATSPM and its features.**

In total, 44 respondents answered question 13. Based on their rankings, the top-rated aspect of ATSPM systems was their ability to provide automation, visualization, and decision-making processes (3.96 points, 79.17% agreement). This was followed by the robust and resilient nature of ATSPM systems (3.75 points, 75% agreement), the ATSPM ease and straightforwardness (3.56 points, 71.25% agreement), and their ability to provide future predictions (3.29 points, 65.83% agreement). The lowest-rated aspect was multimodal metrics (3.23 points, 64.58% agreement).

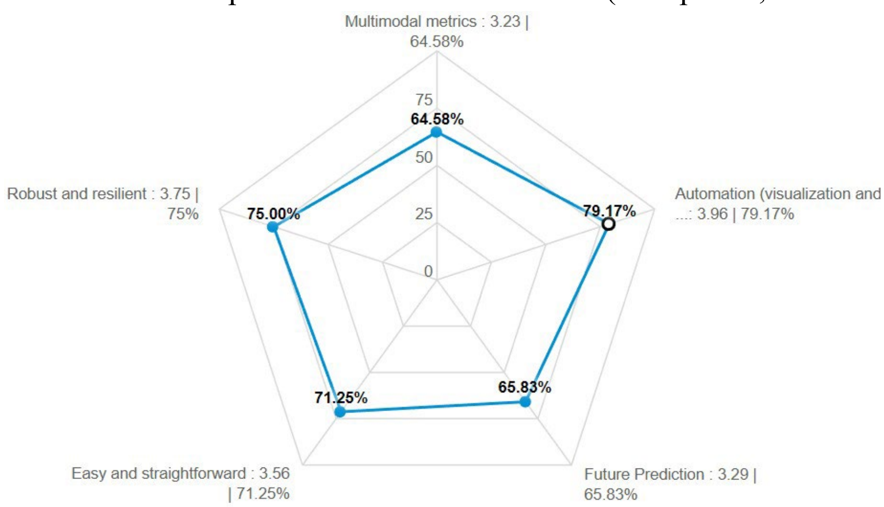


Figure 3-15: Spider chart for opinions about the ATSPM system.

- **Question 14: What are your concerns about the ATSPM system? (Please rank)**

According to the survey responses, the top concerns among respondents who were hesitant about using an ATSPM system were related to long-term maintenance (75% agreement, 3.75 points). This included worries about the time and resources needed to keep the system running smoothly and the potential for unexpected issues. Training (74.58% agreement, 3.73 points) was also a significant concern, with many respondents expressing a need for extensive training to use and maintain the system effectively. Cost (69.17% agreement, 3.46 points) was another concern, with some respondents worried about the upfront and ongoing expenses of implementing and using an ATSPM system. A smaller number of respondents expressed doubts about the reliability of ATSPM systems (67.08% agreement, 3.35 points), questioning whether they would consistently perform as expected.

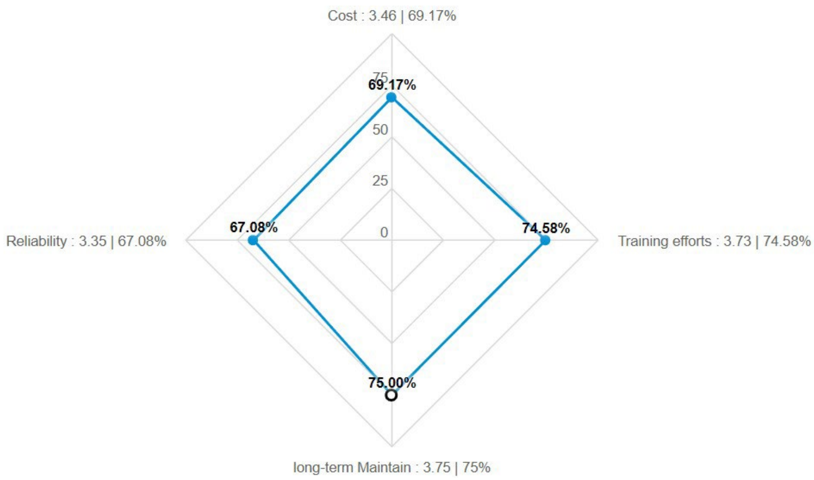


Figure 3-16: Spider chart for opinions about the ATSPM system.

- **Question 15: What's the current traffic signal management platform that you are working with?**

Out of the 69 respondents who have not used an ATSPM system before, a significant portion is currently using other traffic management solutions. Specifically, 22 respondents (31.88%) are working with ATMS, 21 (30.43%) are using open-source software, 15 (21.74%) are using other commercial traffic signal management software, 4 (5.8%) are utilizing research-spinoff systems from academia, and 7 (10.14%) have selected "other" as their current solution. The "other" category includes the use of internal software as well as mentions of transparency and Omni software.

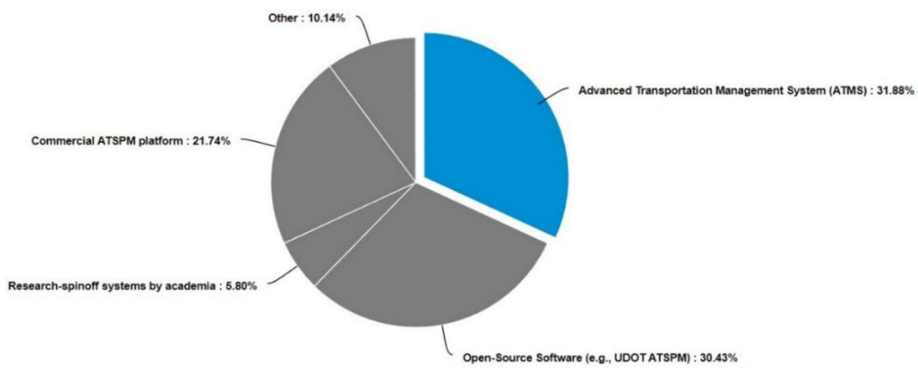


Figure 3-17: Pie chart for current traffic signal management platform.

- **Question 16: Rank the importance of the following performance metrics in traffic signal management. (1: Least important, 5: Most important)**

According to the responses of 45 respondents, the top five performance metrics are, in order of importance, Split Monitor (4.24 points, 84.89% agreement), PCD (4.13 points, 82.67%

agreement), Timing and Actuation (3.96 points, 79.11% agreement), Turning Movements and Counts (3.89 points, 77.78% agreement), and Arrive on Red (3.89 points, 77.78% agreement). The bottom five are Preemption Details (3.49 points, 69.78% agreement), Ped Delay (3.33 points, 66.67% agreement), Multimodal (3.2 points, 64% agreement), Left-Turn Gap (3.31 points, 66.22% agreement), and Approach Speed (3.02 points, 60.44% agreement).

The overall ranking is as follows:

- PCD
- Spilt Monitor
- Timing and Actuation
- Turning Movement Counts
- Approach Volume
- Arrive on Red
- Purdue Split Failure
- Corridor PCD
- Time of Day Volume / Capacity Plot
- Approach Delay
- Wait Time
- Preemption Details
- Yellow and Red Actuations
- Ped Delay
- Left-Turn Gap
- Multimodal PCD > Approach Speed

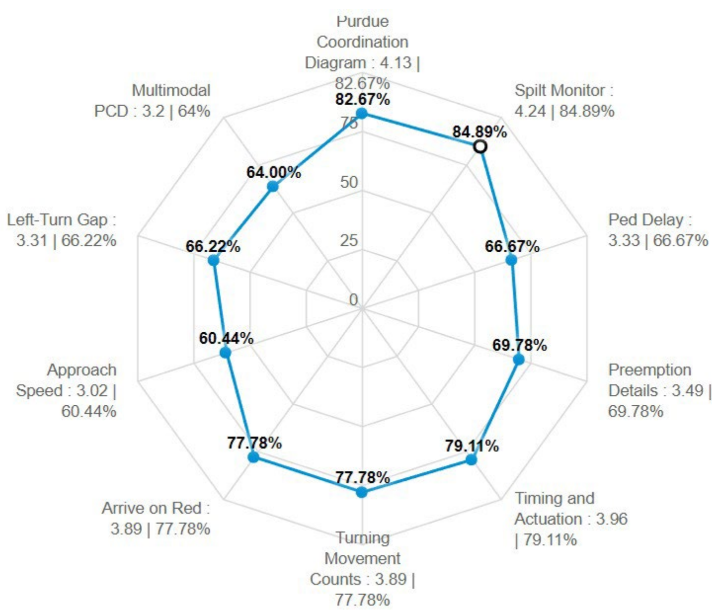


Figure 3-18: Spider chart for importance rank for signal management performance metrics.

3.4. SUMMARY AND RECOMMENDATIONS

The survey data show how the respondents perceive the potential risks related to ATSPM; the actions that some stakeholders are taking to mitigate the operational risks while using ATSPM; and the stakeholders' opinions on various functions. The objective of this survey is to improve the effectiveness and efficiency of traffic signal systems. The results of the survey provide direct facts on how the stakeholders are using ATSPM and what should be improved as a priority in the future. The Survey obtained information from a diverse group of academic, government, and industry experts.

To further promote ATSPM, decision-makers must understand the barriers, concerns, and risks associated with ATSPM deployment. It is also the same crucial to understand the motivations and interests of stakeholders to fully embrace the benefits of the ATSPM system. Overall, the survey findings suggest that the concepts of ATSPM have been broadly accepted by the stakeholders. Many respondents consider expanding their existing ATSPM systems and some agencies have deployed the ATSPM at a very large scale (over 1,000 intersections). According to the stakeholders' experiences of the ATSPM systems, the survey results are divided into two groups:

3.4.1. Respondents have experience with the ATSPM system.

According to the survey data, a significant percentage of stakeholders have experience with the ATSPM system and found it useful. Among all the respondents, 74 (62.18%) have experience with deploying and using or considering deploying ATSPM; 53 (71.62%) have used at least one ATSPM system before. The most popular ATSPM system is the open-source ATSPM contributed by Utah DOT or its variants. Other popular ATSPM systems include the Q-free Maxview's ATSPM module, Iteris Cloud-based ATSPM, and INRIX ATSPM Module. Many respondents have deployed the ATSPM systems at a large scale. 54.35% of respondents have deployed the ATSPM systems at over 30 intersections. The top three attractive performance measures in the ATSPM systems to the stakeholders are <u>Split Monitor</u>, <u>PCD,</u> and <u>TSD across multiple intersections</u>. The survey also indicates that the stakeholders also like the ATSPM functions such as monitoring the health of traffic signals and reducing delays and emissions at intersections. Most of the respondents (93.94%) are positive about upscaling their existing ATSPM systems, although some showed hesitance due to concerns about hardware/software costs and the maturity of current systems. For future development of ATSPM, the respondents were the most interested in corridor-level traffic signal performance metrics, safety-related measures, and the introduction of emerging big data such as connected vehicles and crowdsourced traffic data. In summary, the survey conveys the following information for the stakeholders who have experience and/or knowledge of ATSPM.

- ATSPM has been commonly recognized and accepted as a set of effective tools for managing traffic signals, and most respondents are positive about expanding the scope of existing ATSPM systems.
- UDOT's open-source ATSPM system and its variants are the most popular ATSPM systems among the respondents.

- Split Monitor and PCD are the most popular performance measures provided by the ATSPM systems.
- Among those respondents who have concerns about scaling up their existing ATSPM systems, the main concerns are the software/hardware costs and the maturity of current systems.
- Toward future ATSPM functions, the respondents are highly interested in the corridor-level traffic signal performance metrics (e.g., time-space diagram), safety-related measures, and the use of emerging big data such as connected vehicles and crowdsourced data.

3.4.2. Respondents have no experience with the ATSPM system.

Among the 119 respondents, 37.82% of 45 respondents stated that they had no prior experience with the ATSPM system. A vast majority of these respondents had a positive opinion of the automated visualization and decision support offered by the system. 75% of the 45 respondents rated the ATSPM functions as better than neutral. They also believed that the ATSPM systems are robust and resilient (49% of the responders rated it as better than neutral, while 43.75% of them rated it as neutral). 50% of the respondents said that operating the ATSPM system was simple and uncomplicated. Nonetheless, only 39.58% were satisfied with the multimodal monitoring capabilities of the current ATSPM systems. Regarding the capability of traffic state prediction, 43.75% of the responders thought that it would benefit traffic operations.

The survey also indicates that the necessary training and long-term maintenance are the major concerns of deploying the ATSPM system (78.17% of responders agreed to the training effort and 75% of responders agreed to maintenance, respectively). Other concerns include cost (56.25%) and dependability (54.16%). 31.88% of the respondents indicated that they were using an ATMS, 30.43% were using other open-source software, 21.74% were using a commercial traffic signal management platform, and 5.8% were using research spinoff systems developed by academia. These statistics reveal the current status of ATSPM applications. Respondents agreed that the PCD and the Split Monitor were the top two metrics for measuring performance in traffic signal control. In summary, the survey conveys the following information for the stakeholders who have no experience with ATSPM.

- They can likely operate the ATSPM system comfortably.
- They are very interested in the ATSPM systems' automated visualization and decision support.
- Required training and long-term maintenance are the key obstacles to new ATSPM system deployment.
- The respondents are mostly utilizing ATMS and other open-source software to manage traffic lights.
- The PCD and the Split Monitor are the most attractive traffic signal metrics.

CHAPTER 4:
VISSIM MODEL CALIBRATION

4.1. INTRODUCTION OF VISSIM MODEL CALIBRATION

Traffic analysis is to assess the impact of traffic demand and supply on roads to examine if the target standards or goals are met. VISSIM Calibration refers to the procedure of fine-tuning the VISSIM model's parameters to precisely replicate real-world driving behaviors and traffic conditions. It includes comparing the model's outputs to actual observation and adjusting various parameters until the model's outputs match the observation in the field. This chapter is to provide guidance and instruction to demonstrate how the traffic simulation models are modeled and calibrated based on all available information, including traffic volumes, signal timing, and the connected vehicle data (i.e., Wejo data) to produce accurate and reliable results.

The Wejo Dataset refers to the data collected from connected vehicles which includes vehicle trajectories, speeds, and headings. Using the Wejo data, we can estimate the distributions of vehicle speed as well as acceleration and deceleration functions. The guidelines also include recommendations for selecting appropriate MOEs to ensure that the VISSIM model produces the desired results by effectively using all available datasets in TxDOT. Applying these findings, we developed and calibrated the VISSIM model along Cooper Street in Arlington Texas.

4.2. DATA COLLECTION AND PREPARATION

A combination of traditional historical traffic data and emerging traffic data was collected and compiled. Traditional traffic data includes traffic volumes, speeds, and signal timings. Such data are typically available at public agencies. The traditional traffic data are used to calibrate fundamental parameters in the VISSIM model. Connected vehicle data, on the other hand, provide more details about individual vehicles' behaviors, such as instantaneous longitude, latitude, speed, and heading, etc. The behavioral data are used to further fine-tune the base parameters of VISSIM models. The connected vehicle data are commercially off-the-shelf, such as the Wejo data. In this project, the project team used the Wejo data they previously procured to calibrate the behavioral distributions in VISSIM. Using both traditional traffic data and emerging connected vehicle data for calibration can further improve the model's accuracy to replicate as well as predict real-world traffic conditions. Nonetheless, it also requires extensive data collection and preparation.

The City of Arlington provided the project team with historical traffic volumes collected in 2020 (during the pandemic) along the 15 intersections of Cooper Street, stretching from the Division Road and the I-20 interchange, except for a major intersection, the Pioneer Pkwy. It was found that the traffic volumes in 2020 were significantly lower than normal conditions. Collecting the latest traffic volumes at all intersections is not within the scope of this project. So, we decided to augment the historical volumes according to the latest mainline volumes that we collected at a few locations on Cooper Street. The mainline volumes were collected with the *AI-empowered Video Analytics for Smart Transportation* or AVAST, developed by the project team at UT Arlington. Traffic videos at two locations were recorded from Arlington's cameras around UTA Blvd and

Park Row Dr during peak hours. The collected mainline volumes were compared with the historical numbers at the same places to estimate adjustment factors (between 1.5 and 2). All the traffic turning movement counts have then been augmented accordingly.

Since the arterial scope covers major intersections traffic characteristics may significantly change before and after vehicles pass those major intersections. As shown in Fig. 4-1, the entire corridor was divided into three zones: Zone 1 from Division Street to Mitchell Street, Zone 2 from Park Row Dr to W Inwood Drive, and Zone 3 from Pioneer to I20 Interchange. In Zone 3, so we used the increment factor from UTA Blvd to increase the traffic volume in Zone 1, while the factor obtained from Park Row Dr intersection data was utilized to increment the remaining locations.

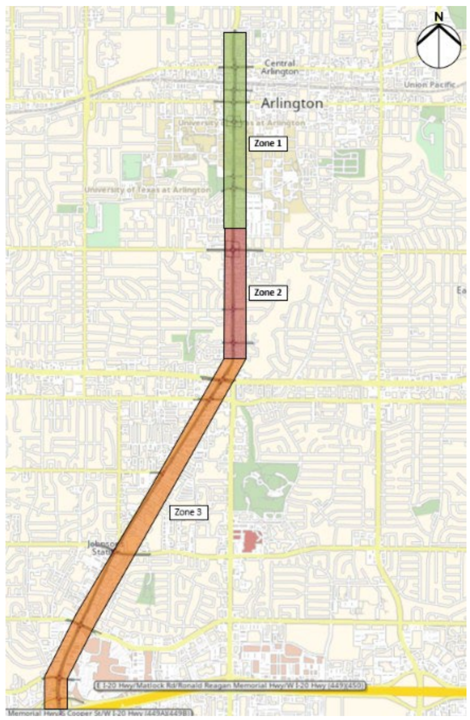


Figure 4-1: Different zones in Cooper Street Corridor.

Traffic signal timings are all available at the 15 intersections. The City of Arlington collected the comprehensive timings at 15 intersections from its central ATMS, TACTICS®. At each intersection, the phasing sequences, timings, detector mappings, overlaps as well as any relevant configurations were input into the default traffic signal emulator in VISSIM, the RBC controller. As for the Wejo data, the project team drew a geofence of a polygon shape and only kept the Wejo data points within the polygon. The connected vehicle (CV) data sample represented 3%-10% of the total traffic which is sufficient to provide accurate speed and acceleration distribution data. A previous study shows that the speed profile from the CV data around the infrastructure sensors is consistent with the measured speed by the infrastructure sensors during the peak hours, shown in Fig. 4-2. (Khadka, Wang, Li, & Torres, 2023).

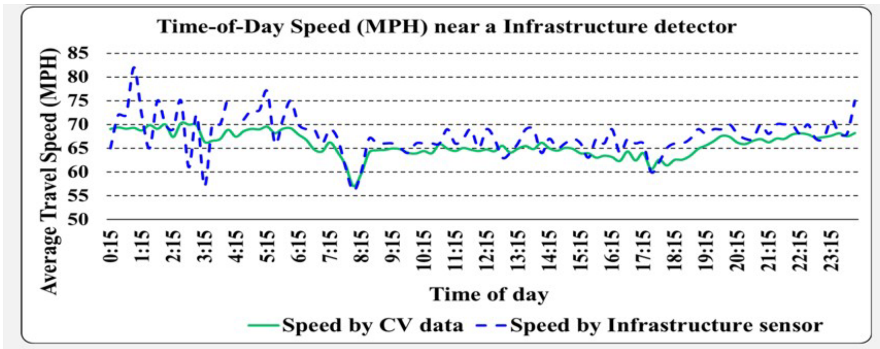


Figure 4-2: Comparison of Average speed distribution between CV data and Infrastructure sensor.

4.3. CALIBRATION PARAMETERS AND PROCEDURE

The following parameters were calibrated in VISSIM according to the available data sets:

4.3.1. Parameters of the car-following models in VISSIM

Step 1: Users can access the Driving Behavior window located under the "Base Data" tab. Within this window, different driving behaviors, such as arterial and freeway, can be selected depending on the type of network being modeled. Additionally, for each driving behavior, there are two options for the car following model: "Wiedemann 74" and "Wiedemann 99". It is generally recommended to use the "Wiedemann 99" model as it operates at a higher level of detail than the Wiedemann 74 model, simulating the flow of groups of vehicles instead of individual vehicles while still being based on the same car-following theory. Please see Fig. 4-3.

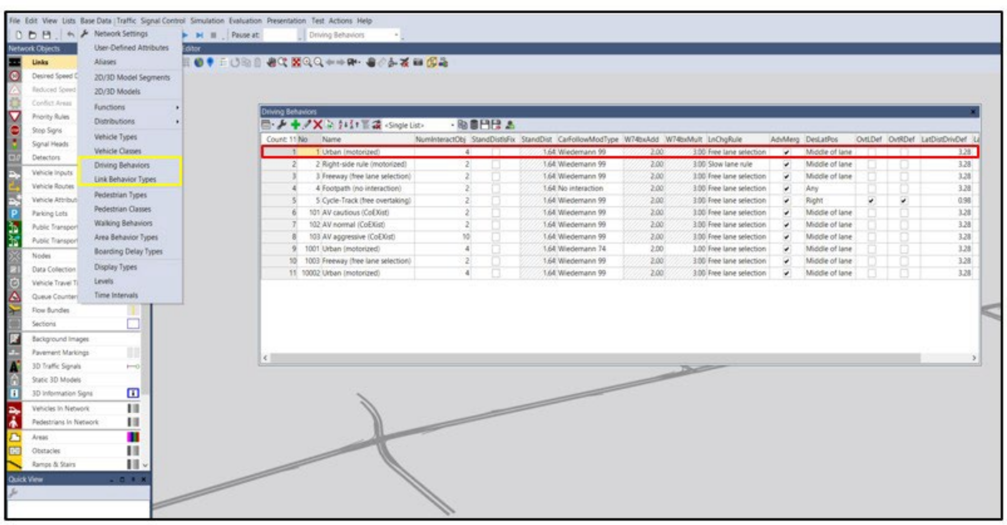


Figure 4-3: VISSIM tutorial snapshot on selecting driving behavior.

Step 2: The "Look Ahead Distance" tab allows for the configuration of various parameters that define the range of visibility for a vehicle while in motion. The tab includes the following parameters:

1. Minimum: This parameter represents the minimum distance from which a vehicle can detect all objects. It serves as the lower limit for the look-ahead distance. Please see Fig. 4-4.
2. Maximum: This parameter represents the maximum distance up to which a vehicle can detect all objects/vehicles. It serves as the upper limit for the look-ahead distance. A vehicle can interact with or detect any object/vehicle within the range of the minimum to the maximum distance.
3. The number of interaction objects: This parameter includes the number of objects, other than vehicles, that a vehicle can interact with. These objects may include stop signs, signal heads, speed limits, etc. The vehicle itself is also included as an interaction object. In urban arterial conditions, this value is typically larger than in freeway conditions. The number of interaction vehicles: This parameter includes the number of vehicles that a vehicle can detect other than itself.

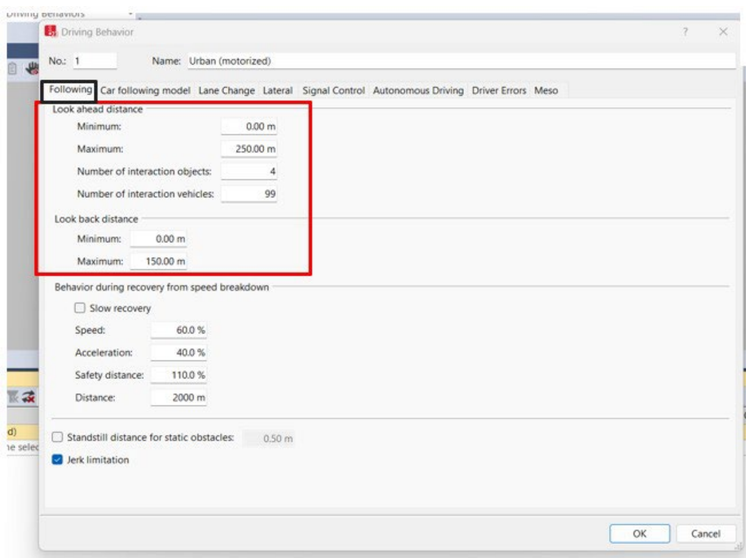


Figure 4-4: Snapshot of Vissim with the following tab highlighted to calibrate the overall following model.

Step 3: The "Wiedemann 99" model is utilized for this project, which is preferred due to its detailed simulation of vehicle flow in comparison to the Wiedemann 74 model while being based on the same car-following theory. The Wiedemann 99 model consists of 10 different parameters ranging from CC0 to C9. One of the advantages of this model is its linear relationship between desired safety distance and speed, which makes it suitable for modeling both freeways and other road networks. Please see Fig. 4-5.

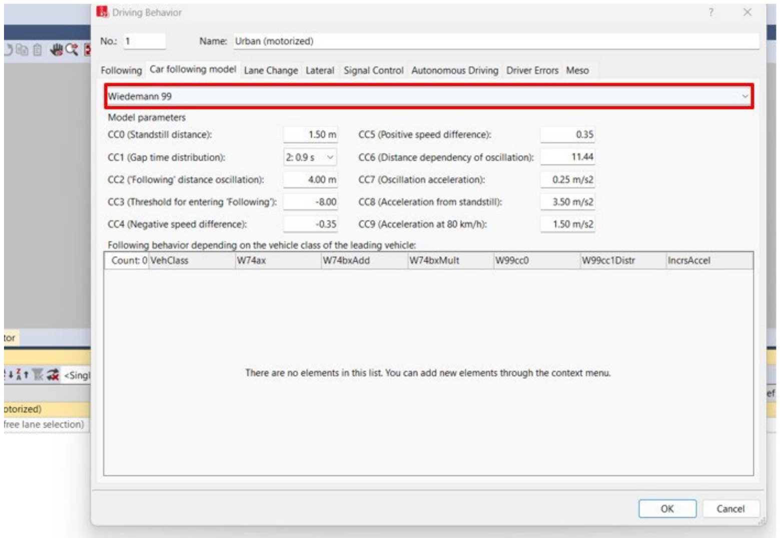


Figure 4-5: VISSIM interface with 'Car-Following' tab and Wiedemann 99 selected.

Step 4: The first four parameters from CC0 to CC3 pertain to standstill distance, gap time, and oscillation distance. These parameters include the following:

1. CC0 (Standstill distance): This parameter determines the desired distance between two vehicles when they are in a standstill scenario. It has a default value of 1.50m and is inversely proportional to the saturation flow rate. The value of CC0 affects jam density. Please see Fig. 4-6.
2. CC1 (Gap time distribution): CC1 refers to the time headway or the distance that the driver wants to maintain between vehicles. It is provided in the form of a distribution and is the time distribution of the speed-dependent part of the desired safety distance in the car-following model. A new gap time distribution can be created using the "Base Data" menu. In general, this value is higher when calibrating freeway models compared to arterial settings.
3. CC2 (Following distance oscillation): CC2 determines the additional safety distance beyond the desired safety distance that is required when vehicles are following each other before the following vehicle moves closer to the leading vehicle.
4. CC3 (Threshold for entering the following model): This parameter is a threshold for the vehicle to begin deceleration when following a slower-moving vehicle.

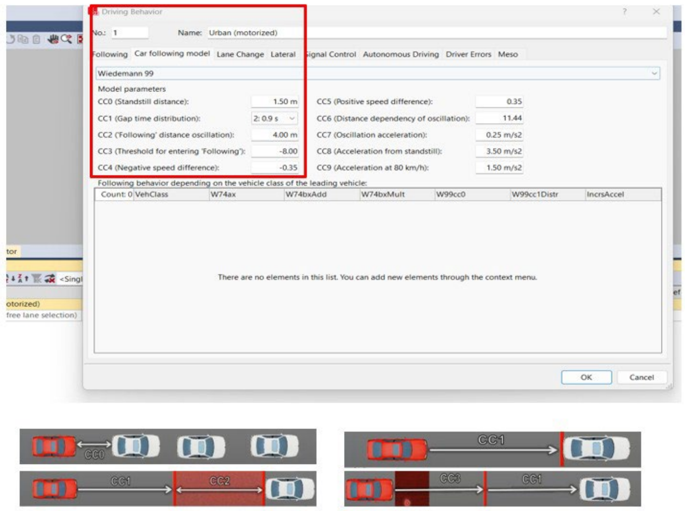


Figure 4-6: VISSIM interface showing the Wiedemann 99 model selected, and fine-tuning parameters highlighted (CC0-CC4).

Step 5: CC4-CC9 are related to acceleration/deacceleration during the oscillation phase.

1. CC4 (Negative speed difference) and CC5 (Positive speed difference) are parameters that deal with the relative speed difference between the following and leading vehicles. The default values of CC4 and CC5 are similar, but with opposite signs, and represent the lower and upper thresholds for the speed difference. Lower values of CC4 and CC5 result in more sensitive driving behavior. Please see Fig. 4-7.
2. CC6 (Distance dependency of oscillation) determines the speed oscillation based on the distance between the two vehicles. A value of 0 means that the distance has no impact on the speed oscillation, resulting in a constant speed with distance. Higher values of CC6 indicate a greater speed oscillation with increasing distance.

The following CC7, CC8, and CC9 refer mostly to the parameters that affect acceleration.

3. CC7 (oscillation acceleration): This parameter specifies the acceleration used during the oscillation phase. The default value is 0.3- 25 meters/s^2.
4. CC8 (Acceleration from Standstill): This parameter determines the desired speed when a vehicle starts from a standstill position.
5. CC9 (Acceleration at 80 km/h): This parameter specifies the acceleration at a speed of 80 km/h or 50 mi/hr. The software will automatically interpolate acceleration values for speeds below 80 km/h, but for speeds exceeding 80 km/hr., a constant acceleration will be applied as provided by this parameter.

Additionally, the software provides a feature that allows users to adjust the following behavior based on the vehicle class of the leading vehicle. This can be accessed through a dialogue box located below the aforementioned parameters. By utilizing this feature, users can simulate the behavior of a passenger car following an 18-wheeler truck or any other combination of vehicle classes. To do so, simply add the relevant vehicle class and adjust the parameters accordingly. This can greatly enhance the accuracy and realism of traffic simulation models.

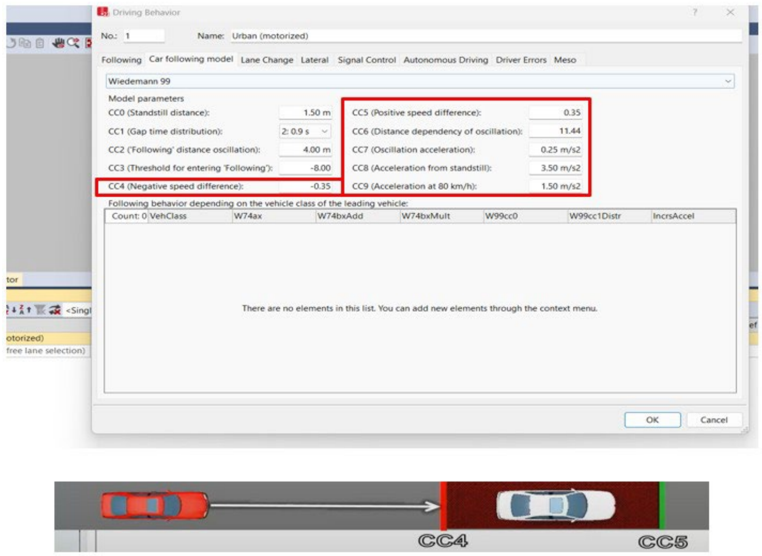


Figure 4-7: VISSIM interface showing the Wiedemann 99 model selected, and fine-tuning parameters highlighted (CC4-CC9).

4.3.2. Lane Change Behaviors in VISSIM

To configure the lane change behavior, we can use the "Lane Change" tab. It is important to understand the different types of lane change behavior before fine-tuning the parameters in this tab.

1. Necessary Lane change: This occurs when a vehicle needs to change lanes to follow a routing decision. For example, if a vehicle is in the leftmost lane and needs to turn right at an intersection, it will change lanes to the right to follow the routing decision. Please see Fig. 4-8.
2. Free Lane change: This occurs when a vehicle changes lanes to improve its driving experience, such as when the leading vehicle is driving slower. Unlike necessary lane change, free lane change is not governed by a routing decision. In either type of lane change behavior, there must be a suitable gap in traffic for the lane change to occur.

In VISSIM, all of the lane change parameters can be done in the “lane change” tab.

Step 1: First, the general behavior has to be selected. In the general behavior, two options are provided:

1. Free lane change: As previously mentioned, the free lane selection (default) allows the vehicle to travel in any lane.
2. Slow lane rule: If this option is selected, the vehicle traveling at a slower speed will remain in the right lane, while the vehicle with a higher speed will remain in the left lane, similar to a freeway. This option is mostly chosen if the model is intended for freeway design. For arterial network design, Free Lane change is the preferred option.

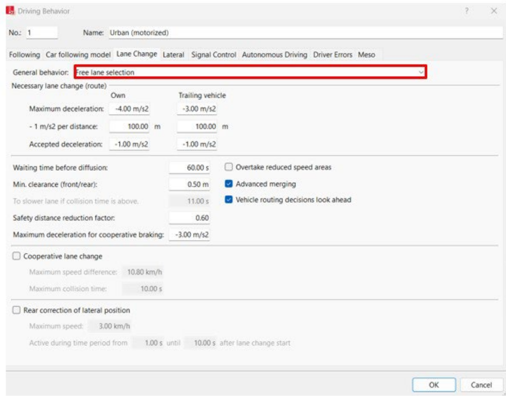


Figure 4-8: Snapshot of VISSIM interface for lane change tab showing general behavior.

Step 2: The necessary lane change (route) is configured immediately after the general behavior. This tab has two columns, one for the following vehicle (the vehicle that is changing lanes) and the other for the trailing vehicle (Leading Vehicle).

1. Maximum deceleration: This parameter determines how quickly the vehicle will slow down when it reaches the point of lane change, which is typically at a designated emergency stopping distance on a connector. Please see Fig. 4-9.
2. -1 m/s^2 (ft/s^2) per distance: This parameter specifies the rate at which deceleration increases over a given distance. For example, the default value of 100 ft (100 m) means that the deceleration rate will increase by 1 unit for every 100 ft (100 m) of distance traveled.
3. Accepted deceleration: This parameter sets the lower limit for the deceleration rate. In contrast, maximum deceleration represents the upper limit.

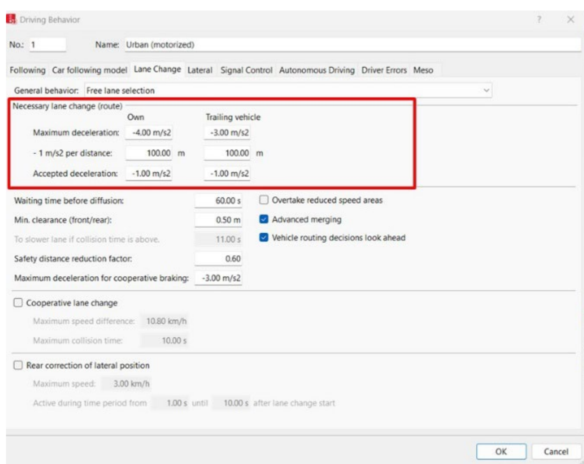


Figure 4-9: Snapshot of VISSIM with the 'Necessary Lane Change' feature.

Step 3: After adjusting the acceleration parameters, additional parameters are tuned for merging decisions and safety distance. These include:

1. Waiting time before diffusion: The default value for this parameter is 60.00 seconds, which means that the vehicle will wait until the given time has passed before disappearing from

the network with a warning sign displayed at the end of the simulation. A more realistic approach is to set this parameter to 999,999 (infinite) to ensure that no vehicle is removed from the network. Please see Fig. 4-10.

2. Minimum clearance (front/rear): This parameter specifies the minimum distance that must be maintained between two vehicles during a lane change. The minimum clearance only applies during the lane change.
3. Safety distance reduction factor: During a lane change, the safety distance is reduced by a certain factor, which is restored after the necessary lane change is completed. A value of 1 for this parameter means that there will be no reduction in safety distance, while a value lower than 1 will ensure that the safety distance is reduced by that factor.
4. Maximum deceleration for cooperative braking: This parameter specifies the maximum deceleration that can be applied during cooperative braking.

Several additional functions can be checked based on desired scenarios:

1. Overtake Reduced Speed Areas are usually unchecked, meaning that reduced speed areas are considered in the model. However, if this is checked, reduced speed areas will not be taken into account.
2. Advance Merging is an effective parameter that should be enabled to allow vehicles to make necessary lane changes earlier.
3. Vehicle Routing Decision Look Ahead allows vehicles to identify downstream routes earlier, which is useful for long corridors of the road with multiple routing decisions, especially for arterials where multiple routes are provided from one intersection to another.

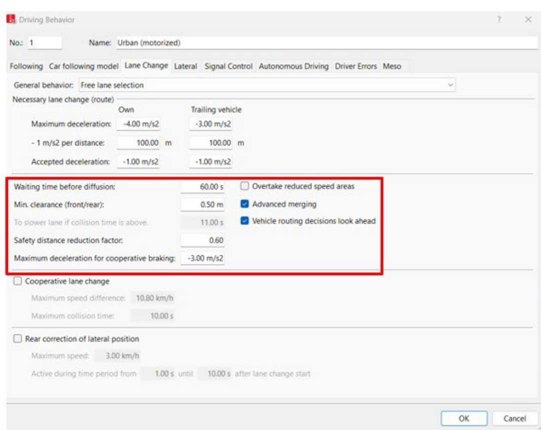


Figure 4-10: Snapshot of VISSIM interface with additional parameters for fine-tuning.

Step 4: The last option is the cooperative lane change, which involves setting the parameters for maximum speed difference and maximum collision time. If this option is enabled, vehicles can cooperate to change lanes, particularly in weaving situations. As shown in Fig. 4-11, if the trailing vehicle (A) observes that the leading vehicle (B) is attempting to change lanes and move towards its lane, it may change lanes or switch to another lane to facilitate the lane change of vehicle B, given that there is enough space, and the speed difference is acceptable. It is important to note that this parameter may not be effective during congested conditions (Jam density) as there may not be enough space for trailing vehicles to move to another lane.

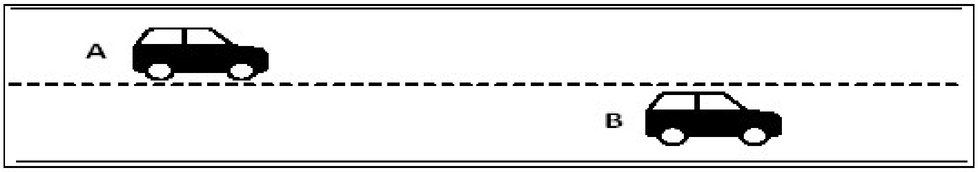


Figure 4-11: Cooperative Lane changes behavior between two vehicles.

Step 5: After completing the driving behavior, the next step is to create the link behavior type and assign the specific driving behavior type to it. To do so, navigate to "Base Data" and select "Link Behavior". Then, create the link behavior by clicking the "+" sign and giving it a suitable name. Next, assign the driving behavior that was created earlier and assign the required vehicle class. It is possible to assign different driving behaviors to different vehicle classes, depending on the design criteria. Please see Fig. 4-12.

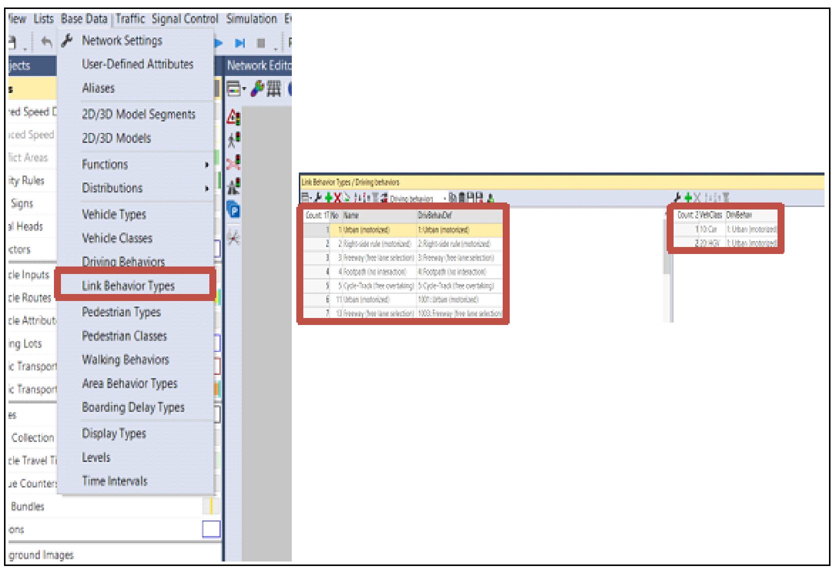


Figure 4-12: VISSIM tutorial snapshot on selecting link behavior types.

After completing the car following and lane changing behavior steps as outlined earlier, the next step is to apply these driving behaviors to the link. This can be achieved in two ways:

1. By selecting each link in the model and manually changing the Lane behavior type one at a time, as shown in Fig. 4-13 (B) below.
2. Selecting the list view of all the links and changing them all at once, as shown in Fig. 4-13 (C) below.

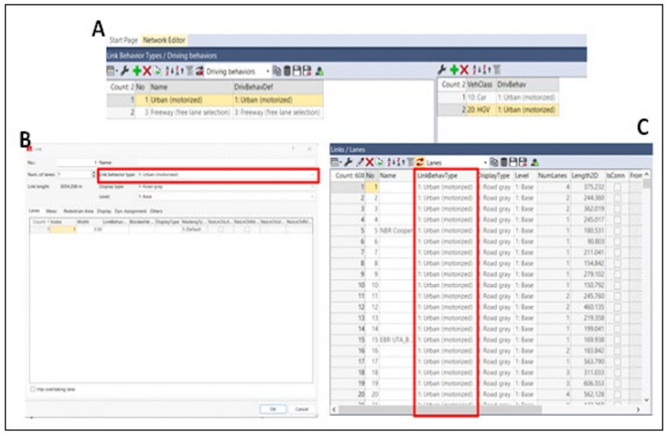


Figure 4-13: Snapshots showing how to apply link behavior types to a link in VISSIM.

4.3.3. Parameters of Driving Behaviors in VISSIM

We adjust and refine all the numerical values that have been previously discussed in the Driving Behavior Parameters section. Fig. 4-14 shows the recommended values for all the calibration parameters recommended by the Florida Department of Transportation Traffic Analysis Handbook ("Traffic Analysis Handbook Florida Department of Transportation," 2014). To further enhance our model, we fine-tuned the speed distribution and acceleration/deceleration function in addition to the calibration parameters discussed earlier. One way to achieve this is by using a dataset such as the Connected Vehicle dataset (Wejo) to collect vehicle trajectory information and corresponding speed data. This enables us to determine the actual speed distribution of a given road link and use it to improve the accuracy of our VISSIM modeling.

Calibration Parameter	Default Value	Suggested Range	
		Basic Segment	Weaving/Merge/ Diverge
Freeway Car Following (Wiedemann 99)			
CC0 Standstill distance	4.92 ft	>4.00 ft	>4.92 ft
CC1 Headway time	0.9 s	0.70 to 3.00 s	0.9 to 3.0s
CC2 'Following' variation	13.12 ft	6.56 to 22.97 ft	13.12 to 39.37ft
CC3 Threshold for entering 'following'	-8	use default	
CC4 Negative 'following' threshold	-0.35	use default	
CC5 Positive 'following' threshold	0.35	use default	
CC6 Speed Dependency of oscillation	11.44	use default	
CC7 Oscillation acceleration	0.82 ft/s^2	use default	
CC8 Standstill acceleration	11.48 ft/s^2	use default	
CC9 Acceleration at 50 mph	4.92 ft/s^2	use default	
Arterial Car Following (Wiedemann 74)			
Average standstill distance	6.56 ft	>3.28 ft	
Additive part of safety distance	2.00	1 to 3.5[i]	
Multiplicative part of safety distance	3.00	2.00 to 4.500[i]	
Lane Change			
Maximum deceleration	-13.12 ft/s^2 (Own) -9.84 ft/s^2 (Trail)	< -12 ft/s^2 < -8 ft/s^2	
-1 ft/s^2 per distance	200 ft (Freeway) 100 ft (Arterial)	>100 ft >50 ft	
Accepted deceleration	-3.28 ft/s^2 (Own) -1.64 ft/s^2 (Trail)	<-2.5 ft/s^2 <-1.5 ft/s^2	
Waiting time before diffusion	60 s	Use default	
Min. headway (front/rear)	1.64 ft	1.5 to 6 ft	
Safety distance reduction factor	0.6	0.1 to 0.9	
Max. dec. for cooperative braking	-9.84 ft/s^2	-32.2 to -3 ft/s^2	
Overtake reduced speed areas	Depends on field observations		
Advanced Merging	checked		
Emergency stop	16.4 ft	Depends on field observations	
Lane change	656.2 ft	>656.2 feet	
Reduction factor for changing lanes before signal	0.6	default	
Cooperative lane change	Unchecked	Checked especially for freeway merge/diverge areas	

Figure 4-14: Calibration parameters default value and recommended values (source: Florida DOT).

4.3.3.1. Speed Distribution Configuration

Speed distribution pertains to the statistical representation of the speed of simulated vehicles that are moving along a given roadway. Using the trajectory data of each vehicle taken from the Wejo data set, one can determine the speed distribution on a particular link. To compute the speed distribution near an intersection, a lengthy segment is chosen, allowing vehicles to reach the posted speed limit and move freely for a certain distance. Statistical techniques are then utilized to obtain the speed distribution, as illustrated in Fig. 4-15.

Bin	Frequency	Cumulative %
0	0	0.00%
5	14	2.89%
10	7	4.33%
15	12	6.80%
20	7	8.25%
25	14	11.13%
30	21	15.46%
35	67	29.28%
40	123	54.64%
45	165	88.66%
50	40	96.91%
55	11	99.18%
60	3	99.79%
65	1	100.00%
More	0	100.00%

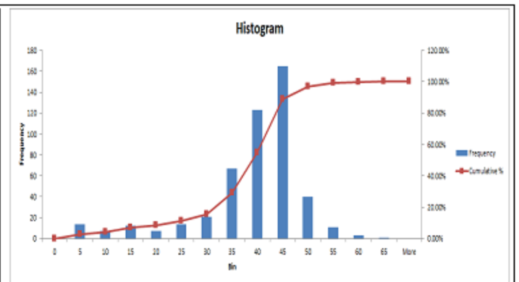


Figure 4-15: Speed Distribution Data and Histogram.

After obtaining the desired speed distribution for a specific link, that information can be applied to the same link in the simulation network model. To do so, the given steps should be followed:

1. Navigate to the "Base Data" tab and select "Distribution", followed by "Desired Speed" (shown in Fig. 4-16A).
2. Create a new speed distribution using the "+" sign (shown in Fig. 4-16B)
3. Copy the histogram data obtained from the measured "Speed distribution" and paste it onto the speed distribution in VISSIM (shown in Fig. 4-16B).

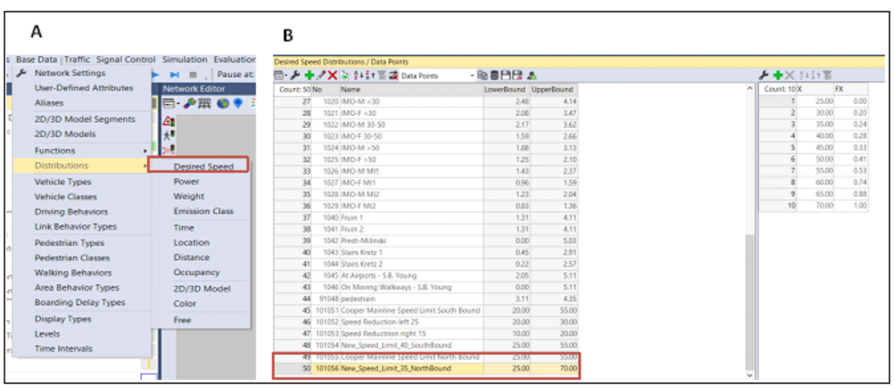


Figure 4-16: Snapshot of VISSIM interface showing how to create a speed distribution.

4. When editing the speed distribution in VISSIM, it is important to set the lower bound 10 miles less than the posted speed limit, but the upper bound can be set to the maximum speed that has been measured for vehicles traveling on the link. It is also important to verify

that the speed distribution in VISSIM is like the one obtained from the connected vehicle dataset. Please see Fig. 4-17.

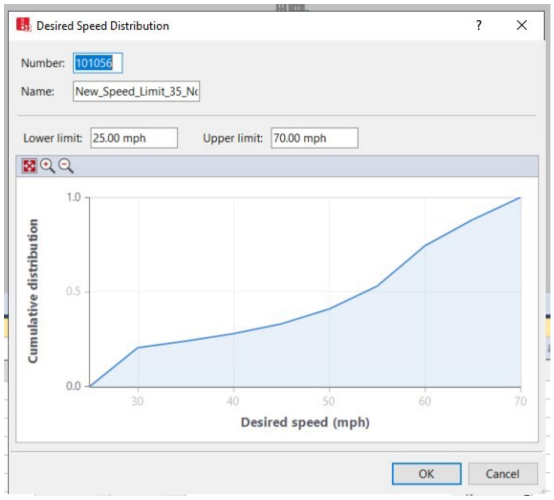


Figure 4-17: Desired Speed Distribution graph in VISSIM.

4.3.3.2. Desired Acceleration/Deacceleration: Configuration

The acceleration and deceleration functions are represented by a curve that plots speed (in mph) against acceleration/deceleration (in ft/s2) on the X and Y axes, respectively. The minimum and maximum acceleration must also be obtained from the actual dataset. Based on the speed, the average, minimum, and maximum accelerations can be calculated. The 25th percentile is used to obtain the minimum acceleration/deceleration, and the 95th percentile is used to obtain the maximum acceleration/deceleration. The graphs of the acceleration functions and the corresponding dataset are illustrated in Fig. 4-18.

Speed_range	Average_Acceleration	Lower_bound_acceleration	Upper_bound_acceleration
5	0.92	0.26	1.57
10	1.92	1.31	2.53
15	2.36	0.28	4.43
20	2.74	0.52	4.96
25	2.19	0.52	3.86
30	2.02	0.26	3.78
35	1.57	0.26	2.88
40	1.37	0.24	2.51
45	0.96	0.26	1.66
50	0.39	0.27	0.52
55	3.93	3.93	3.93

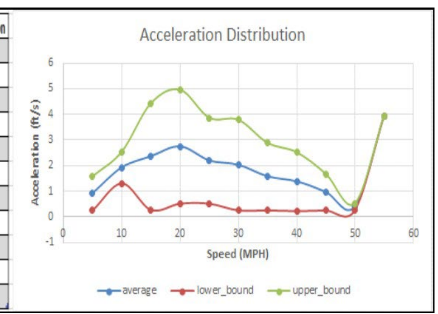


Figure 4-18: Average Acceleration Data and Acceleration function graph.

The process of configuring the acceleration and deceleration function in VISSIM is similar to that of speed distribution. To do this, go to the "Base Data" tab and select "Desired Acceleration" (shown in Fig. 4-19). It should be noted that this function is different for each vehicle class. Therefore, the desired acceleration/deceleration function should be fine-tuned separately based on the data. It is important to keep in mind that the measured acceleration/deceleration function graph may not be completely smooth, so some empirical adjustments may be necessary to smooth out the numbers.

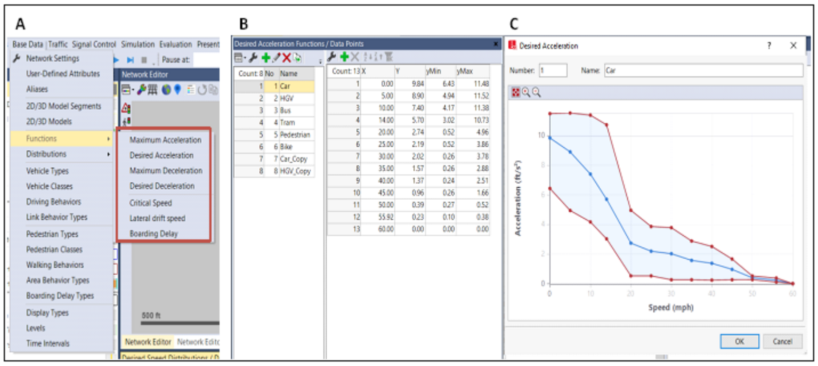


Figure 4-19: Average Acceleration Data and Acceleration function graph in VISSIM

4.4. MODELING AND CALIBRATING THE SIMULATION MODEL OF COOPER STREET IN ARLINGTON, TEXAS

Cooper Street was approved by the TxDOT to be used for this research. The simulated network was designed to cover the TxDOT-owned road segment from Division Road & Cooper Street to Cooper Street at the I-20 Interchange. The long arterial was divided into three different zones, namely Zone 1, Zone 2, and Zone 3. Zone 1 covers the intersection from Division Road to Mitchell Street, Zone 2 covers the intersections from Park Row Dr to Pioneer Pkwy and Zone 3 covers the intersections from Pioneer Pkwy to Interstate 20 Interchange. There are a total of 16 intersections, and we used the Bing Map background as the georeferences. After completing the network modeling, various additional data were incorporated to calibrate the model to match real-time scenarios. Some of the data utilized to create a base model are listed below:

1. Historical Volume Data
2. Signal Timing data for each intersection.
3. Detector Layout data for each intersection
4. Connected Vehicle Wejo dataset for Speed/travel time/acceleration & deacceleration calibration.

The Cooper Street Corridor model was calibrated using the methods outlined in the Calibration section. To ensure that the simulation had enough vehicles at the beginning and end of the simulation time, the first and last 15 minutes of the volume data were duplicated and added to the initial and final phases of the simulation (warm-up and cool-down periods). The overall simulation time was set for 2 hours and 30 minutes but the actual time when results were recorded was just 2 hours, excluding the initial and final 15 minutes. Data was recorded every 5 minutes or every 300 seconds, and the model evaluation was configured to begin data recording after 15 minutes and end 15 minutes before the end of the simulation. Some warnings were generated during the simulation, but they did not prevent the simulation from being completed.

Upon completion of the simulation, warning notifications may appear. If an error occurs, the simulation will not run, while warnings will allow the simulation to be completed with warning notifications. Upon completion of the model, the following warnings were displayed as shown in Fig. 4-20.

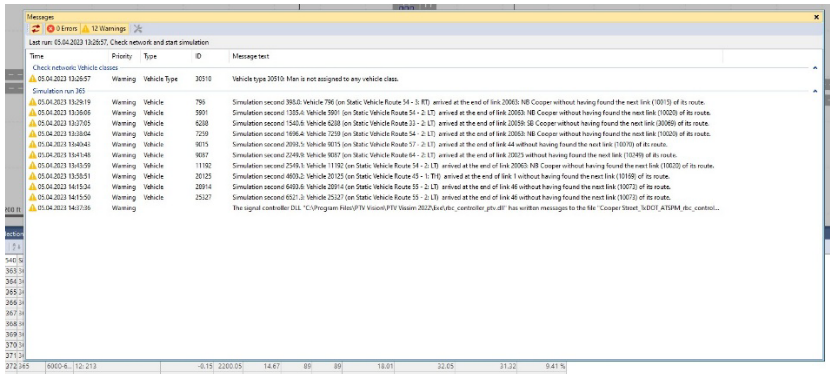


Figure 4-20: VISSIM simulation warning message.

From the snapshot, it can be observed that several warnings were generated after the completion of the simulation. Most of these warnings indicated that the vehicles arrived at the end of the Pioneer & Cooper intersection link without finding the next link. This is a common warning in VISSIM when the distance for the vehicle to take turns is too short for it to accept certain routing decisions. The Arkansas Ln and Pioneer Pkwy intersections are too close to each other, making it difficult for vehicles to make routing decisions at higher speeds. However, not many warnings regarding this were observed, indicating that the number of warnings is relatively low compared to the overall number of vehicles in the network. Therefore, specific actions to solve this problem are not required. Despite optimizing all possible values, these warnings may still occur, suggesting the possibility of bottlenecks around these intersections.

4.4.1. Volume Calibration

The initial step in verifying the accuracy of the model output is to compare the simulated volume with the adjusted input volume. The simulation was run for 2.5 hours with multiple runs, and the resulting volume was examined. To check if the simulated results were accurate enough, two intersections, UTA BLVD, and Park Row, were chosen for verification. Since vehicle input was only provided at the beginning of the Cooper Street link and the in-between volumes were adjusted based on routing decisions and inbound and outbound vehicles, analyzing two independent intersections that are far apart ensures that the model generates correct outputs for vehicle volumes.

Table 4-1 displays the simulation results for all directions at the UTA Blvd and Cooper Street intersection. The Table 4-2 presents the adjusted vehicle counts based on the actual counts for the same intersection. To assess whether the adjusted counts and simulated counts are acceptable, the GEH statistics method is often used. This formula is used in traffic modeling to compare two sets of traffic. In traffic modeling, a GEH value below 5.0 indicates a good match between the modeled and observed hourly volumes according to FDOT’s recommendations.

At the UTA BLVD intersection, a comparison between the simulated and adjusted volumes shows that the maximum and minimum values for GEH are 2.09 and 0.02 respectively. This means that the modeled volume is a good match for the adjusted volume, as the values are lower than 5.0.

Table 4-1: Calibrated Traffic Volume Simulation Output for UTA Blvd & Cooper Street.

START TIME	Southbound				Westbound				Northbound				Eastbound			
	S Cooper ST				**W Park Row DR**				**S Cooper ST**				**W Park Row DR**			
	RT	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**
7:15 AM	6	216	10	232	11	8	7	26	28	223	10	261	10	18	5	33
7:30 AM	6	207	11	224	4	13	16	33	33	336	17	386	7	27	11	45
7:45 AM	21	222	14	257	8	17	13	38	25	309	7	341	18	36	20	74
8:00 AM	16	207	14	237	5	14	10	29	47	442	19	508	10	30	7	47
8:15 AM	9	249	13	271	11	24	3	38	46	383	21	450	12	24	21	57
7:30 AM	9	199	12	220	4	23	11	38	48	320	15	383	5	12	14	31
7:45 AM	11	206	13	230	12	25	11	48	29	307	14	350	20	34	15	69
8:00 AM	22	234	24	280	7	14	10	31	43	439	19	501	13	27	8	48
8:15 AM	11	222	11	244	10	19	4	33	39	355	18	412	12	13	11	36
8:30 AM	21	215	2	238	11	29	5	45	36	254	20	310	20	26	19	65
Total	132	2177	124	2433	83	186	90	359	374	3368	160	3902	127	247	131	505
GEH	1.75	0.67	0.02	1.06	0.2	0.81	0.41	0.46	2.09	0.45	0.56	0.37	1.42	0.39	0.89	0.87

Table 4-2: Measured Traffic Volume (Adjusted) for UTA Blvd & Cooper Street.

START TIME	Southbound				Westbound				Northbound				Eastbound			
	S Cooper ST				**W Park Row DR**				**S Cooper ST**				**W Park Row DR**			
	RT	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**
7:15 AM	8	223	10	240	6	14	10	30	40	336	14	390	6	18	10	33
7:30 AM	8	223	10	240	6	14	10	30	40	336	14	390	6	18	10	33
7:45 AM	19	217	14	250	10	24	13	46	37	368	13	417	18	33	13	64
8:00 AM	19	217	14	250	5	11	8	24	54	369	16	439	10	33	10	53
8:15 AM	18	225	13	255	11	21	8	40	38	287	22	347	13	22	16	51
7:30 AM	8	223	10	240	6	14	10	30	40	336	14	390	6	18	10	33
7:45 AM	19	217	14	250	10	24	13	46	37	368	13	417	18	33	13	64
8:00 AM	19	217	14	250	5	11	8	24	54	369	16	439	10	33	10	53
8:15 AM	18	225	13	255	11	21	8	40	38	287	22	347	13	22	16	51
8:30 AM	18	225	13	255	11	21	8	40	38	287	22	347	13	22	16	51
Total	153	2208	124	2485	81	175	94	350	416	3342	167	3925	111	253	121	486

Likewise, at the Park Row intersection, the greatest value for GEH is 3.45, which is below 5.0. (Please see Table 4-3) Hence, it is considered to be a satisfactory fit. Both intersections are distant from each other and have several intersections in between them. This suggests that if both are accurately modeled independently, then the whole Cooper Street Corridor must have been properly modeled.

Table 4-3: Calibrated Traffic Volume Simulation Output for Park Row Dr & Cooper Street.

START TIME	Southbound				Westbound				Northbound				Eastbound			
	S Cooper ST				**W Park Row DR**				**S Cooper ST**				**W Park Row DR**			
	RT	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**
7:15 AM	17	195	20	232	30	59	42	131	30	338	24	392	62	62	13	137
7:30 AM	23	216	26	265	28	64	39	131	41	528	29	598	76	76	20	172
7:45 AM	33	227	34	294	29	76	62	167	35	437	12	484	71	71	28	170
8:00 AM	25	300	18	343	23	79	30	132	27	485	35	547	61	61	14	136
8:15 AM	37	187	40	264	39	73	26	138	33	515	8	556	80	80	20	180
7:30 AM	18	220	21	259	38	68	49	155	43	444	20	507	61	61	18	140
7:45 AM	17	247	37	301	24	54	68	146	37	437	16	490	76	76	28	180
8:00 AM	23	311	26	360	26	82	28	136	27	466	30	523	62	62	20	144
8:15 AM	20	205	17	242	35	84	28	147	51	472	18	541	78	78	21	177
8:30 AM	25	189	19	233	27	46	28	101	22	367	4	393	52	52	20	124
Total	237	2232	258	2727	299	685	400	1384	340	4499	190	5029	378	680	202	1260
GEH	1.11	0.05	0.69	0.48	1.42	3.15	0.13	2.83	3.17	2.07	3.17	3.45	1.61	0.7	0.74	0.65

Table 4-4: Measured Traffic Volume (Adjusted) for Park Row Dr & Cooper Street.

START TIME	Southbound				Westbound				Northbound				Eastbound			
	S Cooper ST				**W Park Row DR**				**S Cooper ST**				**W Park Row DR**			
	RT	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**	**RT**	**TH**	**LT**	**Total**
7:15 AM	18	192	16	226	34	72	38	143	52	480	36	568	34	66	14	115
7:30 AM	18	192	16	226	34	72	38	143	52	480	36	568	34	66	14	115
7:45 AM	23	279	38	340	32	72	54	158	39	537	18	594	29	61	21	111
8:00 AM	20	252	16	288	25	77	32	134	27	453	36	516	38	63	18	118
8:15 AM	27	199	30	256	36	86	38	159	38	406	7	451	38	84	23	145
7:30 AM	18	192	16	226	34	72	38	143	52	480	36	568	34	66	14	115
7:45 AM	23	279	38	340	32	72	54	158	39	537	18	594	29	61	21	111
8:00 AM	20	252	16	288	25	77	32	134	27	453	36	516	38	63	18	118
8:15 AM	27	199	30	256	36	86	38	159	38	406	7	451	38	84	23	145
8:30 AM	27	199	30	256	36	86	38	159	38	406	7	451	38	84	23	145
Total	220	2234	247	2702	324	770	397	1491	401	4639	236	5276	347	698	192	1237

4.4.2. Speed Calibration

Speed is an important calibration parameter, and it is determined by measuring the speed of vehicles in the connected vehicle dataset. Although the speed limit of Cooper Street is known, vehicles do not always follow the posted speed limit, so a speed distribution is used instead. The speed distribution is obtained from the Wejo dataset. To ensure that the simulated average speed is within ±10 mph of the actual measured speed, as recommended by FDOT, the average speed of vehicles is evaluated in three different segments of the road, since there is a change in speed limit from 35mph to 40mph. The "START" point is located near the UTA BLVD & Cooper intersection for southbound traffic, while the same point on the northbound is considered the "END." The "MIDDLE" point is the location where the actual change in speed limit occurs, which is near the Arkansas Ln & Cooper intersection. The "END" point is located at the intersection near the Interstate 20 interchange & Cooper for southbound traffic, and simultaneously the "START" point for northbound traffic. Fig. 4-21 below is the overall configuration.

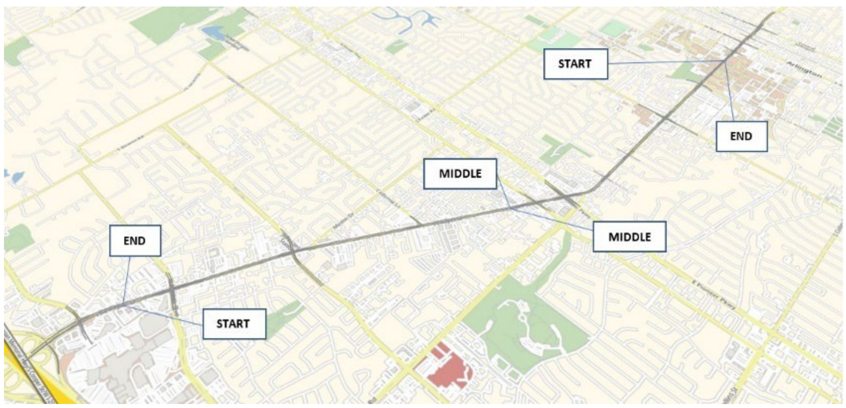


Figure 4-21: Location of Data collection point in Cooper Street Corridor.

Data collection point objects in VISSIM were used to record the average speed of vehicles passing through them. The recorded speeds have been presented in Table 4-5.

Table 4-5: Average and Harmonic reported speed from Calibrated VISSIM Simulation.

Location	Southbound (mph)		Northbound (mph)	
	Average Speed (ARTH)	**Average Speed (Harmonic)**	**Average Speed (ARTH)**	**Average Speed (Harmonic)**
Start	35.66	34.05	33.76	32.75
Middle	38.8	37.84	31.27	28.98
End	36.33	35.23	36.21	34.18

Table 4-6 displays the average arithmetic speed measured at the start, middle, and endpoints. To assess the accuracy of the simulation's speed measurements, we compare them to the actual measured speeds. If the difference between the measured average arithmetic speed and the average arithmetic speed obtained from the simulation is less than 10 mph, the simulation's speed measurements are deemed acceptable.

Table 4-6: Average measured speed reported Connected Vehicle Data.

From Wejo (Connected Vehicle data)		
Location	**Southbound**	**Northbound**
	Average Speed (ARTH)	**Average Speed (ARTH)**

Start	36.68	30.70
Middle	44.06	40.90
End	34.29	32.06

Table 4-7: Difference between measured average arithmetic speed and the average arithmetic speed obtained from the simulation.

Difference		
Location	**Southbound**	**Northbound**
	Average Speed (ARTH)	**Average Speed (ARTH)**
Start	1.02	3.06
Middle	5.25	9.63
End	2.04	4.15

The average speed of vehicles passing through a data collection point in VISSIM was recorded, and Table 4-7 shows the reported speeds and the difference between the measured average speed and the speed obtained from the simulation, which does not exceed 10 mph. Thus, the VISSIM model is considered calibrated for speed. The calibration for volume and speed is complete and both appear to be good. The next measure for calibration is travel time. For travel times less than 7 minutes, the calibrated time should be within a range of ±1 minutes compared to the actual travel time, and for travel times greater than 7 minutes, the calibrated time should be within a range of 15% compared to the actual travel time. Two different sources were used to obtain the travel time: Google Maps and Connected Vehicle (CV) data. The travel time from Google Maps for Cooper Street was 12 minutes, but it is not exact since it is already rounded off. CV data provided an average travel time of 533 seconds for northbound and 585 seconds for southbound. Table 4-8 shows the upper and lower bounds of the accepted travel time. The average travel time for northbound vehicles was 791 seconds, and for southbound vehicles, it was 686 seconds, both of which fall within the acceptable range of travel time. Thus, the calibrated model is accepted for the given parameters.

Table 4-8: Measured Travel Time from CV data & Google Maps.

Direction	**CV Travel Time**		**Google Travel Time**	
	Northbound (Seconds)	**Southbound (Seconds)**	**Northbound (Seconds)**	**Southbound (Seconds)**
Lower Bound	695	560	612	612
Upper Bound	940	758	828	828

The final stage of the calibration process involves examining the model visually for any abnormal driving behavior. The model shows some shock waves, and since it operates during peak hours, longer vehicle queues are observed. Some snapshots from the model are presented below for visualization purposes.

Figure 4-22: Visualization of simulation at Arkansas Ln & Pioneer Pkwy intersection.

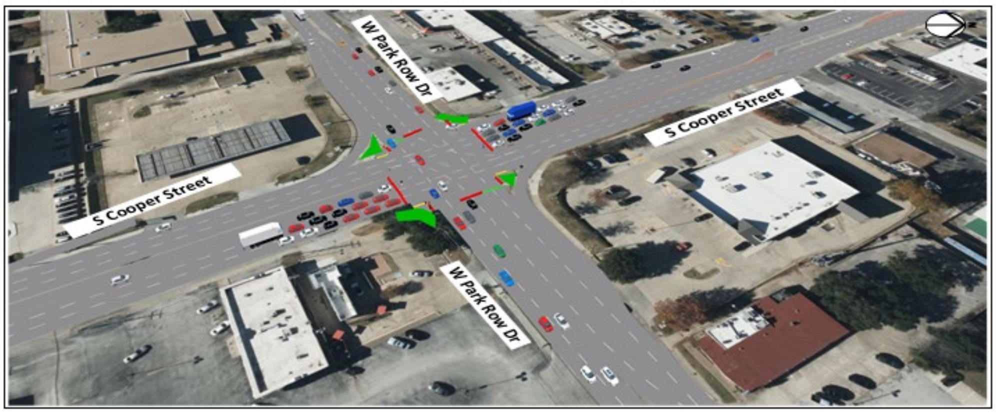


Figure 4-23: Visualization of simulation at Cooper & W Park Row Dr.

4.4.3. Limitations in Traffic Simulation Model

VISSIM modeling plays a critical role in providing valuable insights for planners and engineers regarding real-world scenarios. However, it is important to note that there are limitations to this study.

- The vehicle counts used in the study were obtained from 2020. The traffic volumes were impacted by COVID-19 and travel restrictions. To adjust for this, the counts were modified to reflect current traffic conditions. However, it was not feasible to access cameras at every intersection, and adjustments were made based on one intersection per zone. It was assumed that the turning ratios at each intersection are still the same, but this must be carefully examined for real signal retiming projects. As a result, the accuracy of the volumes may not be 100% as the adjustment factor may not be consistent across all intersections. Therefore, using queue length to calibrate the model may not be the optimal approach.
- One limitation of our modeling approach is that we do not recommend using intersection delay specific to the South Cooper Street Corridor due to a freight train passing through the Cooper and Main intersection, which we ignored in our VISSIM modeling. As a result, the intersection delay in the model may not match real-world data. Additionally, our comparison data was obtained from a CV dataset, which has a lower penetration rate of 3%-10% of total vehicles, making it difficult to accurately measure the speed reduction

factor during turning movements and resulting in less accurate intersection delay. Therefore, we did not consider intersection delay and queue length as calibration measures.

4.4.4. Model Validation

Once the calibration process is finished, the VISSIM model is executed, and the simulation results are documented. There are several methods of verifying the accuracy of the designed model. Table 4-9 outlines some of the calibration parameters. These methods of effectiveness have been obtained from the FDOT traffic analysis handbook ("Traffic Analysis Handbook Florida Department of Transportation," 2014).

Table 4-9: MOE for calibrating a model (Source: Florida DOT).

Calibration Measures	**Tools**	**Calibration Goal**
Traffic Volume	Node Analysis	Within 100 vph for a volume less than 700 vph Within 15 vph for volume between 700 vph and 2700 vph Within 400 vph for a volume greater than 2700 vph The sum of link volumes to have a GEH statistic value of 5 or lower.
Travel Time	Travel Time Measurement	For travel time lower than 7 minutes, Calibrated time should be in a range of ±1 minute compared to actual travel time
		For travel time higher than 7 minutes, Calibrated time should be in a range of 15% compared to actual travel time.
Speed	Data Collection Point	Simulated Average Speed to be within ±10 mph of actual measured speed.
Queue Length	Queue Counter	Simulated and field-measured queue length should be within 20%
Intersection Delay	Node Analysis	Simulated and field-measured link delay should be within 15% for more than 85% of cases
Visualization		Check for bottlenecks and unusual turns

The **GEH** statistics is a formula used in traffic modeling to compare two sets of traffic. For traffic Modeling work, a GEH value less than 5.0 is considered a good match between the modeled and observed hourly volumes ("Traffic Analysis Handbook Florida Department of Transportation," 2014).

$$GEH = \sqrt{2 * \frac{(M-C)^2}{(M+C)}} \tag{1}$$

Where:

- M = Simulated Volume
- C = Actual Counted Volume

Note: The GEH Statistic is a formula used in traffic engineering, traffic forecasting, and traffic modeling to compare two sets of traffic volumes.

4.5. CUSTOMIZING THE VISSIM MODEL TO OUTPUT SIMULATED HIGH-RESOLUTION TRAFFIC SIGNAL EVENTS AND VEHICLE TRAJECTORIES IN THE WGS84 FORMAT

The selected microscopic simulation software for this project, PTV's VISSIM has the capability of outputting traffic signal phase transition events, detector status, and vehicle movement records. Although such outputs share a similar format to the high-resolution traffic signal events for automated traffic signal performance metrics (ATSPM) and CV data like trajectories, the project team must overcome the inconsistency between VISSIM outputs and real-world data feeds for the ATSPMs.

The traffic signal event data for two ATSPM systems (Utah DOT's *ATSPM* and UTA's *UTA-In-Motion*) must comply with the event codes defined by INDOT which defined more than 100 traffic signal events in collaboration with Purdue University. As shown in Fig. 4-24 each event is allocated with a specific number. Traffic signal controllers capture many types of signal and detector state changes. For most traffic signal events, a secondary parameter is necessary to clarify the corresponding phase or detector channels as well.

Event Code	Event Descriptor	Parameter	Description
Active Phase Events:			
0	Phase On	Phase # (1-16)	Set when NEMA Phase On becomes active, either upon start of green or walk interval, whichever occurs first.
1	Phase Begin Green	Phase # (1-16)	Set when either solid or flashing green indication has begun. Do not set repeatedly during flashing operation.
2	Phase Check	Phase # (1-16)	Set when a conflicting call is registered against the active phase. (Marks beginning of MAX timing)
3	Phase Min Complete	Phase # (1-16)	Set when phase min timer expires.
4	Phase Gap Out	Phase # (1-16)	Set when phase gaps out, but may not necessarily occur upon phase termination. Event may be set multiple times within a single green under simultaneous gap out.
5	Phase Max Out	Phase # (1-16)	Set when phase MAX timer expires, but may not necessarily occur upon phase termination due to last car passage or other features.

Figure 4-24: Snapshot of Indiana traffic signal events logging enumerations.

In the real world, the central ATSPM system(s) will reach out to controllers in the field periodically, take away the raw data, decode, and then log the records into the ATSPM's database (MS SQL Server for UDOT-ATSPM and MySQL for UTAIM).

4.5.1. Simulated traffic signal transition events output:

It is relatively easy to directly store the simulated traffic signal group events from VISSIM to the SQL Server database, but all the simulated events must be parsed from their native form to the ATSPM-compliant format, as illustrated in Fig. 4-25.

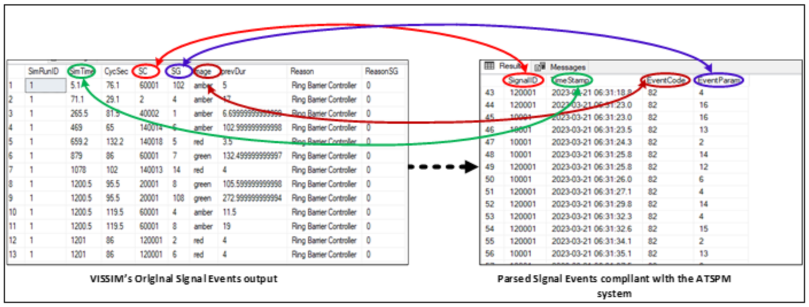


Figure 4-25: Comparison between PTV VISSIM's simulated signal transition events and ATSPM-compliant signal transition events.

4.5.2. Simulated detector events output:

As of VISSIM version 2022, the simulation engine cannot output the detector actuation events into a database but output into a text file with the extension name, "ldp". After each simulation run, VISSIM will automatically save the detector actuation events into a text file per intersection. The file names follow the format of "XXX_YY_ZZZ.ldp". XXX is the VISSIM model's name, YY is an intersection ID in the VISSIM model and ZZZ represents the simulation run number. In each file, the detector status (ON/OFF) at each time step is represented with a string. As shown in Fig. 4-26, "**.**" Represents an "OFF" status while other characters represent an "ON" status. The step-by-step detector status for each intersection will be read line by line and their status changes will be translated into ATSPM detector events.

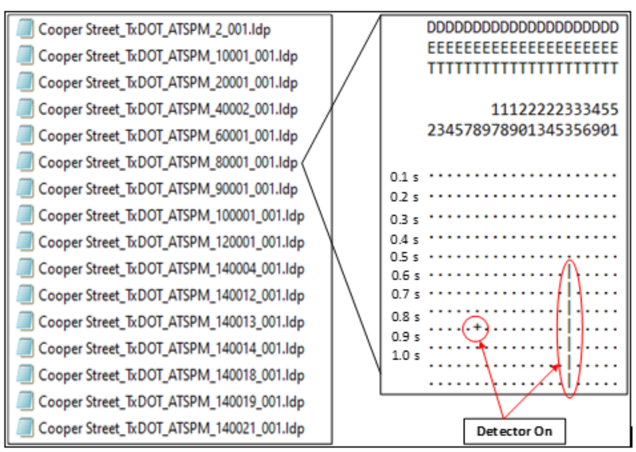
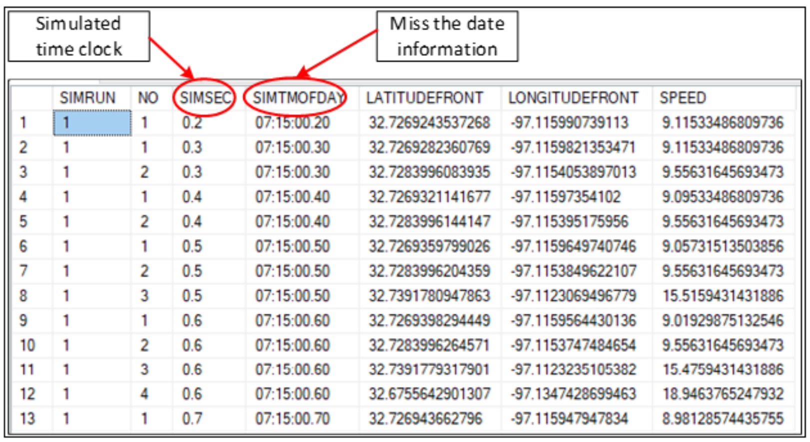


Figure 4-26: VISSIM's detector events output.

Vehicle trajectory output is a basic function in VISSIM. Nonetheless, the native vehicle trajectory output is a format like *(x, y, t)* where *x, and y* are a waypoint's relative positions from the reference point in the simulation model and *t* is the simulated time clock when the simulation is running. As a result, the output of simulated vehicle trajectories cannot be directly fed into the real-world ATSPM database. More geographic information is needed to make the simulated vehicle trajectories actionable.

The recent version of the VISSIM simulator (version 11 and later) provides a feature, referred to as "*User-Defined Attributes*" (UDA), allowing users to customize the simulation outputs. In addition, PTV provides an example of using the UDA feature to transform relative location (*x, y*) to the real-world WGS84 format (*latitude, longitude*) according to the model's geolocation references. It is also necessary to specify a real-world timestamp for each WGS waypoint. VISSIM can output both simulated time steps and simulated time-of-day timestamps. However, it would be necessary to specify a date as well to make the simulated vehicle trajectories compliant with the ATSPM requirements.



	SIMRUN	NO	SIMSEC	SIMTMOFDAY	LATITUDEFRONT	LONGITUDEFRONT	SPEED
1	1	1	0.2	07:15:00.20	32.7269243537268	-97.115990739113	9.11533486809736
2	1	1	0.3	07:15:00.30	32.7269282360769	-97.1159821353471	9.11533486809736
3	1	2	0.3	07:15:00.30	32.7283996083935	-97.1154053897013	9.55631645693473
4	1	1	0.4	07:15:00.40	32.7269321141677	-97.11597354102	9.09533486809736
5	1	2	0.4	07:15:00.40	32.7283996144147	-97.115395175956	9.55631645693473
6	1	1	0.5	07:15:00.50	32.7269359799026	-97.1159649740746	9.05731513503856
7	1	2	0.5	07:15:00.50	32.7283996204359	-97.1153849622107	9.55631645693473
8	1	3	0.5	07:15:00.50	32.7391780947863	-97.1123069496779	15.5159431431886
9	1	1	0.6	07:15:00.60	32.7269398294449	-97.1159564430136	9.01929875132546
10	1	2	0.6	07:15:00.60	32.7283996264571	-97.1153747484654	9.55631645693473
11	1	3	0.6	07:15:00.60	32.7391779317901	-97.1123235105382	15.4759431431886
12	1	4	0.6	07:15:00.60	32.6755642901307	-97.1347428699463	18.9463765247932
13	1	1	0.7	07:15:00.70	32.726943662796	-97.115947947834	8.98128574435755

Figure 4-27: Snapshot of simulated vehicle trajectories in the WGS 84 format.

Activating the simulation outputs for traffic high-resolution signal transition, detector actuation events, and vehicle trajectories.

4.5.3. Turn on high-resolution simulation output:

By default, VISSIM will not output the above results. Users must turn them on. This section summarizes the procedure of output simulated data for the ATSPM systems.

Step One: Import the user-defined attributes (UDA) features for the WGS84 vehicle trajectories using the enclosed VISSIM template. Please see Fig. 4-28.

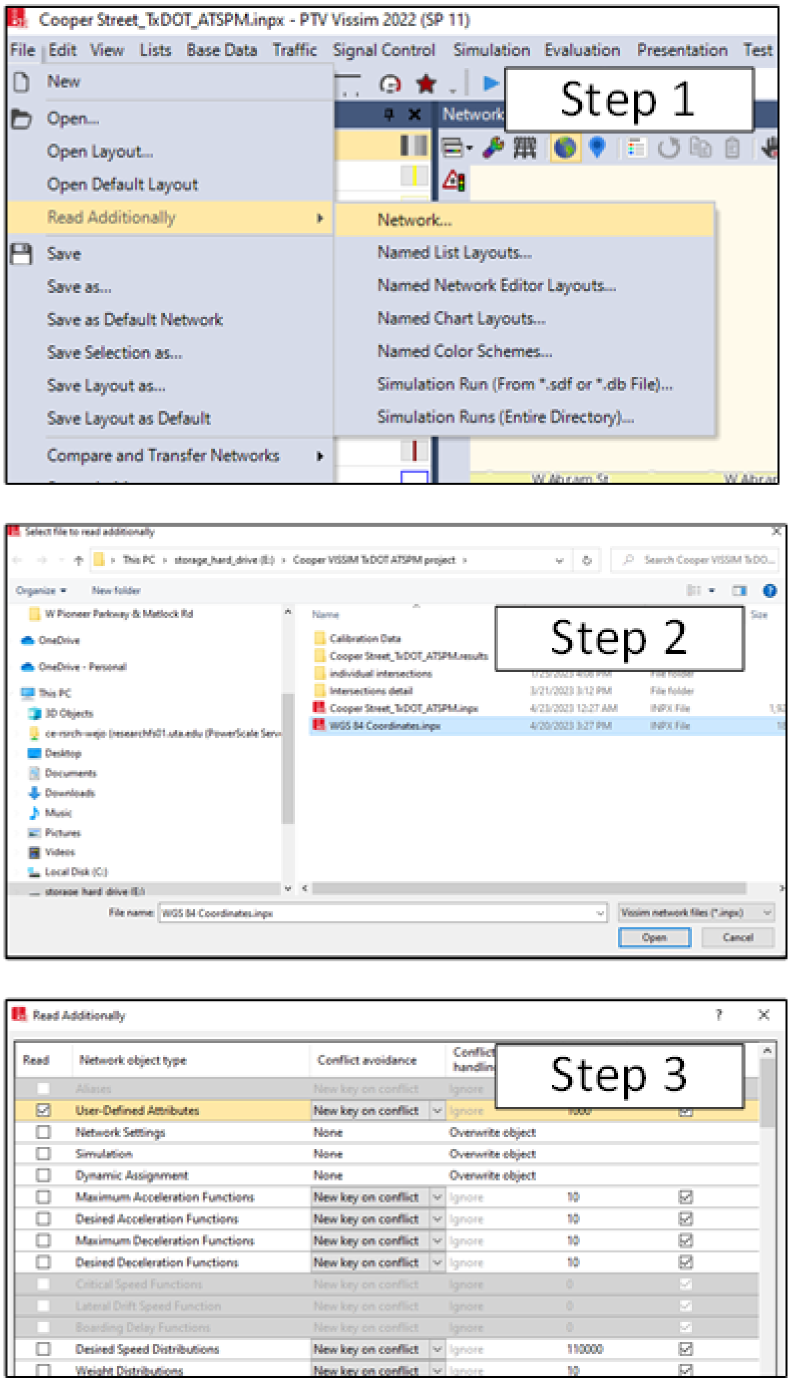


Figure 4-28: Loading the UDA for the WGS84 vehicle trajectory output.

Step Two: Turn on high-resolution traffic detector actuation storing for each intersection Fig. 4-29 shows the four sequential steps to turn on the detector actuation output.

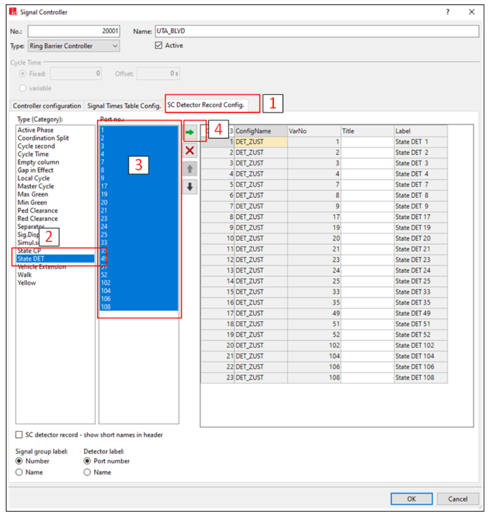


Figure 4-29: Turn on detector actuation outputs for each intersection.

Step Three: Output Database Configuration. VISSIM provides the capability of storing simulation results in various databases. Since the UDOT-ATSPM system uses the Microsoft SQL Server database, it is convenient to output the simulation results into the SQL server data as well. Users can either create a new database in the existing SQL Server or download/install a free community version of SQL Server. It is recommended that installing/creating the SQL Server database should be done with assistance from the IT expert. Once the SQL Server database is successfully set up, users should first test if VISSIM can connect to the database to save the simulation outputs. Fig. 4-30 shows how to connect to an SQL Server database from VISSIM. In step 4, input the corresponding server name, username, password, and database name, and then click test. If VISSIM can access the database, then a message box will pop up to indicate the connection test is successful.

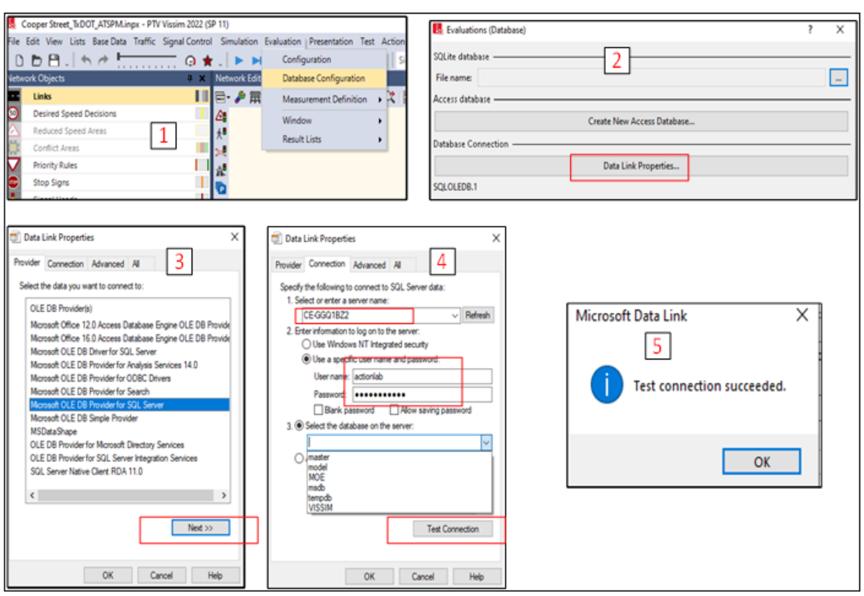


Figure 4-30: Connects VISSIM to a SQL server database to store the simulation output.

Step Four: Turn on the related simulation outputs. To turn on the simulation output. Users need to go to the main menu, then click Evaluation, and choose Configuration. As shown in Fig. 4-31. Users need to check three direct outputs to save them in the database. Then click the attributes of "vehicle record". The vehicle outputs should choose the attributes as shown in the third step.

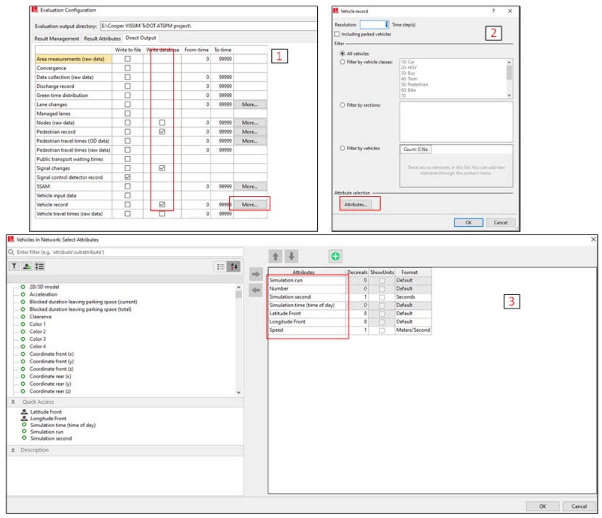


Figure 4-31: 8 Simulation output evaluation.

After the above four steps, VISSIM will automatically store the simulation outputs related to the ATSPM system into three tables in the SQL Server database containing the data of pedestrian behaviors. vehicle trajectories and high-resolution traffic signal events. The results of multiple simulation runs can be saved in the same database. The simulation output is ready to transform into ATSPM-compliant data records.

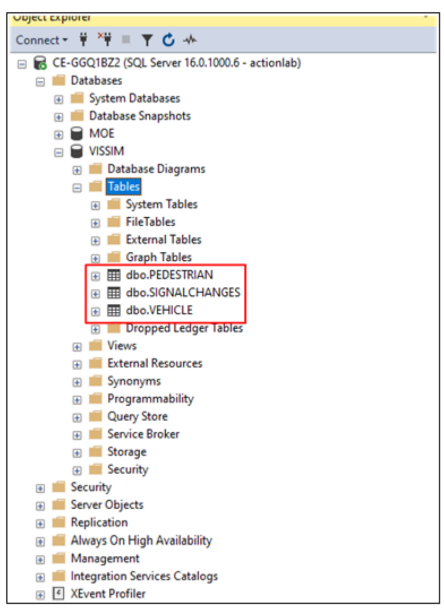


Figure 4-32: SQL Server database for VISSIM output.

CHAPTER 5: SOFTWARE DEVELOPMENT

5.1. Q-FREE'S MAXTIME SIGNAL SOFTWARE-IN-THE-LOOP (SILS) SIMULATION FOR THE ATSPM DATA GENERATION

The project team adopted two kinds of signal emulators in VISSIM: the default RBC controller in VISSIM, and the more advanced software-in-the-loop (SILS) signal emulation package provided by Q-free Inc. Using RBC and/or Q-free's SILS for the ATSPM-in-the-loop simulation framework will not add additional software cost to agencies, consultants, or academia to their projects. Setting up the default RBC controllers in VISSIM simulation models for ATSPM has been described in detail in TM-3. Therefore, in this TM, the focus is on how to set up a more advanced traffic signal emulator in VISSIM referred to as (signal control) software-in-the-loop simulation or SILS.

The concept of SILS in VISSIM became mature around 2010. PTV, the developer of VISSIM, and Econolite, a controller manufacturer based in California, decided to collaborate to integrate Econolite's flagship signal control software, ASC/3, into VISSIM. In this collaboration, Econolite's ASC/3 software receives the emulated detector actuation from a running VISSIM model and returns its control decision (e.g., each signal head's color) following the same control strategy deployed in the real world. The resulting traffic signal performance can be easily evaluated from VISSIM outputs. The SILS simulation technique is a big leap in traffic signal studies as it shortens the gap between traffic signal simulation and real-world implementation. Other than Econolite, other controller manufacturers also developed their own SILS kit on various traffic simulation platforms, such as the VISSIM SILS package provided by Q-free Inc. (Intelight, 2000), another major traffic controller manufacturer.

The SILS technique has been evolving since it was invented. One of the latest features is that it can record and output full-scale traffic signal event logs for the ATSPM systems. This feature allows the project team to develop a traffic simulation model to output most signal event logs for generating the ATSPM performance measures. As one example, the default RBC controller in VISSIM can output an event whenever a green signal phase terminates, but it cannot explain the reason such as "gap out" or "max out". By contrast, the advanced SILS technique can output traffic signal logs including both traffic signal transitions and reasons. The latest SILS module can output almost all the signal events used in the real-world ATSPM system(s).

Among all the off-the-shelf SILS packages in the market, the project team selected Q-free's SILS solution based on its control software, MAXTIME, and VISSIM. Q-free controllers are popular in Texas. Many state and local agencies are considering upgrading their traffic signal systems with Q-free controllers. Besides, this choice was made based on the following facts.

- Using the Q-free SILS package for VISSIM will not require additional software cost if the users, including all agencies, consultants, and academia, have purchased any Q-free traffic control product or are willing to sign an End-user license agreement (EULA) with the Q-free company. In practice, almost all users will be able to meet these requirements and so can use the Q-free's SILS for free. By contrast, other off-the-shelf SILS packages may cost upfront payment and/or annual license fees.
- The Q-free company is responsive to practitioners' requests and its control software MAXTIME is frequently updated. In the meantime, whenever the MAXTIME control software is updated, SILS will also be updated to the latest control software as well. Therefore, the Q-free's SILS package is relatively futureproof.

Note that, although the project team advocates the Q-free's SILS package for the ATSPM-in-the-loop simulation framework, the proposed framework in this project also supports other SILS packages, such as Econolite's SILS package. If a user wishes to use a specific SILS package (e.g., to match their existing controllers), the project team can include other SILS packages as well in the future.

5.2. ENABLING VISSIM TO OUTPUT VEHICLE TRAJECTORIES IN THE FORM OF WGS84

The default coordinates of simulated vehicles in VISSIM are the Cartesian coordinates relative to the reference point of the simulation network. They must be transformed to the real-world format of (latitude, and longitude), referred to as WGS84 (DMA, 1987) to become interpretable by other GIS or map systems. This need can be carried out in VISSIM with ease. In VISSIM's evaluation module, there is a feature called "User-Defined Attributes or UDA" which enables VISSIM to output customized results according to users' design. On top of this feature, the VISSIM's developer also provides an example model "WGS 84 Coordinates.inpx" to transform the simulated vehicle trajectories to the WGS84 format. The example model is in the accompanied example folders. Note that users need to adjust the directory according to where their VISSIM is installed. The pdf document in the same folder explains how to import the WGS84 UDA in detail.

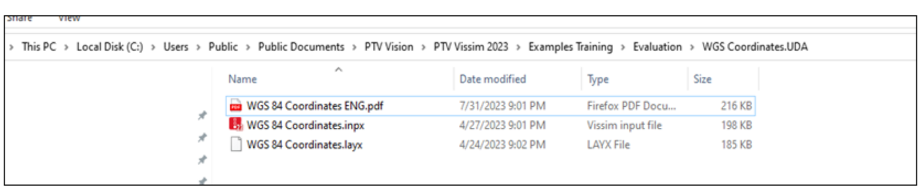


Figure 5-1: Location of the WGS 84 transforming model provided by PTV.

To output all or part of simulated vehicles' trajectories. A user needs to click "Evaluation" in the VISSIM menu then "Configuration". In the configuration windows, the user needs to follow the steps demonstrated in Fig. 5-2. Specifically, the user needs to click the "Direct Output" tag and click the "More" Button of "Vehicle Record", also making the "Write database" checkbox checked. Next, the user can choose one, multiple, or all vehicle classes to output vehicle

trajectories. In this case, a new "CV" vehicle class was created to represent 10% of all vehicles. CV vehicles are to simulate the connected vehicles in the real world and output their trajectories in the form of "(lat, Lon, t)". Finally, the user needs to click the "Attributes…" button to select which UDA attributes should be automatically output by VISSIM. In the example model, the UDAs include simulation time (0.1, 0.2, 0.3…), instantaneous latitude, longitude, speed, and vehicle type. The users can add more attributes according to their project needs in the future.
Note that the zero-starting simulation second is not recognizable by the realistic ATSPM systems. There is an option in VISSIM to specify the starting clock time, but it does not serve the purpose of this project. The target simulation starting date and time will be specified by the customized software while retrieving and parsing the simulated ATSPM and CV data set. To be actionable, the default simulation seconds will be transformed to the commonly used format of "epoch time" (the total seconds since midnight of January 1st, 1970) according to the specified starting time using the developed software tool for this project. Readers are suggested to read TM-3 to understand how to examine the simulated vehicle WGS84 trajectories in the database.

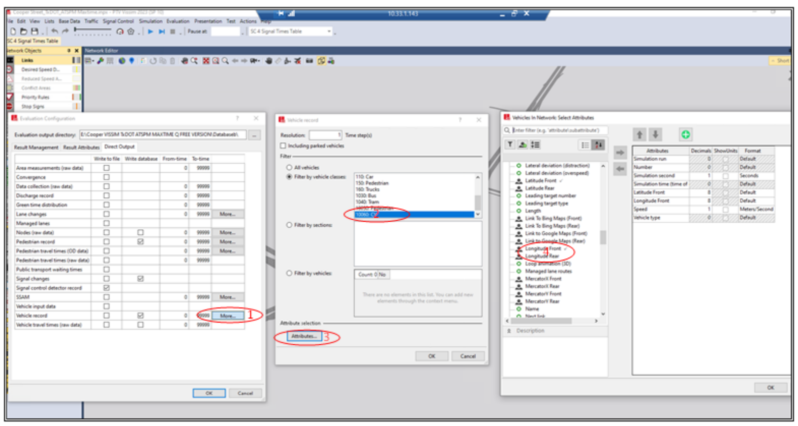


Figure 5-2: Output of the simulated vehicles' trajectories in the WGS84 format.

5.3. MANDATORY REQUIREMENTS FOR AUTOMATED MAXTIME LAUNCHING FOR MULTIPLE INTERSECTIONS.

To configure a MAXTIME SILS controller, users are required to perform multiple specifications, which can be time-consuming as well as prone to errors while covering multiple intersections. Hence, a special VB script[1] has been developed to automate the controller configuration and launching process. Certain specific criteria must be ensured for this script to function effectively. The guidelines for this process are outlined below:

1. In PTV VISSIM, the signal controller numbers should start from 1, increasing by 1 for each subsequent intersection. If a simulation model is already created with a different signal controller number, ensure to adjust the signal controller numbers, starting from 1 and increasing by 1 for each subsequent intersection. This can easily be done in VISSIM. Once the

[1] VBScript, is a scripting language developed by Microsoft. It's modeled on Visual Basic and is primarily used for automating repetitive inputs from keyboards and mouses in various Microsoft environments.

intersection ID is changed, then all the associated components like detectors will be automatically reconfigured.

2. The detector type must be set to "**Standard.**" Otherwise, MAXTIME SILS may wrongfully recognize the detector status.
3. When naming the user database in MAXTIME, the intersection name should include the intersection ID at the beginning, matching the VISSIM signal controller number. For instance, if an intersection is "**W Division St & Cooper St**" and its ID in VISSIM is **N**, and the MAXTIME database name should be "**N: W Division St & Cooper St.**" Repeat this for all intersections. (N=1,2, 3,….)
4. The MAXTIME control software outputs the ATSPM signal events through its populated web portal and XML server. The web port number of each MAXTIME controller will be automatically assigned by the VB Script. The last 1 or 2 digits of the web port number match the intersection ID in VISSIM. For instance, if the intersection IDs in VISSIM are from 1 to 5. Then the web port numbers of five MAXTIME control controllers will be assigned from 1001 to 1005 respectively[2]. If the last intersection is 18, then the web port number will be 1018. The VB script provides flexibility for users to adjust the initial base number within the range of 1000 to 9000 if necessary. Keep in mind that, after moving to new web port numbers, the XML server may still fail to populate. This could be caused by port occupancy by other programs. It is advisable to avoid using those known used web port numbers (e.g., 8000,8080, etc.).
5. Another crucial aspect is the PTV VISSIM TCP Port, the hidden coupling mechanism between VISSIM and all MAXTIME signal control emulators. The TCP ports also must follow the specified numbering rules strictly. The TCP port's last one or two digits must match the intersection's ID. In this project, The TCP ports are numbered as "**10000+X**" where X is the intersection's ID in VISSIM. For instance, if "**W Division St & Cooper St**" is the first intersection (Intersection ID is 1), then the TCP port number should be **10001**. For the last (the 18^{th}) intersections, the TCP port should be **10018.** Additionally, verify that the TCP port number in each intersection database corresponds between VISSIM and MAXTIME.

5.4. SOFTWARE-IN-LOOP SIMULATION CONFIGURATION WITH VISSIM

In TM-4, it is highlighted that a simple Ring-Barrier controller (RBC) is effective for basic scenarios, but for more complex situations including railroad preemptions and transit signal priority (TSP) and/or adaptive traffic signal controls, more advanced control emulators are necessary. The MAXTIME control software developed by Q-Free and its fully functioning emulator in Windows can be coupled with VISSIM. The following sections offer a step-by-step tutorial to set up the MAXTIME SILS in VISSIM at each intersection.

[2] If 1000+x port is occupied, then the VBScript will automatically move up and try to 2000+x, 3000+x, until 9000+x. This step is automated with no needed inputs from users.

5.4.1. Step-by-Step procedure for configuring VISSIM.

Before configuring the MAXTIME control software in VISSIM, users need to ensure that the VISSIM software is ready. Below are the steps to make sure the software is ready to work with the MAXTIME controller.

Step 1: Download the MAXTIME DLL file compatible with your PTV VISSIM version from https://support.intelight-its.com/maxtime-vissim-dll/. After logging in and verifying[3], download the following files:

A. MaxTime.dll

B. STDSC_GUI.dll

C. Maxtime.wtt

Step 2: Copy all the DLL/WTT files to the VISSIM.Exe folder on your computer. The typical path is C:\Program Files\PTV Vision\YOUR_VISSIM_VERSION\Exe.

Step 3: After placing the files in the appropriate location, select the target intersection in the VISSIM simulation software, and switch the signal controller type to "EXTERNAL."

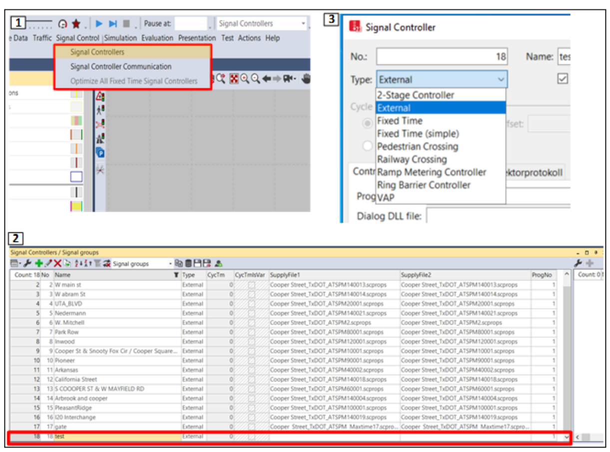


Figure 5-3: PTV VISSIM Signal Controller Interface Snapshot.

Step 4: When selecting the signal controller, a separate dialog box will open. Ensure that the program file and dialog DLL file have the correct file locations, which should be the files downloaded in step 1. For WTT files, click on the green "+" sign and add the downloaded WTT files.

[3] The account is privileged and will need to be approved on Q-free's discretion to be able to download software

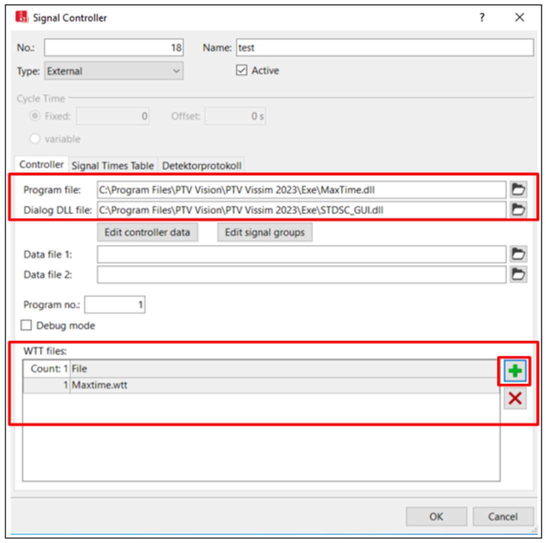


Figure 5-4: PTV VISSIM Signal Controller Dialog Box.

Step 5: Now, the signal controller is prepared for the user to add a signal group. **Keep in mind that in the MAXTIME SILS in VISSIM, a signal group refers to the traffic signal system's I/O (input-output) channel**[4]**, not a vehicle/pedestrian phase**. Therefore, the signal group numbers do not necessarily match the phase number in controllers because the larger I/O channels are often used to output status for pedestrian phases or overlaps that are also numbered as "pedestrian phase 1, 2, or overlap 1, 2". For example, I/O channels 1-8 can match vehicle phases 1-8 while I/O channels 9-12 (signal group 9-12 for the MAXTIME SILS) are often used to output the status of pedestrian phases (pedestrian phase 2, 4, 6, 8) and I/O channel 13-16 (signal group 13-16 for the MAXTIME SILS) are often used to output the status of overlap phases (overlap phase 1, 2, 3, 4). Therefore, users are required to bear in mind the concept of I/O channels rather than the control phases while modeling the MAXTIME SILS in VISSIM.

Starting with clicking on 'Edit Signal Group', a new dialogue box will be opened. By default, a specific TCP port number 1234 is provided as a standard signal control parameter. Ensure to change the port number to the required value (e.g., 10001 for intersection 1, 10018 for intersection 18). The same port number should be used when setting up the SILS MAXTIME later. Refer to Section 0.

[4] The concept of IO channel resides in the traffic signal cabinet, defining which signal head should be activated when the corresponding phase is on.

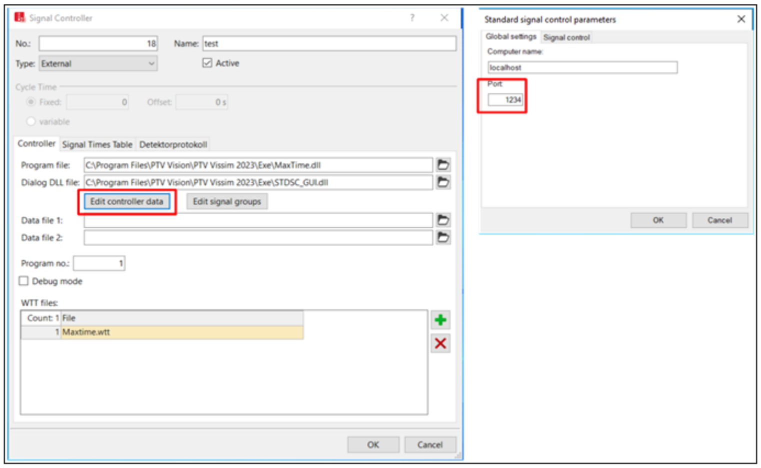


Figure 5-5: PTV VISSIM Standard Signal Controller Parameters Dialog Box.

Step 6: Now, navigate to the Signal Control tab and continuously add the signal control numbers. These numbers will correspond to the I/O channels received from the MAXTIME controller. In general, we have up to 16 I/O channels. Note that not all intersections will have all 16 channels; the number of channels to be added to the signal controller will depend on the virtual signal controller setup.

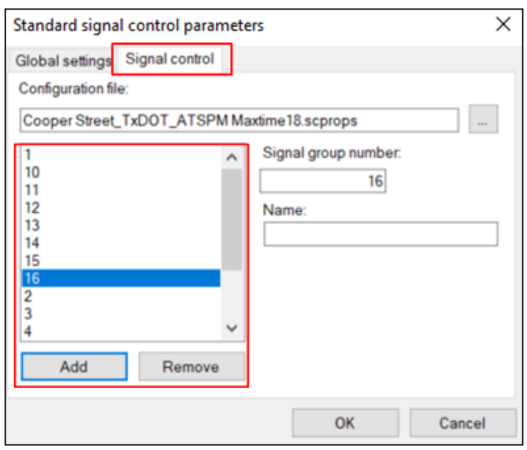


Figure 5-6: Standard Signal Controller Parameters Dialog Box in VISSIM with Signal Control Group Number Setup.

If all the steps shown above are done correctly, it should ensure that VISSIM is ready to couple with the virtual MAXTIME controller. Now, the MAXTIME controller should be set up correctly for proper communication between the controller and the simulation software. The following segment will guide the user to set up the Q-Free MAXTIME controller in VISSIM.

5.4.2. Step-by-step procedure for configuring the Q-Free MAXTIME controller.

Step 1: After installation, open the software in "Run as Administrator" mode to make and save edits. Navigate to the MAXTIME tab, as shown in Fig. 5-7.

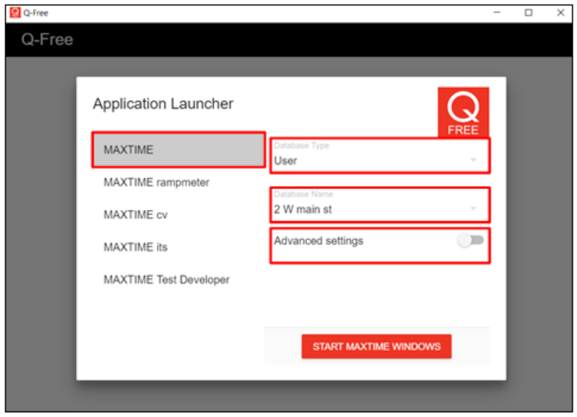


Figure 5-7: Q-Free MAXTIME Application Launcher Snapshot.

Step 2: Within the MAXTIME tab, options include Database type (User or factory), Database Name, and advanced settings. Opt for the USER database type to create a personalized signal timing database, enabling the input of project-specific information later.

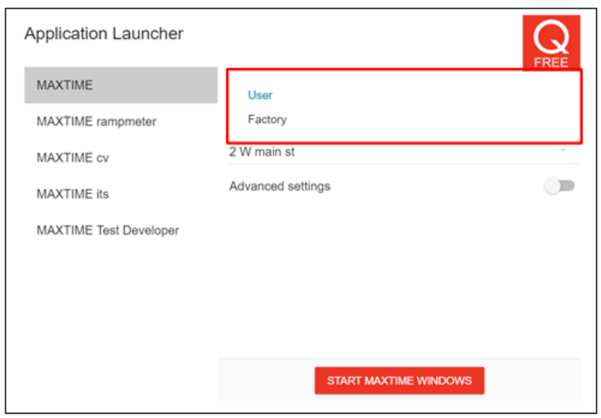


Figure 5-8: Highlighted Database Type in Q-Free MAXTIME Application Snapshot.

Fig. 5-9 displays a list of default factory settings.

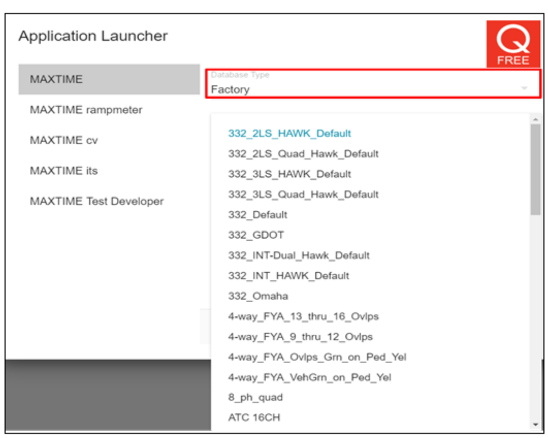


Figure 5-9: Snapshot Displaying List of Factory Setting Defaults in Q-Free MAXTIME.

In the USER database, names created by the user may initially differ. These names are changeable and should be numbered based on the numbering rule demonstrated in Section 0.

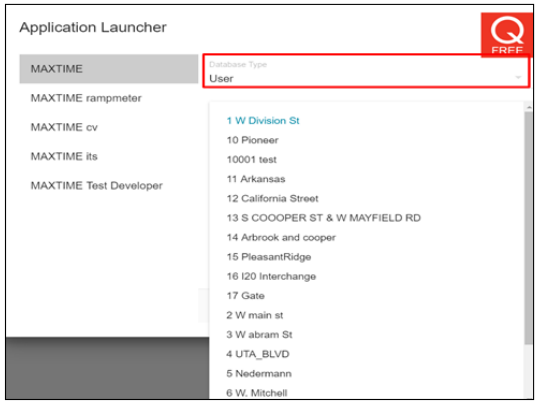


Figure 5-10: Snapshot Displaying List of User created Database in Q-Free MAXTIME.

Step 3: After selecting the desired database, users have two options:

A. Run the controller independently (i.e., not connected to VISSIM) by clicking "START MAXTIME WINDOWS."

B. Opt for advanced settings to connect the MAXTIME window to the desired simulation model. Advanced settings include the following options:
   a. web port.
   b. simulation speed
   c. simulation detector
   d. PTV Vissim IP
   e. PTV Vissim TCP Port
   f. PTV Vissim ID

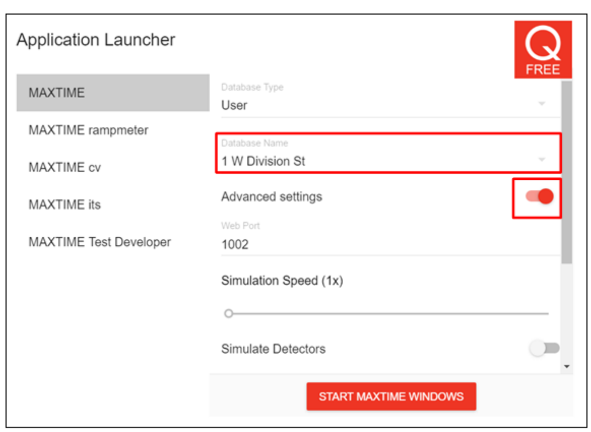


Figure 5-11: Q-Free MAXTIME Application Launcher with advanced settings on.

Step 4: If users choose not to select the advanced settings, it means they are independently running the controller and are not connected to any external simulation model. Once run, the MAXTIME software will prompt users to set up their profiles, select the mode, and configure the time.

A. First-time users see the Welcome page upon clicking "START MAXTIME WINDOWS." Access user profiles locally at C:\ProgramData\intelight\Profile.

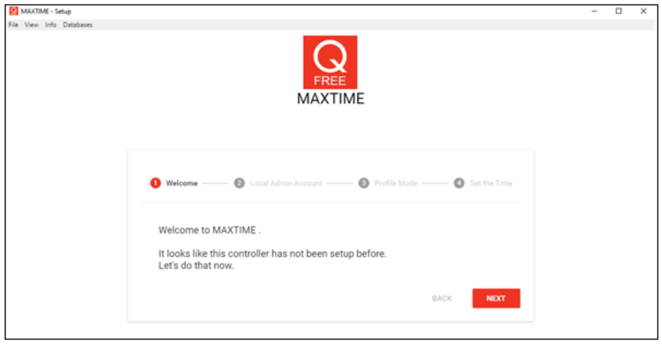


Figure 5-12: MAXTIME Welcome Page Display.

B. Move to the next page to set up accounts. MAXTIME offers three account types, and each user type has specific features, such as editing timing plans, managing databases, adding users, etc. (See Fig. 5-13)
   a. System Administrator
   b. User
   c. Guest Mode

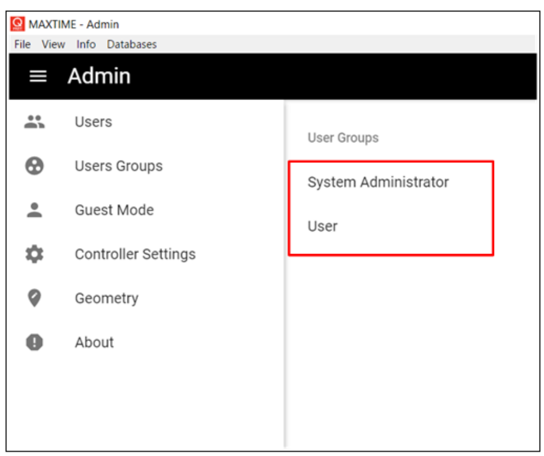


Figure 5-13: MAXTIME User Groups Display.

C. Users also have access to choose the profile mode:
   a. Controller Profile: For local setting Saving.
   b. Cloud profile: For remote settings saving on the cloud.

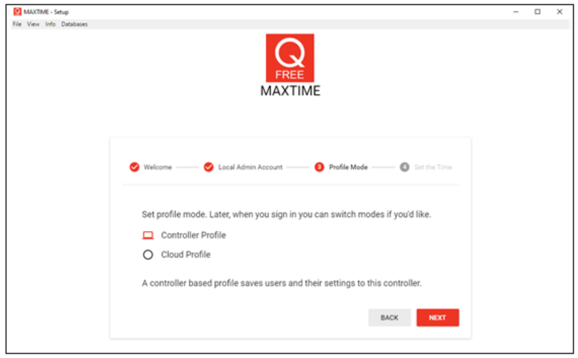


Figure 5-14: MAXTIME profile mode Display.

D. The final step is to set the controller's Date, Time, Time Zone, and Daylight-Saving settings.

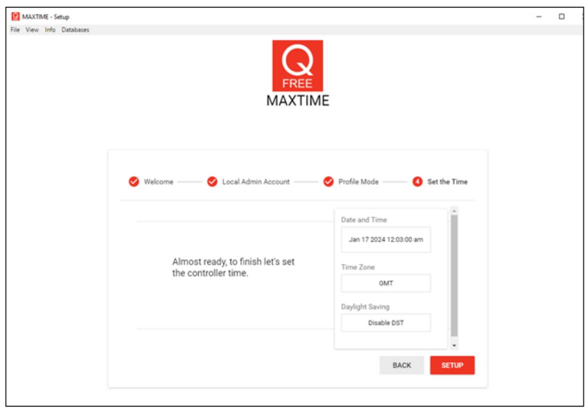


Figure 5-15: MAXTIME Date and Time Setup Display.

E. After configuring the time, the MAXTIME controller is ready to run. The interface of the MAXTIME controller is depicted in Fig. 5-16 below.

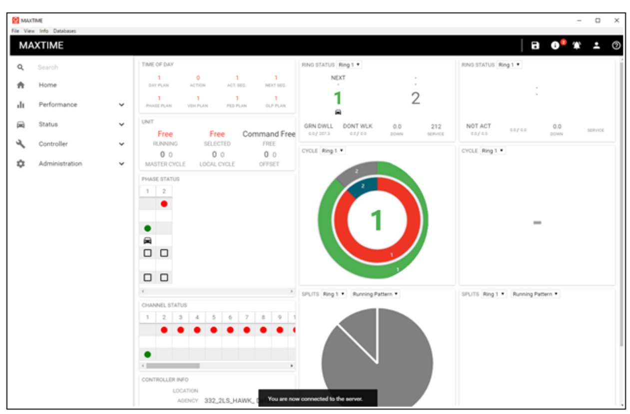


Figure 5-16: Main Display (front panel) of MAXTIME Controller Window.

Step 5: Advanced settings are utilized to couple the MAXTIME controller with PTV VISSIM. To achieve this, the settings must be configured correctly. The details of each setting are provided below:

A. **Web Port:** A web port number is assigned to a specific web service or application on a device. It's crucial to use an unused port number. Users can access the web version of MAXTIME through the active web port. For example, if the IP address is 127.0.0.1 and the web port is set to 1002, users can access it via http://127.0.0.1:1002/maxtime. Each controller must have a unique web port, and it must be unused on the device.
Note: Even after the user closes the MAXTIME Windows, the port number remains activated for some time. It is advisable to allow sufficient time between two consecutive MAXTIME Windows if the controller with the same port number is being used.

B. **Simulation Speed:** Determines the simulation speed, but since VISSIM controls this parameter, changes here won't impact the controller.

C. **Simulation detectors:** Detectors are configured in the simulation software, so it's recommended to leave this unchanged or closed. This feature does not apply to VISSIM.

D. **PTV Vissim IP:** This IP address connects VISSIM with MAXTIME. The default value is 127.0.0.1 and should remain unchanged.

E. **PTV Vissim TCP Port:** The TCP port number must match the port number entered in VISSIM (refer to Step 5 from 0: VISSIM setup, Fig. 5-17). For each signal controller in VISSIM, input a distinct TCP port number in separate MAXTIME windows, adhering to the mandatory guidelines outlined in Section 0.

F. **PTV Vissim Id:** Enter the signal controller number provided in VISSIM. In other words, it's the “NO.” from VISSIM for the specific controller. (See Fig. 5-17)

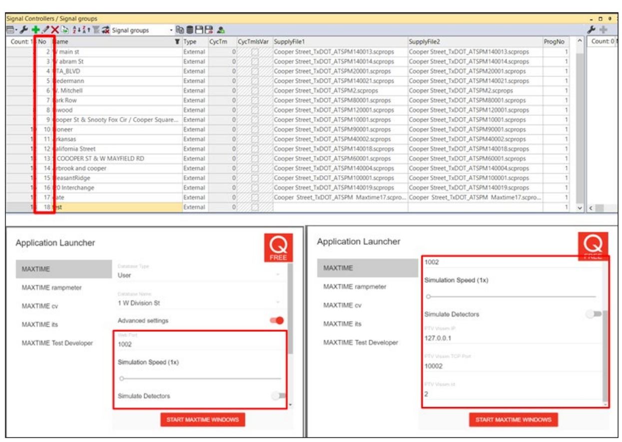


Figure 5-17: Advanced Settings Option in MAXTIME with Multiple Inputs.

Once all inputs are entered, the MAXTIME Window is ready to start. Click on "START MAXTIME WINDOWS" to initiate the controller. If the TCP port number and VISSIM ID do not match based on PTV VISSIM, the controller will load, but the simulation software will not run.

The user interface for MAXTIME is shown in Fig. 5-18. On the left-hand side, the user can find tabs for editing information, designing signal timing, and controllers' information (refer to 1 in Fig. 5-18). Whereas, in the right-top corner, Users can find the buttons that can help them save the file  , configuration detail , active alarm information , user information  , and the help button .

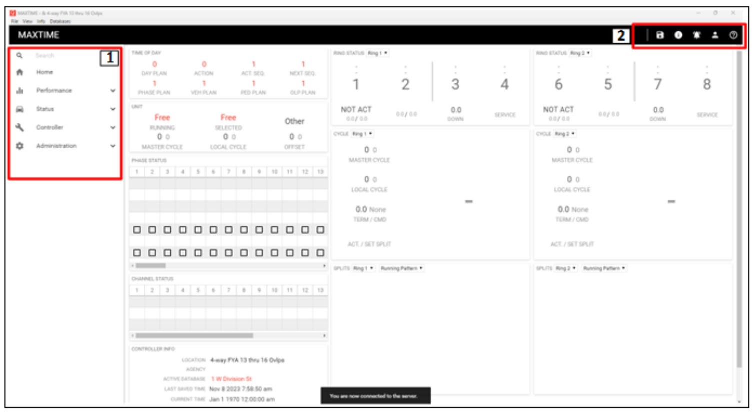


Figure 5-18: MAXTIME Homepage Window Displaying Tabs for Controller, Performance, and Status.

Step 6: To change the cabinet basic information based on intersection details, go to Administration > Unit Information. There, users will have access to modify controller ID, Main Street (Major Street), and Side Street (Minor Street) information.

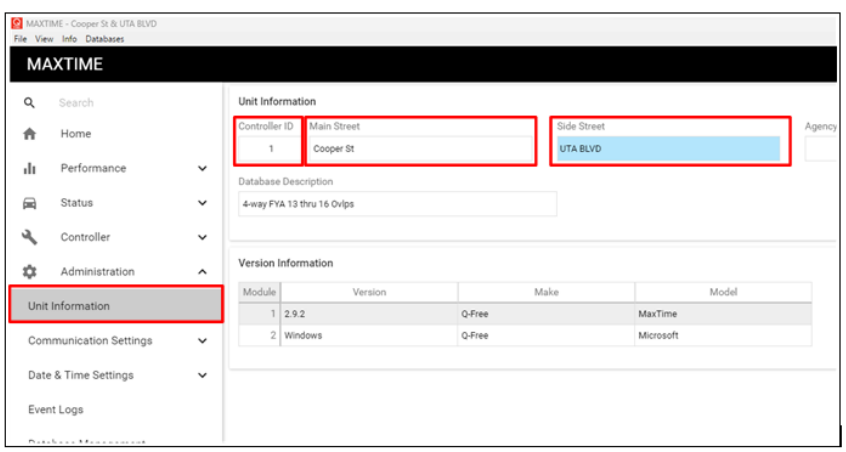


Figure 5-19: Window for Changing Controller Name and Unit Information in MAXTIME.

Step 7: To set up the I/O module type to "Simulation" on MAXTIME, follow these steps: Go to Controller > Advance I/O > Cabinet Configuration > I/O Modules, and change the I/O module type to Simulation. This must be done for coupling MAXTIME with VISSIM. Refer to Fig. 5-20 for visual guidance.

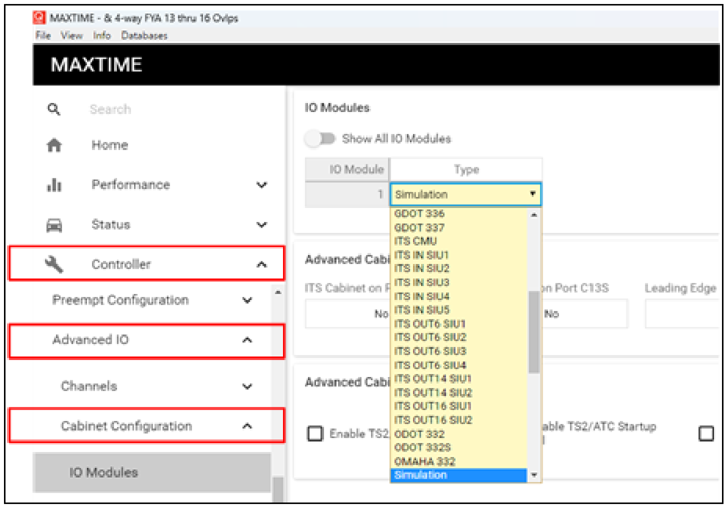


Figure 5-20: Advanced Input-Output (Akhtar & Moridpour) Module Type Configuration in MAXTIME.

Step 8: After this, MAXTIME is ready, and additional signal timing information can be input using the Controller tabs from the user interface. It's important to note that the signal control from Step 6 of VISSIM in 0 (standard signal control parameter) is not directly related to phase numbers in the MAXTIME virtual controller. Instead, it is connected to I/O channels from the I/O modules. For example, based on Fig. 5-21 below, 1 in VISSIM refers to I/O channel 1, correlating to phase vehicle with control source 1 (meaning phase 1). The phase numbers can differ and might not always match. For instance, phase 9 in VISSIM correlates to I/O channel 9, which is a pedestrian phase corresponding to pedestrian phase 2. I/O channels should be modified based on the signal timing plan.

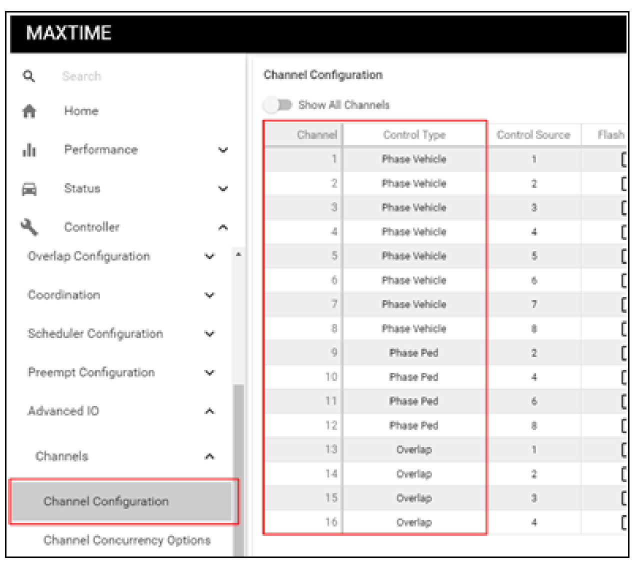


Figure 5-21: Advanced Input-Output (Akhtar & Moridpour) Channel Configuration Page in MAXTIME.

Step 9: The detectors placed in VISSIM are utilized for vehicle calls, and widely used for creating other calls like preemptions and pedestrian calls. To set up detectors correctly, the Input points must be properly placed on MAXTIME; otherwise, calls from the simulation will not be registered on the MAXTIME virtual controller. To achieve this, settings on both MAXTIME and VISSIM need to be adjusted. Firstly, in VISSIM, vehicle detectors should be placed with the correct port number. Please remember that only detectors of the "standard" type are compatible with the required outcome. Choose only the "Standard" type when selecting a VISSIM detector type. Refer to Fig. 5-22 for detector placement. For illustration, Detector 35 is for vehicle detection, and Detector 67 is for pedestrian detection. Port numbers are assigned based on the detector number.

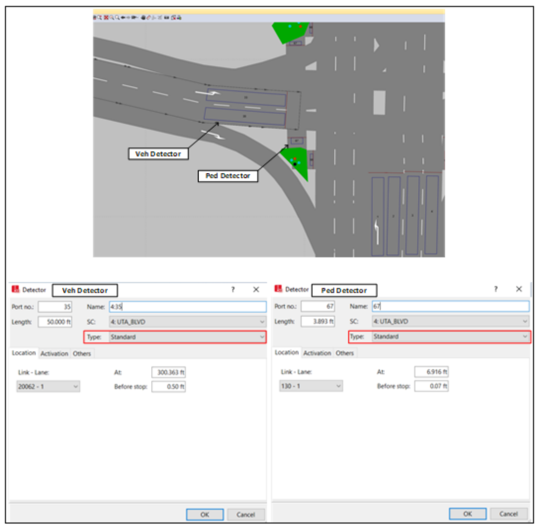


Figure 5-22: Vehicle and Pedestrian Detector Layout.

Step 10: In MAXTIME, ensure that the port numbers are accurately linked to their respective input control types. The index corresponds to the port number, so the input at index 35 should be assigned the "Veh Detector Call" control type. Similarly, the input at index 67 should be designated as "Ped Detector Call." Additionally, various input types, including preemption input, phase hold, and phase omit, can be configured based on the specific output requirements from the detector.

IO Module 1 — Show All Input Points

Input Point	Description	Input Control Type	Index
32	I-32	Veh Detector Call	32
33	I-33	Veh Detector Call	33
34	I-34	Veh Detector Call	34
35	I-35	Veh Detector Call	35
36	I-36	Veh Detector Call	36
37	I-37	Veh Detector Call	37
38	I-38	Veh Detector Call	38
39	I-39	Veh Detector Call	39
40	I-40	Veh Detector Call	40
41	I-41	Veh Detector Call	41
42	I-42	Veh Detector Call	42
43	I-43	Veh Detector Call	43
57	I-57	Veh Detector Call	57
58	I-58	Veh Detector Call	58
59	I-59	Veh Detector Call	59
60	I-60	Veh Detector Call	60
61	I-61	Veh Detector Call	61
62	I-62	Veh Detector Call	62
63	I-63	Veh Detector Call	63
64	I-64	Veh Detector Call	64
65	I-65	Ped Detector Call	65
66	I-66	Ped Detector Call	66
67	I-67	Ped Detector Call	67
68	I-68	Ped Detector Call	68
69	I-69	Veh Detector Call	69

Figure 5-23: Input Points Page for Different Input Control Types in MAXTIME.

Step 11: Configuring overlaps are also different in MAXTIME. In MAXTIME Overlaps are also associated with IO channels. As seen in Fig. 5-24, overlaps 1,2,3,4 is associated with channels 13,14,15,16.

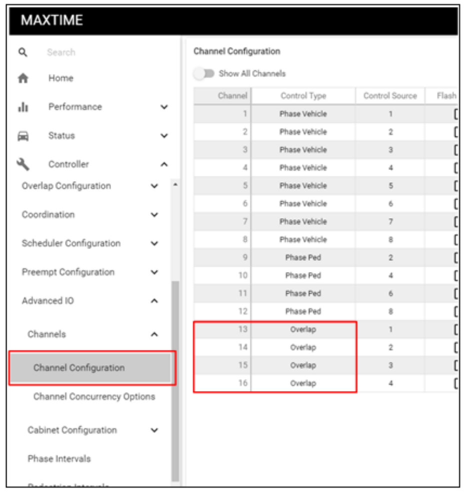


Figure 5-24: IO Channel Configuration for Overlaps in MAXTIME.

Step 12: Create overlaps between two phases from Controller > Overlap Configuration > Overlaps. By default, all inputs are displayed in an "itemized" format. It is advised to switch to the "Table" format for a more user-friendly interface. Toggle between Item and Table formats using the switch tab located at the top right corner.

For instance, in Fig. 5-25, four overlaps are enabled. Included Phases and modifier phases play a crucial role in overlaps. In Overlap 1, phase 2 is the included phase, and phase 1 is the modifier phase. This signifies an overlap between phase 1 (left turn traffic) and phase 2 (opposing through traffic), indicating a flashing yellow or permissive turn. In the IO channel settings, IO channel 13 represents the overlap with control source 1, meaning channel 13 represents Overlap 1. Refer to Fig. 5-25 for details.

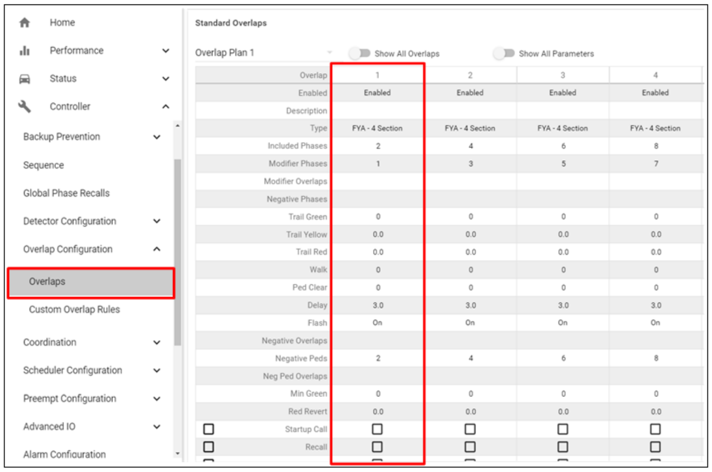


Figure 5-25: Overlaps Configuration in MAXTIME.

Step 13: Once the overlap is properly designed, it must be placed in the VISSIM simulation correctly for it to function as intended. In VISSIM, this can be achieved by locating the IO channel that corresponds to the overlap and placing it in the "Or signal group." Referring to Fig. 5-26 based on the earlier overlap design in the MAXTIME controller (Overlap 1 for flashing yellow involving phases 2 and 1), input the "Or signal group" with Phase number (IO channel) 13 at phase 1 signal head in VISSIM. The modifier phase (phase 1- left-turning phase) is denoted by the red box, and the overlap phase is highlighted in yellow.

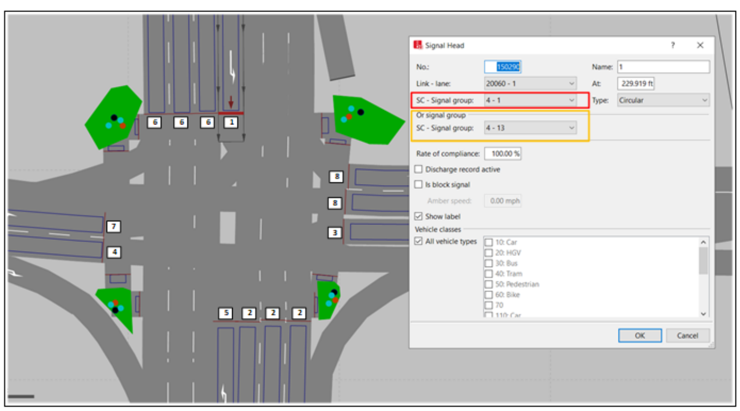


Figure 5-26: Intersection Layout with Different Phases for Overlaps in VISSIM.

Step 14: Lastly, check the status of events such as detector calls, phase status, and overlaps from the main screen > Status. Additionally, the overall controller performance is automatically generated in the form of reports for user access. However, it's important to note that these auto-generated reports have limitations in terms of size and duration.

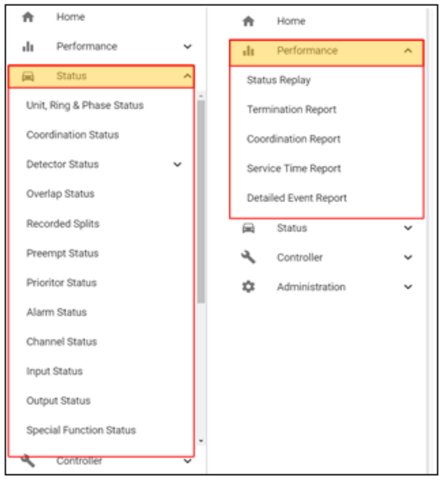


Figure 5-27: Status and Performance Tab Display in MAXTIME.

Step 15: All these previous steps are undertaken to set up a single signal controller at an intersection. If multiple intersections need to be designed based on project requirements, the configuration for each intersection must be done separately. This involves using different signal controller databases and distinct copies of MAXTIME Windows software for each intersection. During configuration, it's crucial to ensure that every intersection has a unique web port number, a unique VISSIM TCP port number, and a corresponding Signal Controller Number from the VISSIM software. Refer to Fig. 5-28, which illustrates two different MAXTIME controllers for two signal controllers in VISSIM.

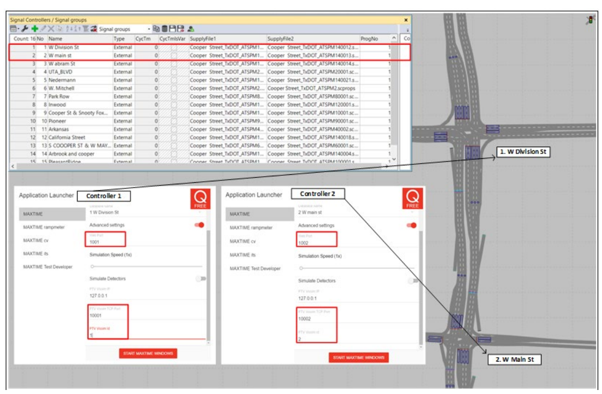


Figure 5-28: Multiple Controllers in MAXTIME Alongside Multiple Intersections in PTV VISSIM.

Note that the highest TCP port is 65,535, and any number higher than that cannot be recognized properly. Similarly, for the VISSIM Signal Controller Number, a value lower than 65,535 is required. Please be advised that this guideline does not provide detailed information on how to enter the signal timing information. The process of entering signal timing information is like entering signal timing in an actual controller of any brand. Users can download and print out the traffic signal timing database of a controller.

5.5. THE SOFTWARE TOOL TO PARSE FROM THE VISSIM OUTPUT DATABASE TO THE ATSPM DATABASE

The above sections focus on preparing the simulation platform to generate the needed data feed for the ATSPM systems. For traffic signal planning or preliminary signal design projects, the default RBC controller in VISSIM is recommended as it is straightforward and commonly used for traffic signal projects. The traffic signal events are derived directly from VISSIM according to the observed signal color transitions without straightforward explanations of what causes the signal transitions. As a result, using the RBC controllers to generate high-resolution traffic signal logs can be quickly configured for multiple intersections but only a small portion of ATSPM MOEs can be generated in the UDOT-ATSPM/ UTAIM because of the limited even types.

In contrast, the more advanced MAXTIME SILS behaves the same as the realistic MAXTIME control software in the field except that the MAXTIME SILS is driven by emulated detector actuations in VISSIM. The high-resolution traffic signal logs in the MAXTIME SILS are generated by the MAXTIME signal emulator while the simulation is running (i.e., generated in a real-time manner), with almost all the defined ATSPM events. As a result, the MAXTIME SILS can provide the data feed for all defined ATSPM MOEs. However, configuring the timing plans in the MAXTIME SILS requires in-depth knowledge of traffic signal control systems and thus the users may take a little longer to configure the MAXTIME signal emulators in VISSIM.

The developed software tool to parse the simulated traffic signal events and connected vehicle trajectories contains several functional modules and they will be explained respectively.

5.5.1. Parsing the RBC signal control events from the VISSIM database to the UDOT-ATSPM Database

Before parsing the RBC-simulated traffic signal events into the UDOT-ATSPM-compliant data, it is assumed that the VISSIM simulation results have been saved into a SQL database and the UDOT-ATSPM has been installed. As an example, shown in Fig. 5-29, the VISSIM output database was set up in parallel with the UDOT-ATSPM's backend database "MOE". Fig. 5-30 demonstrates the data retrieving process if the RBC traffic signal controller is used in VISSIM.

The first step of data parsing will be the locations of retrieving and saving databases, usernames, passwords starting time for simulation, etc. Note that all the relevant tables in the two databases are created by UDOT-ATSPM or VISSIM separately and the database tables' names are static. As such, it is not needed to specify the table names in the parsing software tool.

To avoid tedious user inputs for each new parsing operation, the parsing software allows for loading all the necessary inputs through a few clicks. The parsing inputs are saved in a local text file in JSON format as shown in Fig. 5-31. If a user wishes to customize the parsing inputs, the user can either directly modify the corresponding rows in the JSON file using a text editor or load this JSON file into the graphic user interface first, modify it and then save it. The local JSON file will be updated for the next time of configuration loads (see Fig. 5-32).

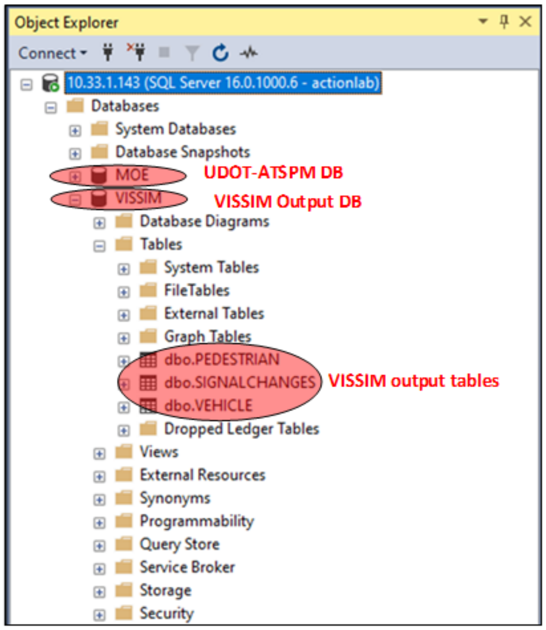


Figure 5-29: UDOT-ATSPM and VISSIM output DBs displayed in SQL Server Management Studio.

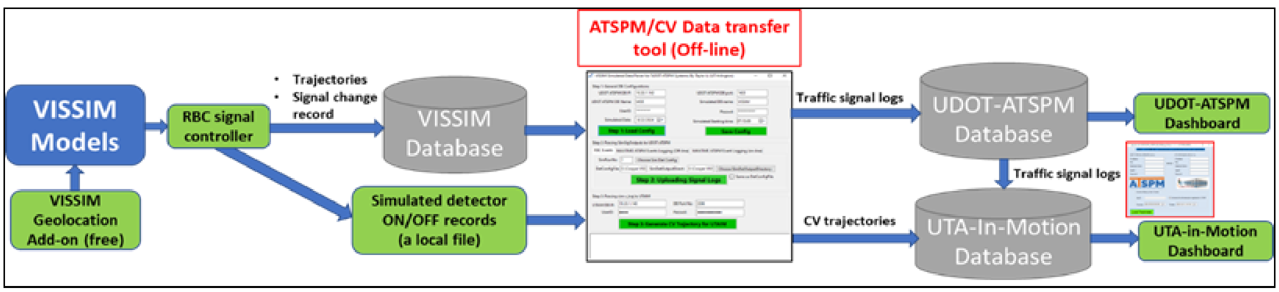
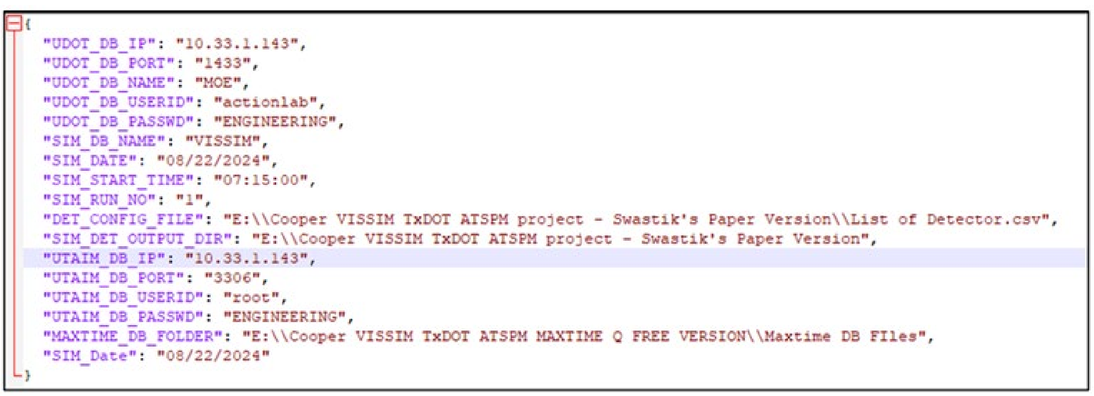


Figure 5-30: Workflow of ATSPM-in-the-loop simulation based on VISSIM+RBC.



```
{
    "UDOT_DB_IP": "10.33.1.143",
    "UDOT_DB_PORT": "1433",
    "UDOT_DB_NAME": "MOE",
    "UDOT_DB_USERID": "actionlab",
    "UDOT_DB_PASSWD": "ENGINEERING",
    "SIM_DB_NAME": "VISSIM",
    "SIM_DATE": "08/22/2024",
    "SIM_START_TIME": "07:15:00",
    "SIM_RUN_NO": "1",
    "DET_CONFIG_FILE": "E:\\Cooper VISSIM TxDOT ATSPM project - Swastik's Paper Version\\List of Detector.csv",
    "SIM_DET_OUTPUT_DIR": "E:\\Cooper VISSIM TxDOT ATSPM project - Swastik's Paper Version",
    "UTAIM_DB_IP": "10.33.1.143",
    "UTAIM_DB_PORT": "3306",
    "UTAIM_DB_USERID": "root",
    "UTAIM_DB_PASSWD": "ENGINEERING",
    "MAXTIME_DB_FOLDER": "E:\\Cooper VISSIM TxDOT ATSPM MAXTIME Q FREE VERSION\\Maxtime DB FIles",
    "SIM_Date": "08/22/2024"
}
```

Figure 5-31: The local JSON file saving the parsing configuration.

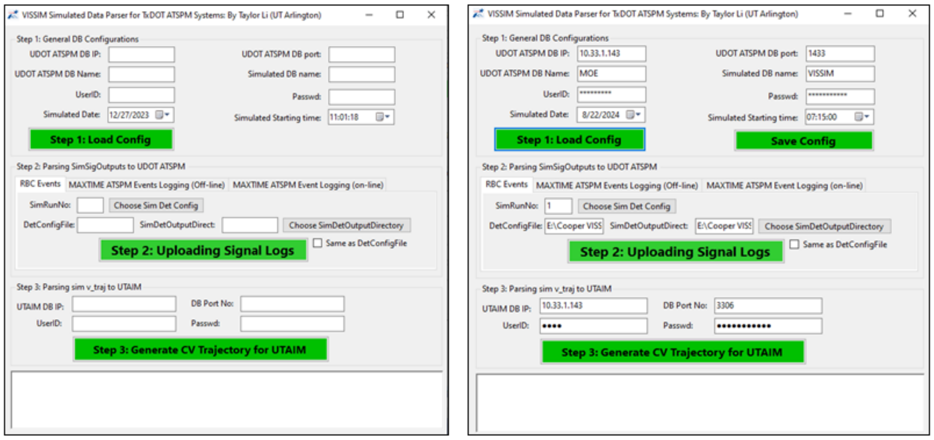


Figure 5-32: Loading the parsing configurations from the local JSON file (Before and After).

Retrieving the RBC-generated traffic signal events and vehicles' trajectories is an offline process, meaning the data retrieving is carried out after a VISSIM simulation run is finished. Simulated traffic signal events in VISSIM include two parts: signal transition records stored in the SQL database and detector actuation records saved as a local text file. The VISSIM output DB has a fixed structure, but each RBC controller's detector output sequence can be customized by users. Therefore, it will be necessary to specify the same detector configuration for the parsing tool to understand the detector actuation records. A CSV file named "List of Detectors.csv".

If the detector output is checked in VISSIM, for each simulation run and each RBC controller/intersection, a text file with the extension name "ldp" will be generated. Each LDP file is automatically named in the form of "xx_a_b.ldp" in which xx is the VISSIM model name, a is an integer representing the intersection ID and b is an integer representing the simulation run number. The parsing tool will first analyze the ldp file's name and identify the simulation run number and intersection ID. Then the parsing software will examine the corresponding row in "List of Detectors.csv" (see Fig. 5-33c) to decode the LDP files. As shown in Fig. 5-33b, each row of the data string represents the instantaneous detector status since the simulation starts. For instance, the first data string representing the selected detectors' status at time=0.1: "+" means occupied, and "." means otherwise. The parsing tool reads each data row and calculates each detector's status at each time step. This decoded information will be parsed into the ATSPM-compliant messages and uploaded into the UDOT-ATSPM database. The user needs to specify the location where the "List of Detectors.csv" is saved and the VISSIM simulation folder where the detector output (ldp) files are generated (in the "Step 2" Section of the graphic user interface).

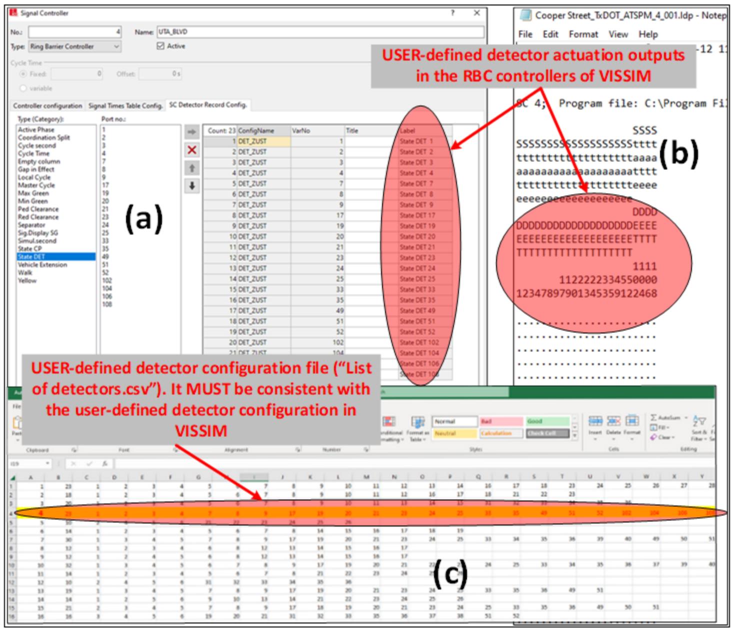


Figure 5-33: RBC detector actuation outputs and decoding.

5.5.2. Parsing the MAXTIME SILS signal control events from the VISSIM database to the UDOT-ATSPM Database

Fig. 5-34 shows the workflow of parsing the simulated traffic signal events into real-world ATSPM systems. Compared to the RBC controller, the traffic signal logs in the MAXTIME SILS are generated by the MAXTIME signal emulators. Each simulated intersection will be coupled with an independent MAXTIME signal emulator. In addition to the settings within the MAXTIME control database, additional steps will be needed for each intersection, including the control database, its web server portal, intersection ID, synchronous TCP socket port, etc.

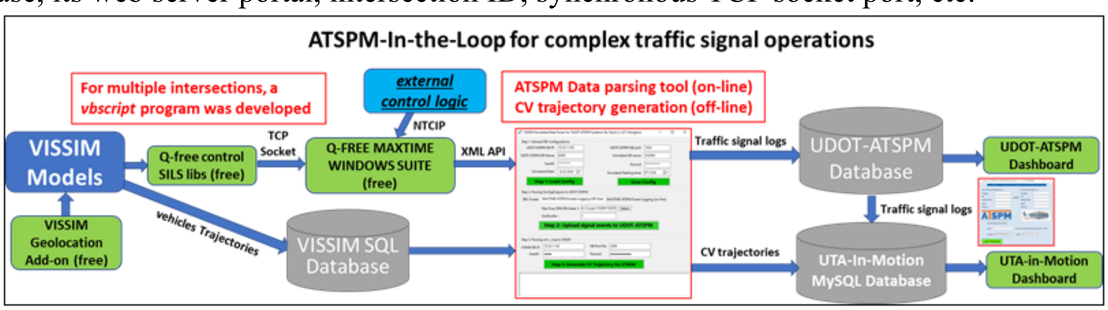


Figure 5-34: Workflow of ATSPM-in-the-loop simulation based on VISSIM+MAXTIME_SILS.

When multiple intersections are coupled with the MAXTIME SILS (e.g., 16 intersections for this project), setting up the MAXTIME SILS manually for each intersection becomes tedious. To address this issue, an automated program written in VBScript was developed to automate the SILS coupling process. The developed program requires regulating the numbering rules for intersection ID, TCP socket ports, and MAXTIME web portal port numbers. It has been described in Section 0

When a MAXTIME signal emulator is fully populated, it will also establish a web server so that a user can access the MAXTIME user interface with a browser to examine and validate the MAXTIME control operations. The web server is also broadcasting the ATSPM-compliant traffic signal events in the Extensible Markup Language or XML form. The developed parsing tool can retrieve these signal events via the XML API and then directly upload them into the UDOT-ATSPM database. (See Fig. 5-35). Note that retrieving traffic signal logs from the MAXTIME SILS must be carried out when the VISSIM simulation is running because of the inherent data archiving limitations in the MAXTIME control software.

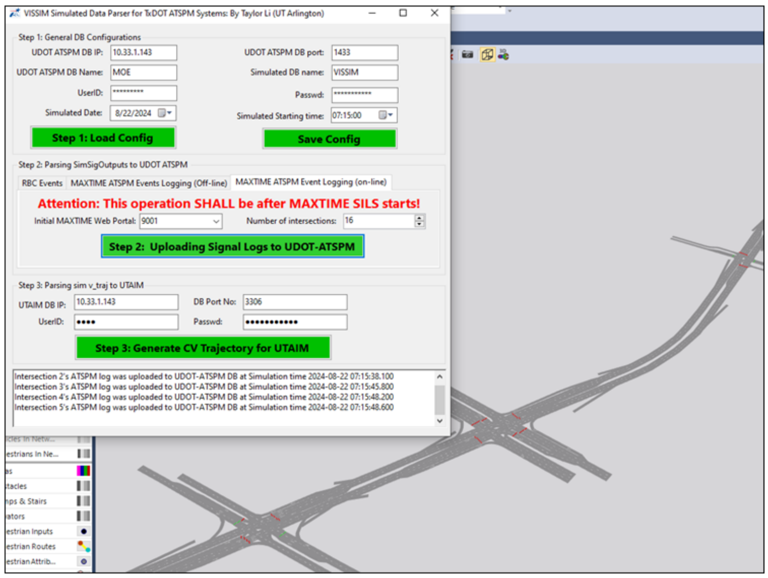


Figure 5-35: Retrieving the ATSPM traffic signal events from the MAXTIME SILS.

5.5.3. Parsing the simulated connected vehicle trajectories and uploading them to the UTA-In-Motion Database

One of the ATSPM systems adopted for this project was developed by the project team at UT Arlington, referred to as UTA-In-Motion. The UTAIM system is an add-on module to the UDOT-ATSPM system, but it can take the emerging traffic big data, such as the commercial connected vehicle or CV data to generate more traffic SPMs across intersections. The CV trajectories are offered in the WGS84 form or (Lat, Lon, t). On top of the transformed WGS84 vehicle trajectories in the VISSIM database, the parsing tool first recalculated each CV waypoint's target timestamp according to its simulated time step (e.g., 0.1, 0.2, 0.3, …) and the specified starting time (e.g., 7:15:00 AM, Aug-22-2024). Furthermore, the parsing tool will downsample the simulated vehicle trajectories' time intervals from 0.1 sec to the realistic 1-3 sec. After these transformations, the simulated CV trajectories will have the same format as the real commercial CV data sets and can be recognized by the UTAIM system. In the parsing tool, this is the last step or Step 3 of data parsing.

5.5.4. Transferring ATSPM data from UDOT-ATSPM to UTA-In-Motion ATSPM

The UTAIM uses a different database engine and design from the UDOT-ATSPM SQL server database. It adopts the free Oracle's MySQL database engine (community version) and a scalable data archiving structure. These configurations ensure a complete separation of the UTAIM system from the UDOT-ATSPM system even if they are hosted on the same computer platform. As a result, when the simulated traffic signal events are uploaded to the UDOT-ATSPM SQL server database, the UTAIM's MySQL database will not be filled with the ATSPM data automatically. A second software tool was developed to transfer the ATSPM data from the UDOT-ATSPM database to the UTAIM MySQL database. Similar to the first parsing tool, users need to load the configuration file to specify the locations of the origin SQL server database and destination databases, usernames, and passwords. Note that the inputs for "From time" and "To time" will ensure that only the ATSPM data timestamped between these two-time points will be transferred. The target simulation time must be within that time range to get the data transferred.

After the above operations, both UDOT-ATSPM and UTAIM will be loaded with the simulated ATSPM data and CV data, and they will be ready to generate and visualize various MOEs.

Figure 5-36: Transferring the ATSPM data from the UDOT-ATSPM database to the UTAIM database.

CHAPTER 6:
A CASE STUDY OF USING ATSPM-IN-THE-LOOP SIMULATION TO OPTIMIZE TRAFFIC SIGNAL TIMINGS

The project team conducted a case study in a controlled scenario using the established VISSIM model and with the help of developed software tools, the two ATSPM systems (UDOT-ATSPM +UTAIM). The study aims to recommend a TM providing instructions on how the ATSPM-In-The-Loop traffic simulation model can give real-time insight into the ongoing traffic signal timing scenario. It examines current traffic conditions using simulation software and compares them with a new model configuration considering increasing traffic volume. The case study identifies opportunities for traffic improvement by comparing travel times, control delays, and queue lengths between the two models. This TM includes a detailed analysis of the findings, accompanied by graphs and metrics, and compares the current and new model configurations. This case study considers the 16 intersections described in the previous chapters.

6.1. EVALUATION OF THE BASELINE TRAFFIC SIGNAL PERFORMANCE USING THE ATSPM-IN-THE-LOOP TRAFFIC SIMULATION MODEL

In the baseline scenario, traffic volume, signal timing, and routing decisions were replicated exactly as they were, without making any adjustments to match the ongoing real-world conditions. To assess how well the traffic signals were performing, four specific intersections - UTA Blvd, Park Row Dr, Pioneer Pkwy, and Interstate 20 were selected from among the 16 intersections along the Cooper Street Corridor. As discussed earlier in TM-4, the Cooper Street Corridor was divided into three zones for analysis. Within each zone, intersections were carefully selected based on various criteria to ensure the model's accuracy. The primary factor guiding the division of intersections into different zones was the vehicular and pedestrian volume.

Intersection 4 (UTA Blvd and South Cooper St) from Zone 1 was opted for due to its proximity to UT Arlington and considerable pedestrian traffic. Two intersections were selected from Zone 2: Intersection 7 (South Cooper St and Park Row Dr) for its high pedestrian volume, and Intersection 10 (South Cooper St and Pioneer Pkwy) due to its significant freight vehicle volume. Lastly, Intersection 16 (South Cooper St and Interstate 20) from Zone 3 was chosen for its distinct characteristics, notably its direct connection to Interstate 20 and unique traffic patterns compared to other intersections. Not all 16 intersections were evaluated due to time constraints and feasibility limitations. Additionally, all these intersections operate on the same common cycle length, with vehicle inputs only occurring at the beginning and end of the Cooper Street Corridor. The interconnected nature of the intersections suggests that evaluating a few intersections in between can validate the accuracy of the results. Hence, these selected intersections were established at various locations as checkpoints for result validation.

6.1.1. Analysis of Performance Metrics from VISSIM

The simulation model ran for two hours, with additional time allocated for model adjustments. Initially, 15 minutes were set aside for adjustment, with an additional 15 minutes provided at the end to ensure that released vehicles could complete their trajectories. This extended the simulation runtime to two and a half hours. Within this controlled condition in the established VISSIM model, various parameters were selected as MOEs. Using node analysis, metrics such as total queue length, average queue length, total vehicle delay, average vehicle delay, and LOS were obtained for each intersection. Table 1 indicates that the average queue length and vehicle delay are significantly higher at the Park Row and Pioneer Pkwy intersection compared to those at UTA Blvd and Interstate 20. This could be attributed to the higher volume of traffic at this intersection. All of these intersections, however, have consistent operating conditions during a simulated peak hour period, as indicated by their LOS ratings of B or C.

Table 6-1: Results obtained from VISSIM.

Intersection	**UTA Blvd**		**Park Row Dr**		**Pioneer Pkwy**		**Interstate 20**	
Movement	NBT	SBT	NBT	SBT	NBT	SBT	NBT	SBT
Queue length	13.94	12.60	82.71	64.86	97.32	31.79	8.71	53.26
Average Queue Length	4.93	9.51	61.14	31.04	81.11	36.80	8.71	26.63
Vehicle Delay	10.38	9.99	26.62	40.65	27.76	22.04	6.10	13.23
Average Vehicle Delay	8.76	14.68	34.83	35.28	37.79	32.08	8.03	12.38
LOS	**B**		**C**		**C**		C	

Note: *The Average queue length, average vehicle delay, and LOS are all calculated by considering all the approaches of the intersection*

6.1.2. Comparison of Measure of Effectiveness (MOEs) from VISSIM and UDOT-ATSPM

From the two ATSPM systems (UDOT-ATSPM + UTAIM), various graphs were generated. The UDOT-ATSPM system comprises different performance metric systems, which are discussed in Table 6-2. These performance metrics are evaluated based on the criteria including time (hours: minutes of the day) as the x-axis and volume, gaps, cycle time, delay, phase duration, and vehicle signal display as the y-axis. The output displays the performance metrics for each phase and all 16 intersections. A brief explanation of the performance metrics based on the parameters considered for the x and y axis is as follows:

Table 6-2: Performance Metrics obtained from UDOT-ATSPM.

SN	Performance Metrics	X-axis	Y-axis	Output	Status
1	Approach Delay	Time (Hours/minutes)	Delay per Vehicle (second)	Approach delay per vehicle, Total delay per vehicle	Used
2	Approach Volume	Time (Hours/minutes)	Volume (Vehicles per hour)	Northbound and Southbound approaches, D-factor	Used

3	Arrivals on red	Time (Hours/minutes)	Volume (Vehicles per hour)	AoR, Percent arrivals on Red, Total vehicles	Used
4	Left turn gap analysis	Time (Hours/minutes)	Gaps	Percent of green time where gap >= 7.4 sec	Not Used
5	PCD	Time (Hours/minutes)	Cycle time (seconds)	AoG, GT, and PR	Used
6	Purdue Phase Termination	Time (Hours/minutes)	Phase number	Force offs, max outs & gap out, split fail, total split failure, split, skips	Not Used
7	Split monitor	Time (Hours/minutes)	Phase duration (seconds)	Percentile split, Avg split, Force offs, gap outs, Skips	Used
8	Timing and actuation	Time (Hours/minutes)	Vehicle signal display	Patterns of signal phases	Not Used

Following the completion of the simulation run, the detector events, signal timing events, and trajectory data were parsed into the UDOT-ATSPM system. This system generated various graphs illustrating traffic patterns. To ensure the accuracy of these graphs and the simulation output, cross-validation was essential. In this process, PTV VISSIM node analysis data served as a ground truth for evaluating the graphs produced by the ATSPM systems. Before delving into a detailed analysis of the outcomes from the two ATSPM systems (UDOT-ATSPM + UTAIM), a comparison was made between the results obtained from VISSIM and UDOT-ATSPM. Graphs depicting approach volume in the northbound and southbound directions at the W Park Row intersection, as well as approach delay, were plotted using node analysis data from VISSIM. Similarly, UDOT-ATSPM was utilized to generate similar graphs illustrating approach volume and approach delay. From Fig. 6-1, both the northbound and southbound approach volumes exhibit similar patterns and values. Fig. 6-1A is drawn on the same scale, ensuring that the patterns and values align precisely, as observed in the figure. This consistency confirms the accuracy of the parsed data from the UDOT-ATSPM system in terms of vehicle-per-hour volume.

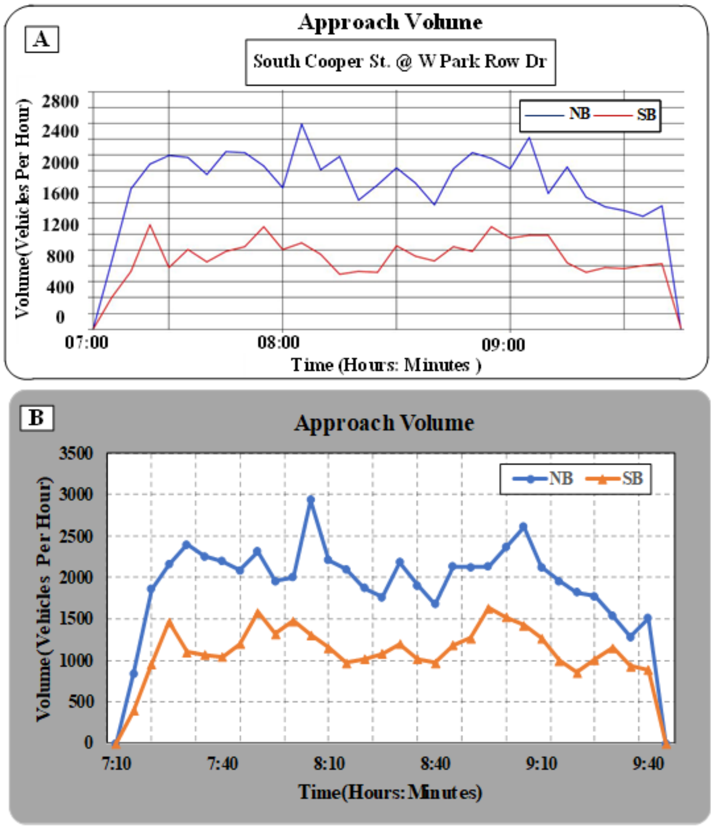


Figure 6-1: Approach Volume for W Park Row Dr obtained from UDOT-ATSPM (A) and VISSIM (B).

However, when it comes to approach delay, some variations are noticeable. This is because UDOT-ATSPM does not factor in startup delay, deceleration, and queue length exceeding the detection zone, unlike VISSIM (UDOT 2022). Consequently, a similar trend in delay with lower Y-axis (approach delay) values would be expected. Fig. 6-2 depicts how these graphs exhibit similar patterns but with different approach delay values, supporting the previous explanation. Furthermore, it's important to note that UDOT-ATSPM generates additional graphs that cannot be replicated using the VISSIM dataset. This limitation restricts our validation process to focusing solely on approach volume and delay when comparing the outputs of the two systems.

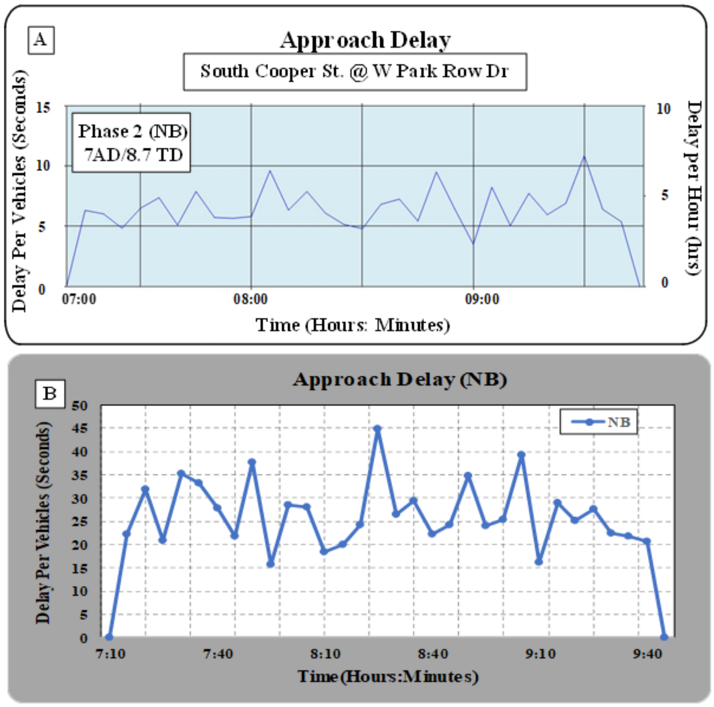


Figure 6-2: Approach Delay for W Park Row Dr obtained from UDOT-ATSPM (A) and VISSIM (B).

6.1.3. Analysis of performance metrics from UDOT-ATSPM

As outlined in Table 6-3 only a few MOEs from the UDOT-ATSPM system were utilized. This decision was made because certain graphs or MOEs were either not applicable to our analysis or the data were not collected in a format that allowed for the creation of those particular MOEs. Therefore, more emphasis is given to the four-performance metrics, approach delay, approach volume, AoR, and PCD. Table 6-3 reveals that Park Row Dr experiences the highest approach delay among all intersections, resulting in longer queue lengths, congestion, and delays totaling 10 hours. Factors contributing to this delay include insufficient green time, heavy traffic volume, coordination issues, intersection geometry constraints, driver behavior, signalized pedestrian crossings, and long queues. Conversely, UTA Blvd benefits from an average of above 90% green time (GT), indicating effective intersection coordination. Despite Park Row Dr's high approach delay, its platoon ratio (PR) of 1.69 suggests relatively efficient traffic progression. This indicates that although vehicles encounter delays approaching the intersection, they tend to travel in groups with minimal interruptions, potentially facilitating smoother traffic flow along the corridor. Overall, it's apparent that traffic volume tends to be higher in the northbound direction compared to the southbound direction. This observation aligns with the fact that the Arlington Central Business District (CBD) falls within this region, attracting a greater number of trips from the northbound direction. Furthermore, it's important to note that in reality, the Interstate 20 intersection functions as a diamond interchange, likely operating with two intersections functioning under different overlap phases. However, it's worth mentioning that this aspect was not considered in our simulation due to its scope exceeding the project's region of interest. As a result, higher volume is observed with a lower PR, as the incoming traffic isn't interrupted or controlled elsewhere.

Table 6-3: Performance Metrics for the Chosen Intersection.

Intersection	UTA Blvd		Park Row		Pioneer Pkwy		Interstate 20	
Performance Metrics	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)
Approach Delay (sec)	2	4	7	15	8	10	1	2
Total Delay (hr.)	2	2.70	8.70	10	7	7.60	2.20	3.20
Approach Volume	3413	2317	4787	2339	3175	2753	7071	5736
Arrivals on Red (AoR-%)	10	19	19	42	23	35	14	25
Red Time (RT-%)	38	36	52	57	45	45	18	18
Arrivals on Green (AoG-%)	90	81	81	58	77	65	86	75
Green Time (GT- %)	62	64	48	43	55	55	82	82
Platoon Ratio (PR)	1.45	1.27	1.69	1.35	1.40	1.18	1.05	0.91
Average Split	94.30	97.07	75.60	68.30	86.50	86.10	125.20	125.20

Using data from the UDOT-ATSPM system, approach volume graphs for Pioneer Pkwy were generated. As shown in Fig. 6-3, the approach volume for both directions can be observed. However, it's important to note that the volume on the Y-axis represents hourly traffic volume, which may be misleading at first glance. Further calculations may be necessary to obtain the actual volume, depending on the selected time interval. In addition to volume, the graph provides other important information such as the peak hour factor (PHF), K-Factor, and the peak hour itself. For Pioneer Pkwy, it was found that the peak hours differed for different approaches. The peak hour for northbound traffic was from 7:30 am to 8:30 am, with a total volume of 3175 vehicles, while for southbound traffic, the peak hour was from 8:30 am to 9:30 am, with a total volume of 2753 vehicles. The PHF for this intersection was 0.964. The directional factors for northbound and southbound traffic were 0.565 and 0.473, respectively, indicating that the northbound approach is dominant and has higher volumes. This information is essential for signal timing and design, serving as the foundation for making informed decisions. Therefore, this graph is a valuable tool for understanding the basic nature of traffic at any specified intersection.

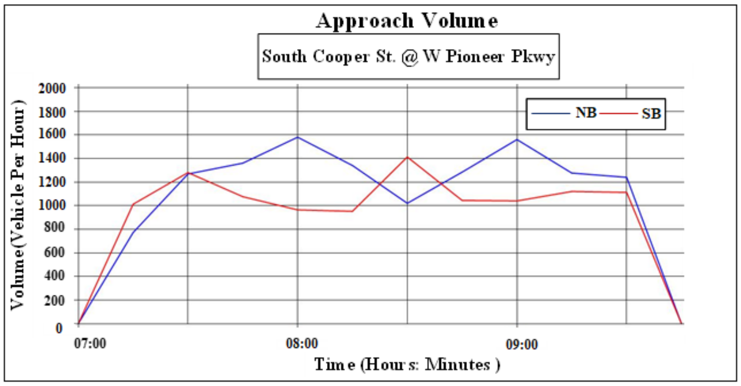


Figure 6-3: Approach Volume at W Pioneer Pkwy.

Similarly, from the UDOT-ATSPM system, approach delay graphs for UTA Blvd were generated. From Fig. 6-4, it's evident that southbound traffic experiences greater delays compared to northbound traffic. The average delay per vehicle is approximately 2 seconds for Phase 2 (NB) and 4 seconds for Phase 6 (SB), with total delays from 7:15 am to 9:45 am reaching 2 and 2.7 hours, respectively. A distinct trend is noticeable: delays rise during the initial hour, decline between 8:00 am to 9:00 am, and then increase simultaneously in both approaches. This pattern is attributed to volume calibration, where the volume from the first hour is duplicated in the second hour of simulation, leading to a decrease in volume after the initial hour. This decrease is reflected in the delay pattern. This highlights the dependence of delay on volume when signal timing is accurate. Without a correct correlation between signal timing and volume, such a pattern would not emerge. The highest average delay per vehicle occurs at 9:15 am southbound. This observation confirms that northbound traffic is favored over southbound traffic in reality.

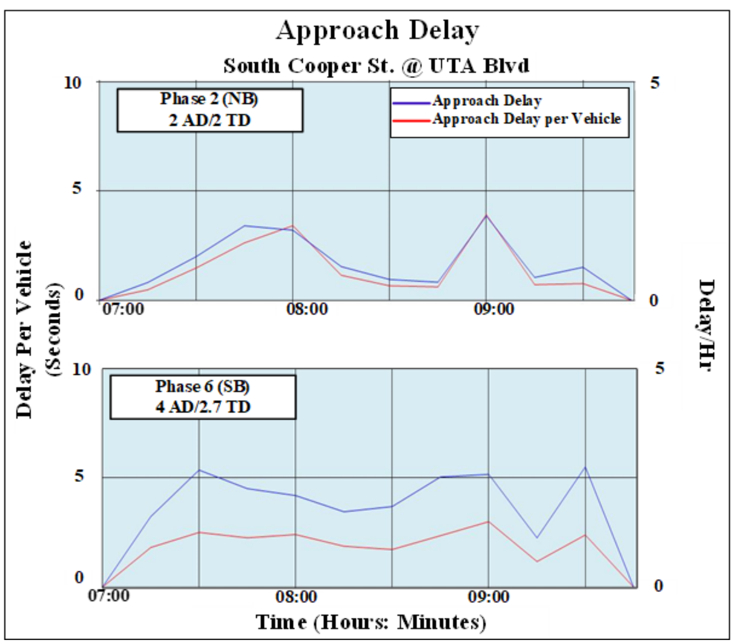


Figure 6-4: Approach Delay at UTA Blvd Phase 2 (NB) and Phase 6 (SB).

The AoR metric visualizes the number of vehicles arriving at an intersection during the red phase of the traffic signal. This metric is valuable for agencies as it enables them to conduct before and after comparison studies, ensures adherence to accepted AoR standards, and identifies potential bottlenecks by comparing arrivals at different coordinated intersections along corridors. In Fig. 6-5, it's observed that Phase 6 (SB) has an AoR percentage of 25%, while Phase 2 (NB) has 14% at Interstate 20. This discrepancy suggests a possible imbalance in traffic signal timing or synchronization, indicating the need for adjustments to improve phase timings, particularly for southbound traffic. The impact of the arrivals on an AoR percentage becomes more apparent through further before-and-after analysis.

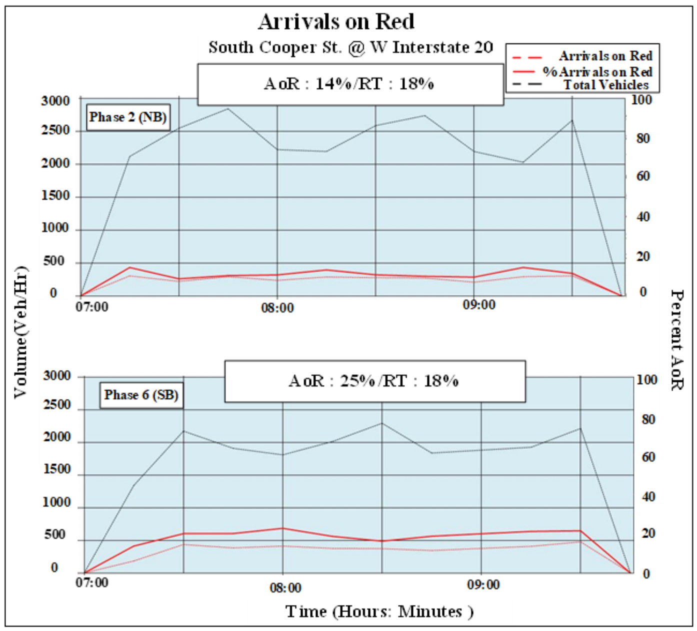


Figure 6-5: Arrivals on Red on W Interstate 20.

The PCD stands out as one of the most unique and crucial graphs, aiding users or agencies in comprehending the signal timing plan. This diagram offers insights into how signal timing is influenced by vehicle arrival, displaying vehicle arrivals alongside the cycle length. These graphical tools, exemplified by Fig. 6-6, illustrate how traffic performance at an intersection can be assessed. In Fig. 6-6, the PCD of Park Row Dr is shown, providing essential parameter values such as AoG, GT, and PR. These parameters enable users to gauge the efficiency of the intersection and serve as tools for fine-tuning traffic performance to alleviate delay, congestion, or any potential bottlenecks.

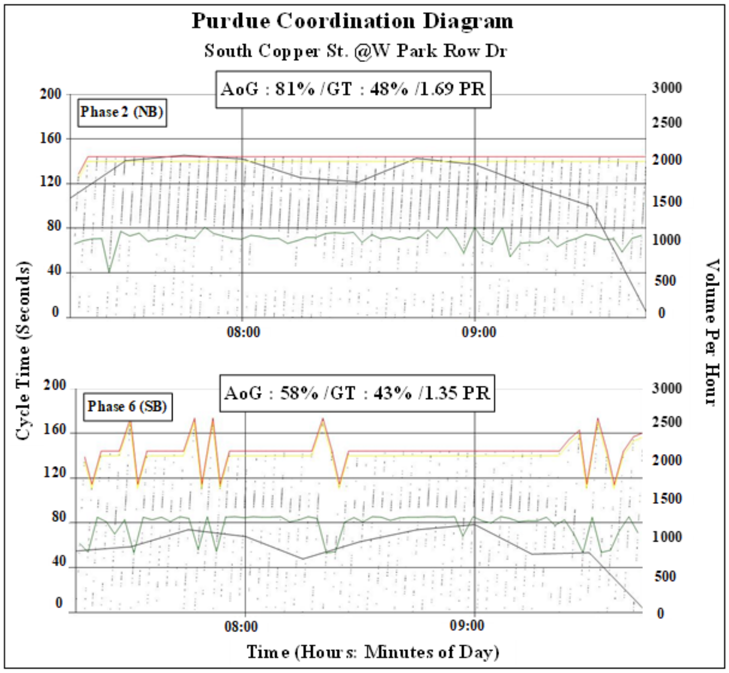


Figure 6-6: Purdue Coordination Diagram for W Park Row Dr.

The PCD of Park Row Dr reveals that the AoG for Phase 2 (NB) is 81%, while for Phase 6 (SB), it's 58%. A higher AoG percentage for the northbound phase suggests that more time is allocated to accommodate traffic flow in that direction compared to the southbound phase. However, relying solely on this value may not provide a comprehensive understanding of the intersection's performance. For example, in scenarios where the queue length is long, the detector remains occupied, leading to higher AoG values. This situation may not accurately depict the actual traffic conditions and cannot be interpreted in isolation using these figures. Therefore, in this case study, a PCD has been integrated along with a trajectory-based TSD from UTAIM to evaluate if such effects are present. Combining PCD and TSD data from both UDOT-ATSPM and UTAIM enables us to delve deeper into assessing whether these locations demonstrate elevated AoG values despite exhibiting poor performance.

6.1.4. Analysis of performance metrics from UTAIM

In line with UDOT-ATSPM, the UTAIM system also requires data parsing. Specific data parsing software is utilized to transfer data from UDOT-ATSPM to UTAIM. After validating the data from the UDOT-ATSPM system and parsing the same data to UTAIM instead of the PTV VISSIM database, there is no need to re-validate the UTAIM system against the ground truth. However, it is important to assess whether the graphs and their outputs are comparable, as UDOT-ATSPM and UTAIM systems may employ different approaches for calculating various parameters. This comparison will provide insights into any differences or similarities between the two systems' outputs, enabling a better understanding of their performance and reliability. Like UDOT-ATSPM, Comprehensive PCDs are provided by the UTAIM system, with volume inputs assigned from northbound to southbound, at each phase of the intersection. To enable comparative analysis, the same intersection was also examined using the UDOT-ATSPM system, which produced PCDs that summarized AoG and the percentage of GT, acting as performance indicators for traffic signals. The AoG and GT values produced by both the UDOT-ATSPM and UTAIM systems for the simulated time were comparable for all intersections. Fig. 6-7 shows the PCDs produced by the two ATSPM systems that exhibit this closeness in Phase 2 (NB) at UTA Blvd and South Cooper St. A similar observation is discussed in Fig. 6-7. further illustrates this discovery for Phase 6 (SB) at W Park Row Dr and South Cooper St.

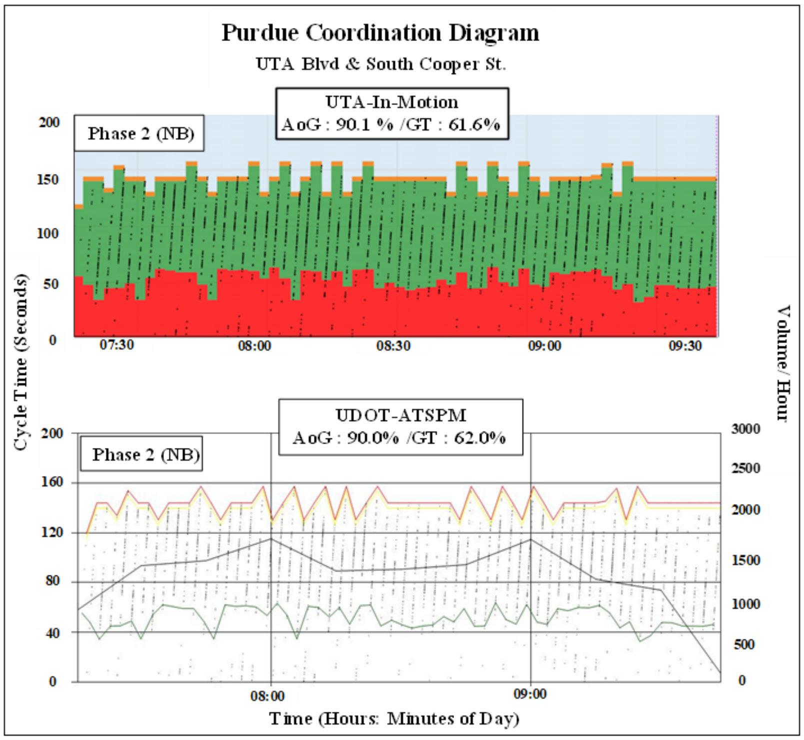


Figure 6-7: Purdue Coordination Diagram for Phase 2 (NB) UTA Blvd & South Cooper St.

Further looking at Fig. 6-8, from almost 8:30 to 9:30 am, the cycle length remains constant with an average length of 144 seconds. This cycle length suggests stable traffic conditions and consistent demand patterns at the intersection at that period. Overall, this showcases efficient signal phasing and allocation of green time, contributing to smoother traffic flow and reduced delays for current traffic volume.

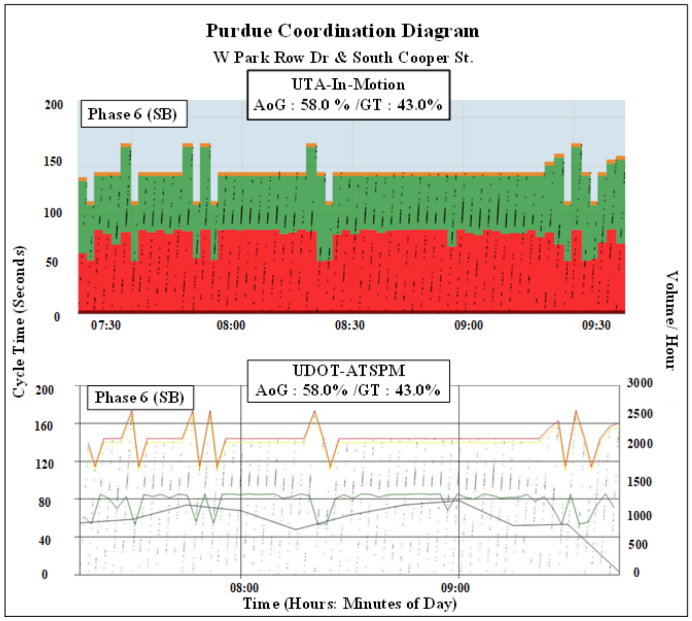


Figure 6-8: Purdue Coordination Diagram for Phase 6 (SB) W Park Row Dr & South Cooper St.

The UTAIM system can generate TSD based on vehicle trajectory. To replicate the output scenario of connected vehicle data, only 10% of the vehicles' trajectories are displayed in the TSD. Fig. 6-9 depicts the coordinated TSD for intersections ranging from 16 to intersection 11. Upon closer examination of the northbound approach at Intersection 11(Arkansas Ln) which is (Region A) in Fig. 6-9, some vehicles are moving slower, resulting in delays and queues. Although the queue is not extremely pronounced or evident in the diagram, it's clear that certain vehicles are stopping. Similarly, Intersection 13 (Mayfield Rd) also experiences higher queues (Region B). Around 8:00 am, at Intersection 13 (Mayfield Rd), many vehicles experienced multiple stops in queues before crossing the intersection. This indicates a split failure, where vehicles are unable to cross in the next cycle and must wait through multiple cycles to pass the intersection. Given the correlation between volume and signal timing, notable congestion isn't observed, as the entire corridor is in coordination. However, it's expected to see more queues in scenarios where vehicle demand increases without changes to the signal timing plan.

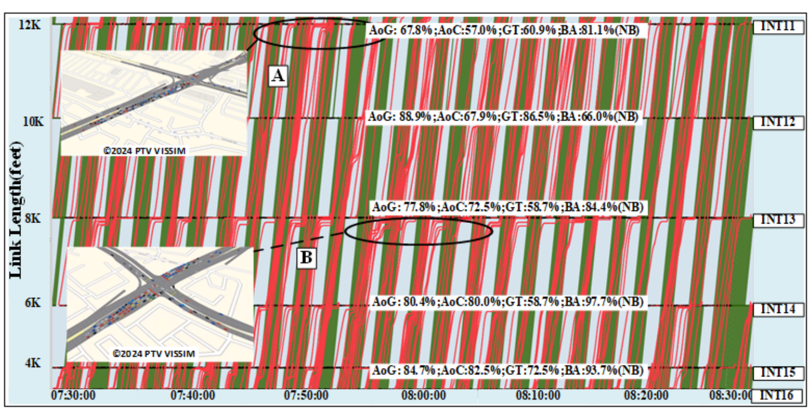


Figure 6-9: Time Space Diagram for Northbound Cooper Street.

6.2. ANALYZING TRAFFIC SIGNAL PERFORMANCE WITH PROJECTED TRAVEL DEMAND GROWTH USING ATSPM-IN-THE-LOOP SIMULATION

The primary objective of developing this model is to enhance the efficiency of traffic signal operations by optimizing signal timings. The aim is to facilitate a seamless transition for increasing traffic volumes while ensuring the optimal functioning of the traffic signal system. To achieve this goal, the traffic volumes on all approaches were simulated to reflect the projected increase in travel demand over five years. The traffic volumes were adjusted using a growth factor $(1 + 3\%)^5 = 1.15$, which represents the anticipated new travel demand in five years. Furthermore, the performance of the traffic signal system was evaluated using the ATSPM-In-The-Loop traffic simulation model. This evaluation is crucial to assess the impact of the anticipated traffic demand and to determine any necessary adjustments or optimizations required for the traffic signal timings. By conducting this evaluation, traffic engineers and planners can gain valuable insights into the performance of the traffic signal system under anticipated future conditions. This information enables them to make informed decisions and implement appropriate measures to ensure the smooth flow of traffic and efficient signal operations, thereby meeting the demands of the growing traffic volume.

6.2.1. Analysis of performance metrics from VISSIM

Fig. 6-10 below is a snapshot to visualize the change in traffic volume at Intersection Park Row Dr for the same period. The "After" image has a comparatively greater number of vehicles northbound and southbound as shown below. This surge in traffic suggests a significant shift in the flow of vehicles through this intersection after the increase in volume. The existing traffic signal timing might not be able to handle this demand, so further need of study can be seen to optimize traffic signals for smooth traffic flow.

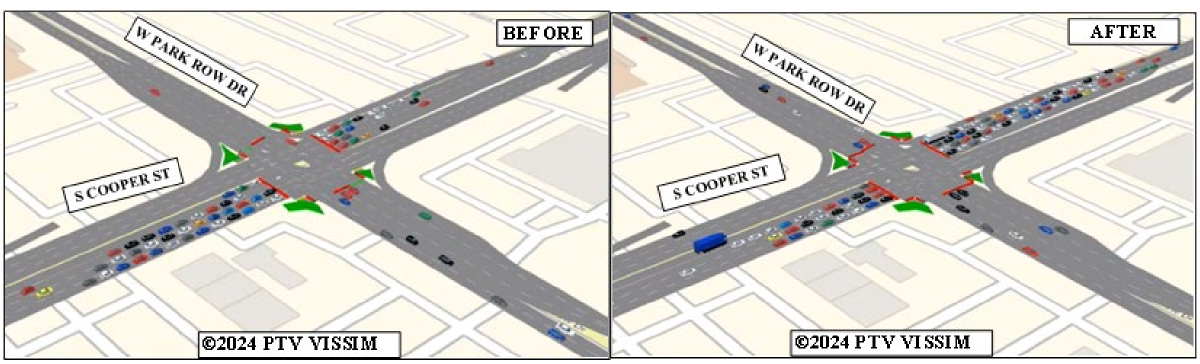


Figure 6-10: Change in Volume at W Park Row Dr.

From Table 6-4, the increase in the traffic volume has significantly affected each intersection. The average queue length and the vehicle delay have been seen growing in almost all the routes of the intersection and the LOS in the Intersections UTA Blvd and Pioneer Pkwy have declined from B to C and C to D respectively. Below is the before and after evaluation for each intersection for the average queue length and average vehicle delay. The percentages denote the percentage change of each parameter for the new volume to the original volume.

Table 6-4: Results obtained from VISSIM.

Intersection	UTA Blvd		Park Row		Pioneer Pkwy		Interstate 20	
Movement	NBT	SBT	NBT	SBT	NBT	SBT	NBT	SBT
Queue Length	14.77 **(6%)**	15.72 **(25%)**	134.77 **(63%)**	62.09 **(-4%)**	175.76 **(81%)**	42.83 **(35%)**	12.43 **(43%)**	90.20 **(69%)**
Average Queue length	5.78 **(17%)**	11.74 **(23%)**	96.83 **(58%)**	30.16 **(-3%)**	130.39 **(61%)**	45.09 **(23%)**	12.43 **(43%)**	45.10 **(69%)**
Vehicle Delay	11.27 **(9%)**	10.56 **(6%)**	32.96 **(24%)**	33.65 **(-17%)**	39.07 **(41%)**	25.41 **(15%)**	8.36 **(37%)**	15.04 **(14%)**
Average Vehicle Delay	10.76 **(23%)**	17.03 **(16%)**	38.63 **(11%)**	31.73 **(-10%)**	52.22 **(38%)**	33.44 **(4%)**	10.48 **(31%)**	14.47 **(17%)**
LOS	C		C		D		C	

Note*: The percent change in the above performance metrics after an increase in volume was calculated as below:*

$$Percentage\ Change = \left(\frac{New\ Value - Orginal\ Value}{Orginal\ Value}\right) * \ 100\%$$

Furthermore, a graph depicting the average travel time of vehicles was plotted before and after increasing the volume based on the VISSIM results. In Fig. 6-11, the average travel time has experienced a significant increase. Particularly notable is the sharp rise in average travel time after 8:00 am, where the travel time from one end of the corridor to the other end saw a drastic increase of more than 60%. This pronounced increase indicates that despite the Cooper Street Corridor being in a coordinated phase, the increase in vehicle volume is not consistent with the previous signal timing. Consequently, this discrepancy leads to vehicle delays, congestion, and longer travel times. Although average travel time serves as a fundamental indicator of deteriorating traffic performance compared to previous scenarios, it also serves as an initial step in assessing whether traffic conditions have worsened. Hence, a detailed node analysis was conducted based on the VISSIM database to further confirm the deterioration of traffic conditions. The findings of this analysis are outlined in the subsequent section.

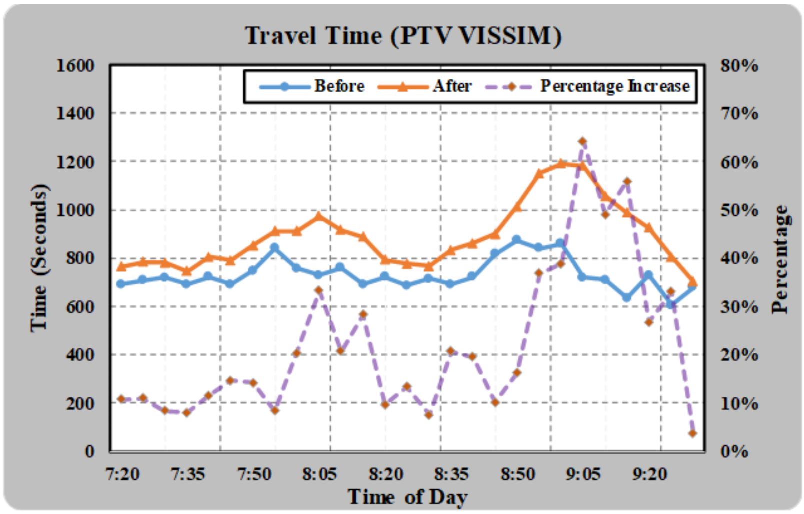


Figure 6-11: Comparison of the Travel Time with Increased Volume.

Using the node analysis data provided by the VISSIM simulation, vehicle average delay and average queue lengths were computed. A comparative study of vehicle delay at the intersection was conducted to assess the impact of increased volume. This study revealed that as the volume of traffic increased, vehicle delays escalated across almost every approach of the intersection. Fig. 6-12A illustrates the average delay for different approaches, showcasing significant disparities within the intersection. Pioneer Pkwy, being a major-to-major intersection, experienced a notable surge in delays, surpassing a 35% increase. Further, the queue length has increased by more than 20% in every direction, as shown in Fig. 6-12B. Northbound Pioneer Pkwy and southbound Interstate 20 have significant increases in queue length. One contributing factor to this phenomenon is the higher traffic volume, resulting in increased interactions between drivers and subsequent congestion. This congestion causes vehicles to wait longer at signalized intersections. Additionally, the heightened demand volume may necessitate adjustments such as increased cycle lengths, or the current signal timing may not adequately accommodate the increased volume, leading to longer queues and delays. This emphasizes how increased traffic volume directly impacts average vehicle delay, average queue length, and overall traffic flow at the intersection. Thus, it is evident that augmenting the volume has indeed impacted the Cooper Street Corridor.

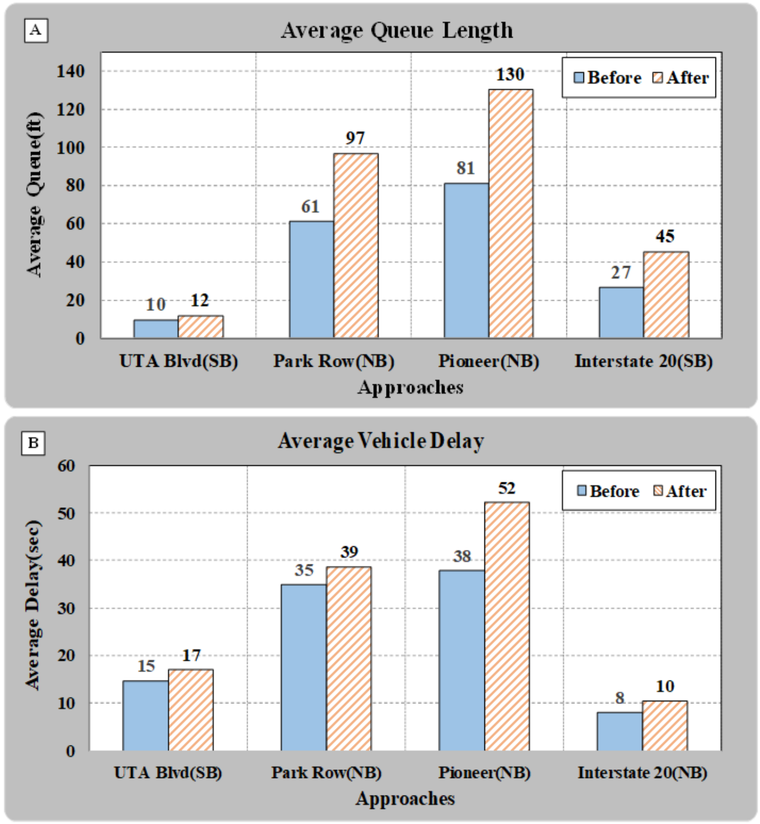


Figure 6-12: Comparative study of Average Vehicle Delay and Queue Length.

6.2.2. Analysis of performance metrics from UDOT-ATSPM

Similarly, to the previous section, all comparable graphs were generated to analyze the performance of traffic signals. In the scenario of increased volume, it was observed that approach delays and total delay, as well as AoR, increased, while AoG and GT decreased. For instance, in the case of the UTA Blvd northbound phase, AoR surged by 20%. This indicates that more vehicles are arriving during the red signal phase, leading to longer queues. Likewise, for Pioneer Pkwy, AoR increased by 4%, while AoG decreased by 4%. This suggests a shift in the traffic pattern, with more vehicles encountering red signals and fewer benefiting from green signals. Additionally, this also suggests a disruption in coordination, where the new traffic arrival pattern may not be optimal for the existing coordinated cycle length.

Table 6-5: Performance Metrics obtained from UDOT-ATSPM.

Intersection	**UTA BLVD**		**Park Row Dr**		**Pioneer Pkwy**		**Interstate 20**	
Performance Metrics	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)
Approach Delay (sec)	3 **(50%)**	4 **(0%)**	7 **(0%)**	14 **(-7%)**	10 **(25%)**	11 **(10%)**	1 **(0%)**	2 **(0%)**
Total Delay (hr.)	3.2 **(60%)**	3.3 **(22%)**	10.9 **(25%)**	10.8 **(8%)**	9.7 **(39%)**	9.5 **(25%)**	2.6 **(18%)**	3.4 **(6%)**
Approach Volume	3900 **(14%)**	2709 **(17%)**	5511 **(15%)**	2738 **(17%)**	3636 **(15%)**	3170 **(15%)**	8115 **(15%)**	6571 **(15%)**

Arrivals on Red (AoR-%)	12 **(20%)**	20 **(5%)**	18 **(-5%)**	38 **(-10%)**	24 **(4%)**	36 **(3%)**	13 **(-7%)**	20 **(-20%)**
Red Time (RT-%)	39 **(3%)**	37 **(3%)**	53 **(2%)**	58 **(2%)**	47 **(4%)**	46 **(2%)**	17 **(-6%)**	17 **(-6%)**
Arrivals on Green (AoG-%)	88 **(-2%)**	80 **(-1%)**	82 **(1%)**	62 **(7%)**	76 **(-1%)**	64 **(-2%)**	87 **(1%)**	80 **(7%)**
Green Time (GT- %)	61 **(-2%)**	63 **(-2%)**	47 **(-2%)**	42 **(-2%)**	53 **(-4%)**	54 **(-2%)**	83 **(1%)**	83 **(1%)**
Platoon Ratio (PR)	1.44 **(-1%)**	1.27 **(0%)**	1.74 **(3%)**	1.48 **(10%)**	1.43 **(2%)**	1.19 **(1%)**	1.05 **(0%)**	0.96 **(5%)**
Average Split	93.3 **(-1%)**	96.6 **(2%)**	74.0 **(-22%)**	66.2 **(-30%)**	83.1 **(-12%)**	84.3 **(-11%)**	133.8 **(42%)**	133.8 **(42%)**

Utilizing updated data from the UDOT-ATSPM system, new approach volume graphs were generated for Pioneer Pkwy. As observed in Fig. 6-13, a slight increase in volume is evident when comparing the before and after scenarios. Also, from Fig. 6-13, it was observed that the peak hour varied for different approaches but remained consistent with the previous baseline scenario. This consistency can be attributed to an evenly increased volume of 15% across all approaches. The peak hour for both northbound and southbound traffic persisted from 7:30 am to 8:30 am and from 8:30 am to 9:30 am, respectively, mirroring the previous scenario. However, the PHF for this intersection shifted to 0.945. Compared to the baseline scenario, the PHF decreased, indicating more variable traffic flow. Furthermore, the directional factors for northbound and southbound traffic were 0.553 and 0.467, respectively. These figures suggest that the northbound approach continues to dominate with higher volumes compared to the southbound direction.

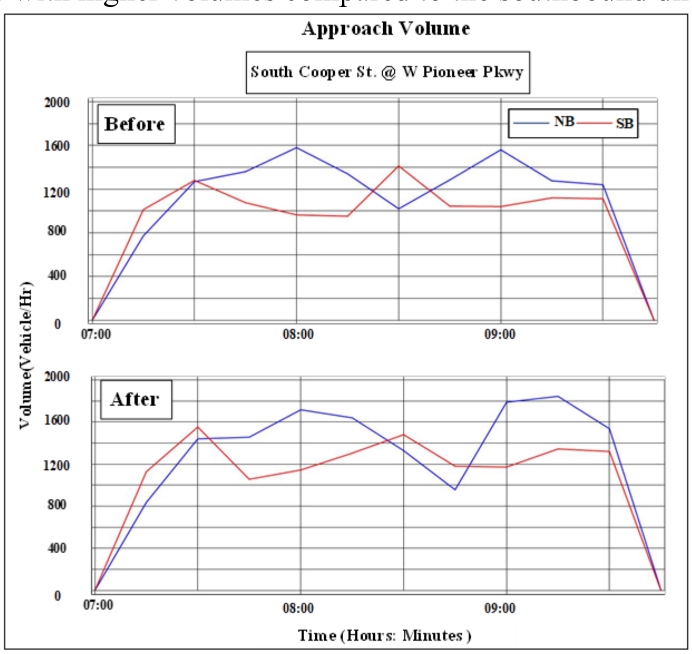


Figure 6-13: Approach Volume at W Pioneer Pkwy.

Likewise, approach delay graphs for UTA Blvd were generated. The comparison between the before and after scenario graphs is presented in Fig. 6-14. It is evident from the figure that southbound traffic experiences greater delays compared to northbound traffic, consistent with the baseline scenario. These additional graphs aid in validating our conclusion regarding the worsening conditions at these intersections with the expected increase in vehicle demand. The average delay per vehicle is approximately 3 seconds for Phase 2 (NB) and has increased by 1 second in the before-after study, as shown in Fig. 6-14. A similar trend observed in the baseline scenario is noticeable here as well.

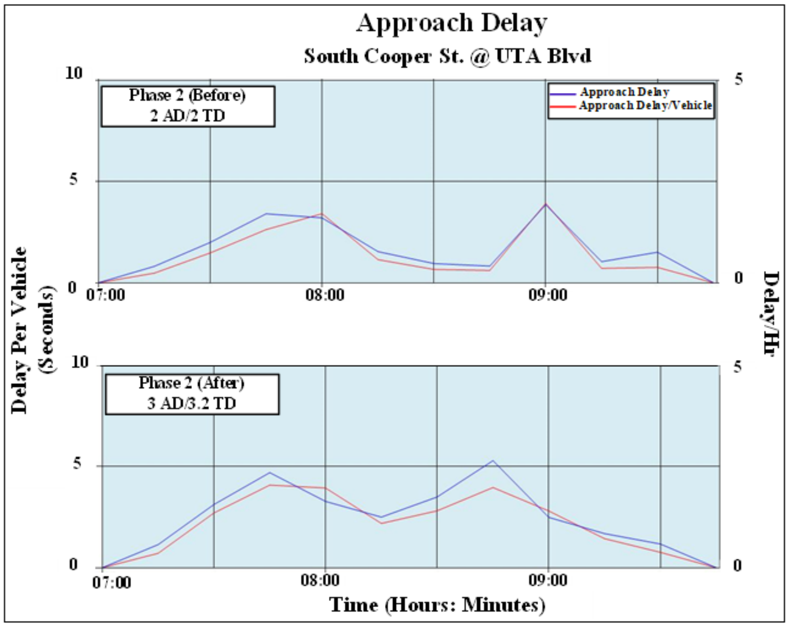


Figure 6-14: Approach Delay at UTA Blvd.

Following the increase in volume, the arrival on red percentage for Phase 6 (SB) notably decreased from 25% to 20% for the Interstate 20 interchange intersection, as illustrated in Fig. 6-15. Conversely, at other intersections like UTA Blvd, the impact was more pronounced, with AoR increasing by 20%. Since there were no adjustments made to the signal timing in response to the change in volume, these fluctuations in arrival rates varied across different intersections. Such significant changes in these values suggest that coordinated signal timing may struggle to efficiently accommodate all vehicles on time, potentially resulting in frequent instances of stop-and-go traffic along the corridor.

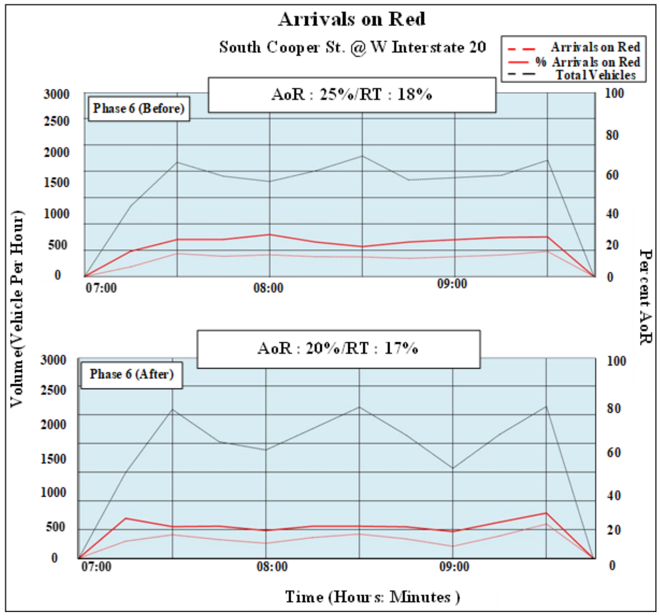


Figure 6-15: Arrivals on Red at W Interstate 20.

In Fig. 6-16, the Park Row Dr PCD (before vs. after) displays essential parameter values such as AoG, GT, and PR. The PCD reveals that for Phase 6 (NB), the AoG is 82%, indicating an increase in AoG. However, despite the traffic congestion, achieving optimal AoG becomes challenging. This underscores the issue of detectors being occupied in queues, leading to higher AoG values even in congested traffic conditions. This scenario serves as a perfect example of how prolonged queue lengths can result in detectors remaining occupied, thereby contributing to elevated AoG. However, such values may not accurately reflect the true traffic conditions and should not be interpreted in isolation using these figures alone. Therefore, while these figures can guide where to focus within the network and assist in assessing the before and after case study, they are complemented by trajectory-based TSD. These diagrams aid in identifying the optimal locations for detectors that can accurately detect queue lengths and provide a better representation of AoG. Further details are provided in the following section 6.2.3.

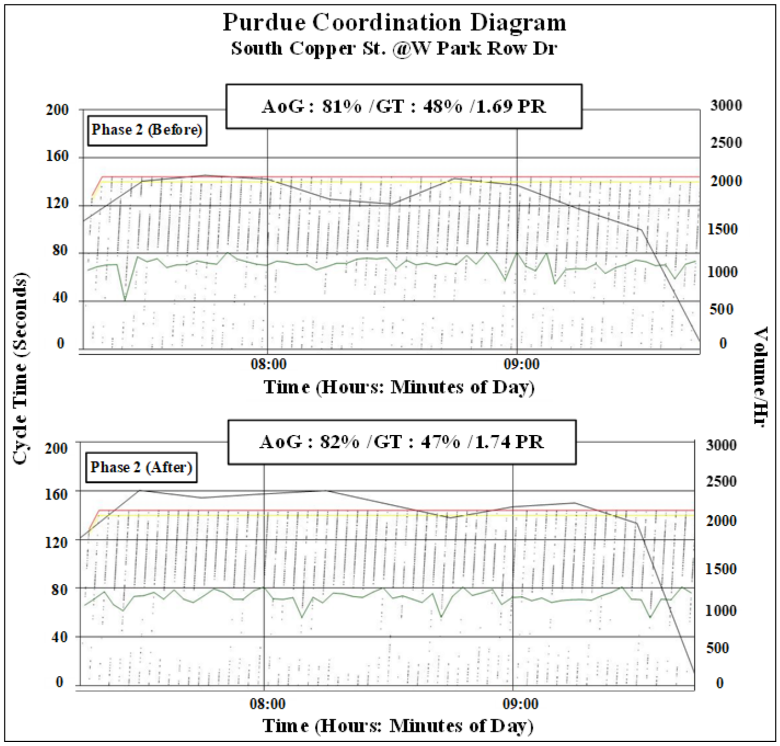


Figure 6-16: Purdue Coordination Diagram for W Park Row Dr.

6.2.3. Analysis of performance metrics from UTAIM

From all the PCDs that were generated for the 16 intersections, a comparative analysis of AoG and GT was conducted with the increased volume. AoG is a measure of how effectively bandwidth promotes progression with coordinated traffic signal parameters designed to promote vehicular flow without stops along arterials (FHWA 2016). Fig. 6-17 shows that AoG is mostly the same despite the volume change. This was overestimated at Intersections 7 (Park Row Dr), 11 (Arkansas Ln), and 13 (Mayfield Rd) which showcased an increase in AoG up to 3%, 5%, and 10% respectively in northbound (A) and around 10% in southbound (B) for Intersection 13(Mayfield Rd).

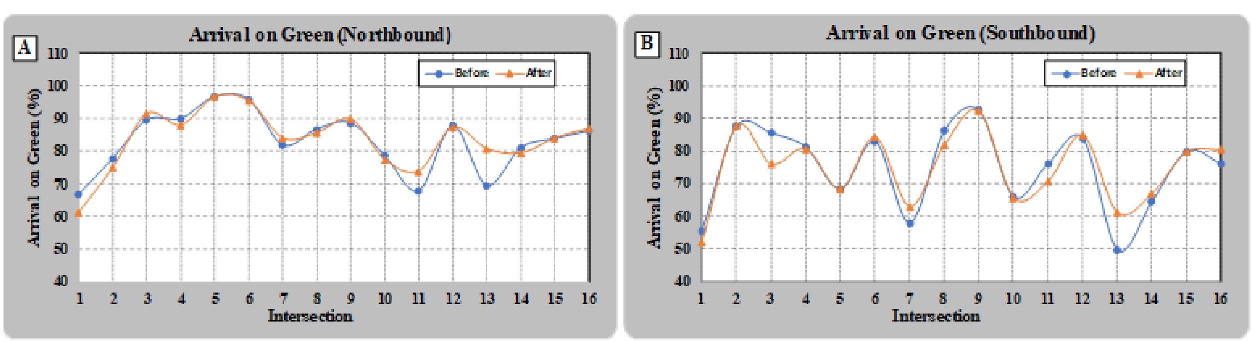


Figure 6-17: Comparative Study of Arrivals on Green.

As shown in Fig. 6-18, the analysis reveals a minor reduction in GT, expressed as a percentage, for both the northbound (A) and southbound (B) directions following changes in volume.

Furthermore, the AoG and GT values produced by both the UDOT-ATSPM and UTAIM systems remained consistent across all intersections, even after the increase in volume scenario.

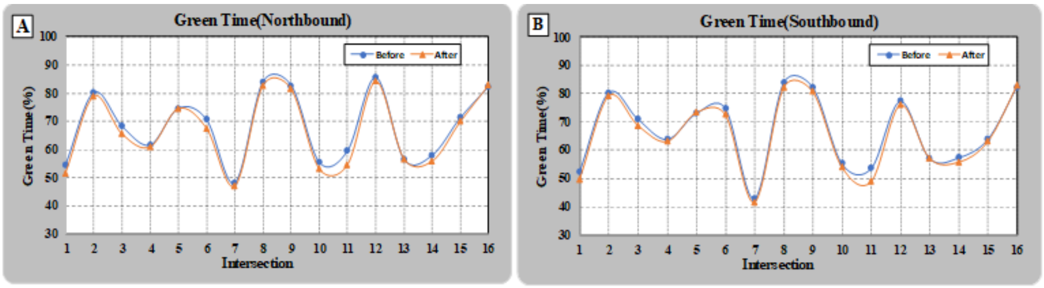


Figure 6-18: Green times for (A) Northbound and (B) Southbound at Cooper Street Corridor.

The PCD for the new increased volume scenario has been reconstructed using the updated parsed dataset. The previously generated PCD has been set aside for comparison purposes. Fig. 6-19 displays the PCDs for the intersection at Park Row Dr in both the before and after scenarios. It is evident from the comparison that the PCDs generated by the UDOT-ATSPM system and UTAIM are similar. However, there is a slight difference in the PCDs: in the scenario with increased volume, the AoG is 82% in the PCD created by UDOT-ATSPM, while in the PCD generated by UTAIM, AoG is 84.1%. Despite this difference, it's notable that in both scenarios, the AoG green has increased. This is an unusual observation, especially considering that other parameters indicate a decrease in the intersection's performance. This provides an opportunity to validate the previous explanation that detectors may have been occupied due to long queues, leading to a higher number of AoG.

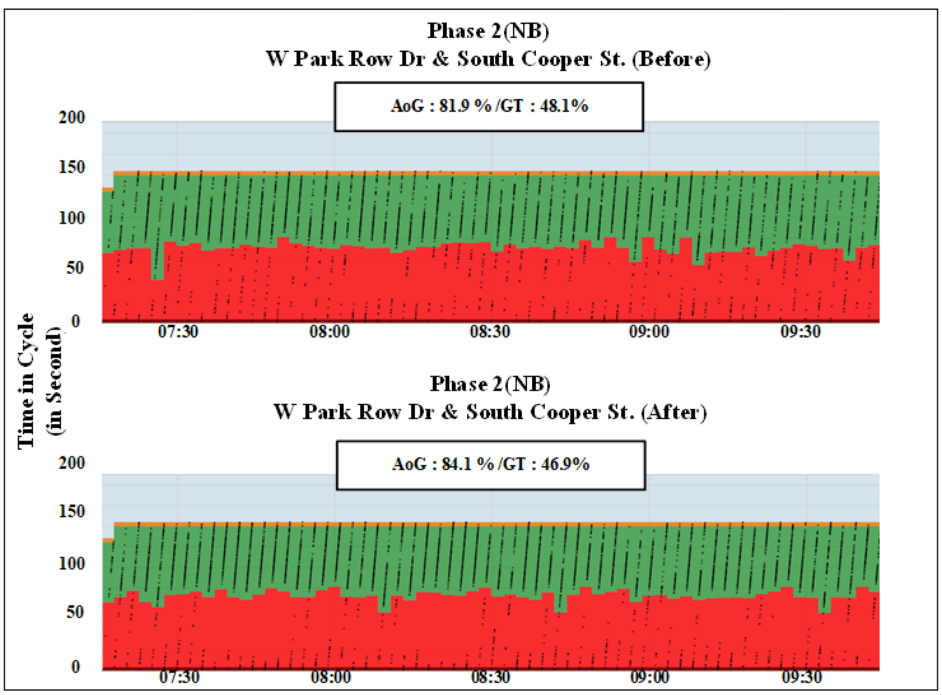


Figure 6-19: Purdue Coordination Diagram for Phase 2 (NB) W Park Row Dr & South Cooper St.

In Fig. 6-20, the TSD for the northbound direction, spanning from Pioneer Pkwy to Interstate 20, is displayed. In regions like A and B, a stop-and-go pattern is noticeable, primarily attributed to queue spillback originating from the Pioneer Pkwy intersection, affecting the adjacent intersection at W Arkansas Ln. The analysis suggests that due to a 15% increase in the original traffic volume, a significant number of vehicles were required to make at least three stops before passing through the intersection. This serves as compelling evidence of cycle failure.

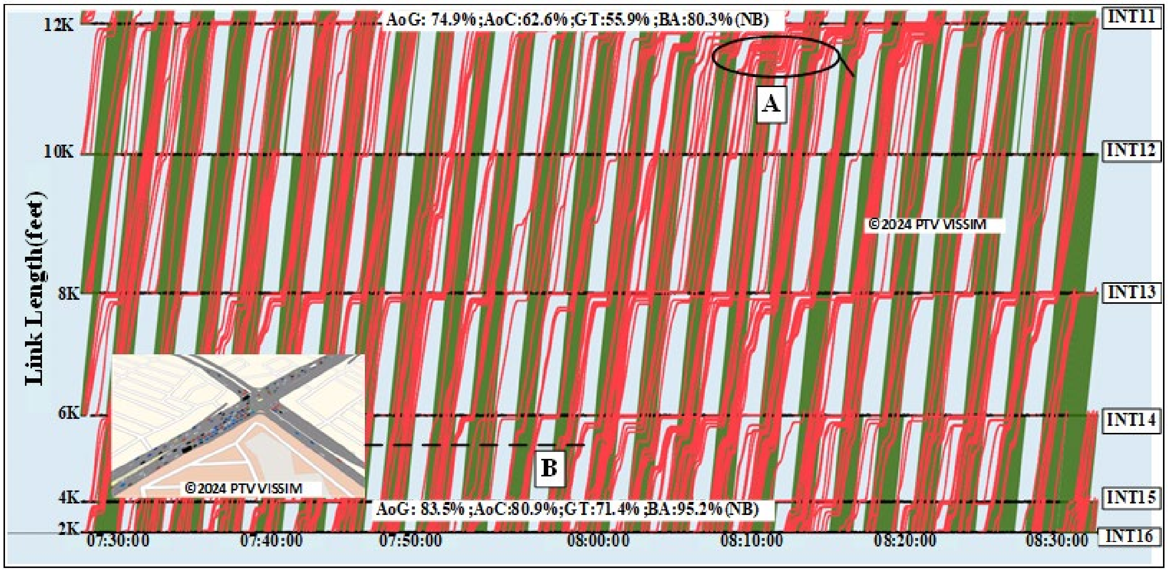


Figure 6-20: Time Space Diagram for Cooper St. Northbound Intersection.

However, between 8:00 and 8:15 am, when long queues are observed at the W Arkansas Ln intersection, the AoG values obtained from UDOT-ATSPM show relatively high values of 83%. Throughout the one hour from 7:30 am to 8:30 am, the AoG value was 74% (see Fig. 6-21). This suggests that despite the intersection experiencing long queues and higher delays, the AoG increases. This underscores the limitation of solely relying on AoG from PCD to assess signal performance. It emphasizes the necessity for additional information, such as trajectory-based TSD, to be considered. In this case, trajectory-based TSD helped identify issues at Intersection 11 (Arkansas Ln). Another contributing factor to these trajectory patterns is the placement of the advanced detectors, situated close to the stop bar. Consequently, many vehicles joining the queue during the red signal phase are not captured by the detectors until they commence movement during the subsequent green signal phase and queue discharge. As a result, the value of AoG is overestimated, as the incoming vehicles are erroneously reported as vehicles arriving during the green signal phase. To address this issue, it is imperative to adjust the placement of the advance detectors to accommodate long queues or leverage the combined strengths of the PCD and the TSD.

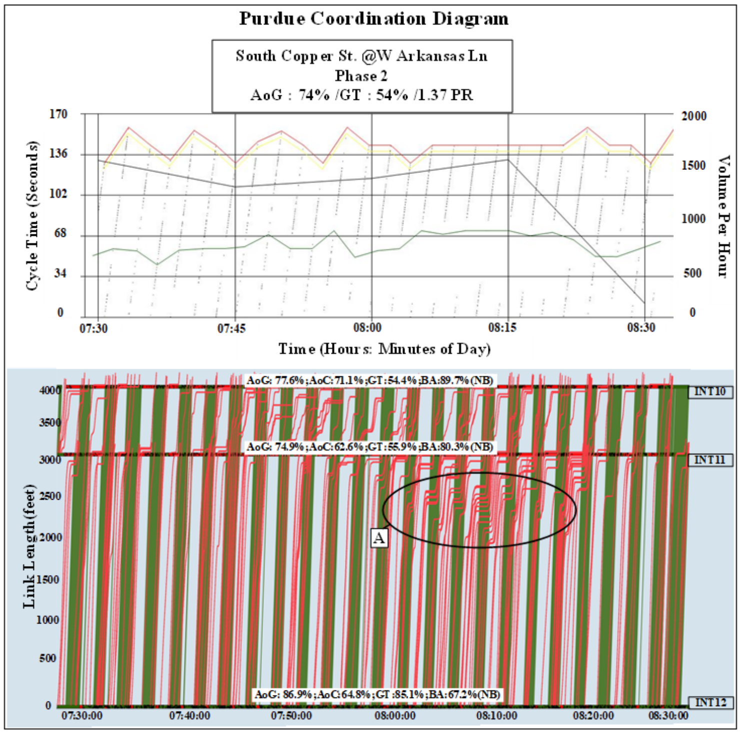


Figure 6-21: Arrivals on green comparison for W Arkansas Ln.

6.3. ANALYZING TRAFFIC SIGNAL PERFORMANCE WITH OPTIMIZED TRAFFIC SIGNAL

Based on the conducted analysis, it's clear that adjusting the signal timing is necessary to facilitate smoother traffic flows, particularly given the increased volume. The current signal timing was established using a common cycle length of 144 seconds. Following calculations based on the HCM method, it was determined that the cycle length should be extended to accommodate the higher volume of vehicles. To address this, the cycle length was recalculated using the HCM method, which does not consider the potential impact of downstream congestion (Urbanik et al. 2015). Equation 1 was employed to calculate the optimal cycle length (C).

$$C = \frac{L}{1 - \frac{min(CS,RS)}{RS}} \quad [1]$$

Where,

C = cycle length (s),

L = lost time per cycle (s),

CS = critical sum of traffic volumes from the critical movement analysis (veh/h),

RS = reference sum flow rate =1710. $PHF.f_a$ (veh/h),

f_a = area type adjustment factor (0.90 if CBD, 1.00 otherwise).

After calculations, the new cycle length was determined to be 160 seconds. Subsequently, split calculations were conducted to evenly distribute this cycle length among each phase. The revised cycle length was then implemented across all 16 intersections to allocate additional GT for each phase. These updated split values were incorporated into the simulation, and a new simulation run was executed. Following the completion of the new simulation scenarios, various graphs were generated based on PTV VISSIM node analysis. The optimization efforts have yielded a notable improvement in the average vehicle delay graph. The major changes are evident in the northbound approach of Pioneer Pkwy, southbound Arkansas Ln, and Arbrook Blvd. However, upon examining the TSD, it is apparent that the trajectories at Intersection 14 (Arbrook Blvd) appear congested, which may lead to concerns about the effectiveness of the optimization measures. Nevertheless, this concern can be addressed by Fig. 6-22. Considering the significant decrease in average delay, which has decreased by an impressive 36% overall after optimization queue length was also improved, particularly in the Pioneer Pkwy and Arkansas Ln. By increasing the minimum green duration, more vehicles were able to proceed through the intersection during each green cycle, leading to a significant reduction in queue length as shown in Fig. 6-22B.

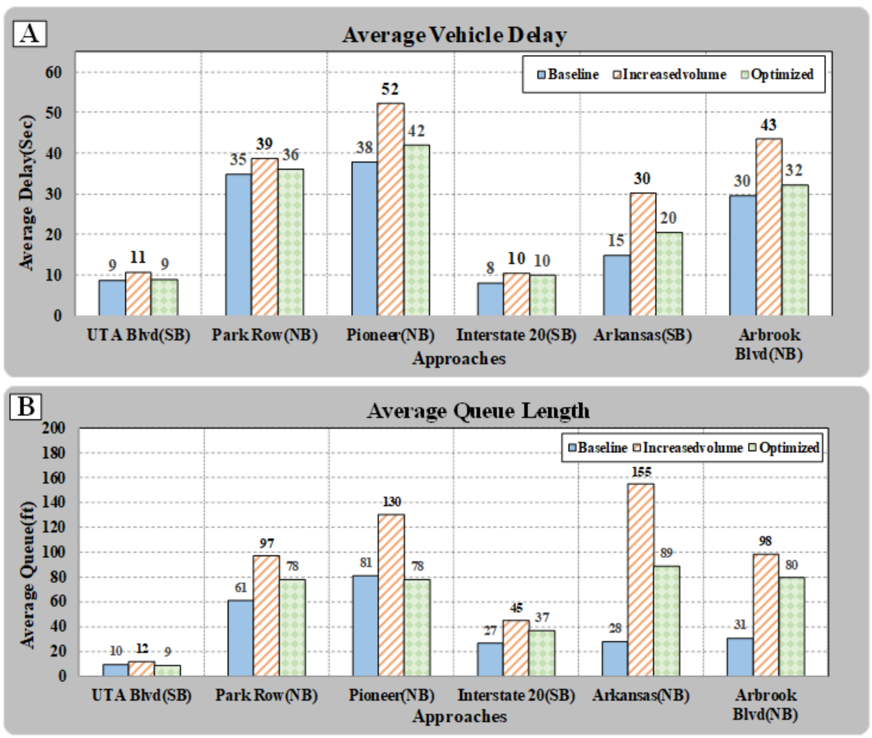


Figure 6-22: Comparison of Average Vehicle Delay and Queue Length for Different Approaches.

Additionally, Fig. 6-23 illustrates the reduced travel time for the northbound approach by a major 84 seconds, while the slight increase in travel time for the southbound approach of 6 seconds is hardly significant.

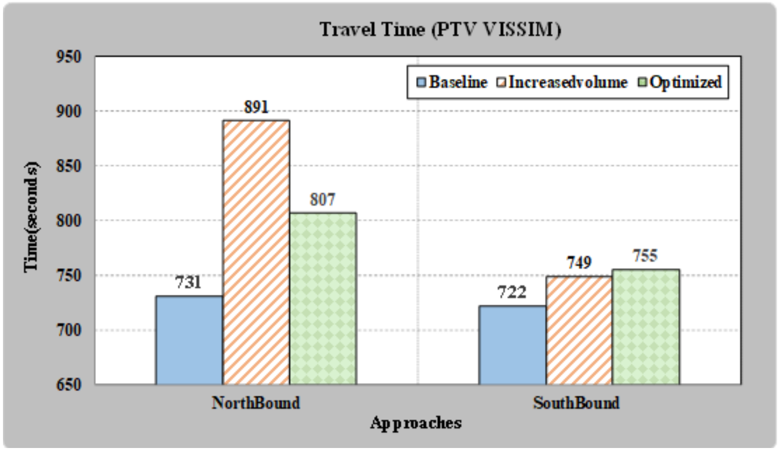


Figure 6-23: Travel Time Comparison at Cooper Street Corridor.

Table 6-6 reveals a decline in the LOS following an increase in volume for three intersections- Abram St, UTA Blvd, and Pioneer Pkwy. However, after optimizing the cycle length based on the critical volume for the worst-case scenario, significant improvements are observed at two intersections, namely UTA Blvd and California Ln. Although the LOS at the Division St intersection deteriorates from grade C to D, the other intersections remain unchanged. These findings indicate that the optimization approach utilizing the cycle length has yielded better promising results, demonstrating its effectiveness in improving traffic flow and overall intersection performance. Furthermore, it's important to note that the goal of the optimization was not solely focused on improving one specific intersection, as all intersections were coordinated. The optimization was performed with a general aim to enhance overall traffic flow across all intersections. It's crucial to remember that these LOS assessments were conducted considering the increased volume, and LOS ratings of B, C, and D are still considered acceptable compared to LOS F.

Table 6-6: Level of Service obtained from VISSIM.

Intersection	Baseline Scenario	Increased Volume Scenario	Optimized Scenario
Division St	C	C	D
Main St	C	C	C
Abram St	B	C	C
UTA Blvd	B	C	B
Nedderman Dr	B	B	B
Mitchell St	C	C	C
Park Row Dr	C	C	C
Inwood Dr	C	C	C
Snooty Fox Dr	C	C	C
Pioneer Pkwy	C	D	D
Arkansas Ln	C	C	C

California Ln	C	C	B
Mayfield Rd	D	D	D
Arbrook Blvd	C	C	C
Pleasant Ridge Rd	C	C	C
Interstate 20	C	C	C

Finally, a comparison between the TSD from both scenarios—Before optimization and after optimization—was conducted. In Fig. 6-24A, it's evident that in the scenario before optimization, Intersection 11 (Arkansas Ln) experienced significant vehicle delay and longer queue lengths (region A in Fig. 6-24A). Similarly, Intersection 14 (Arbrook Blvd) also exhibited extended queues, with vehicles unable to clear the intersection within one cycle, leading to some split failures. However, after optimization, as depicted in Fig. 6-24B, queue lengths have noticeably reduced at both Intersections 11(Arkansas Ln) and 14 (Arbrook Blvd). However, there's a possibility of other approaches, such as side streets, experiencing backups with longer queues, this case study aimed to illustrate how traffic signal performance at any intersection can be evaluated using graphs obtained from these two systems. While VISSIM results were utilized as ground truth for validation in the analysis, once the model is appropriately calibrated, any scenario can be replicated, and PCD along with TSD should help identify any traffic performance improvements needed at any intersection.

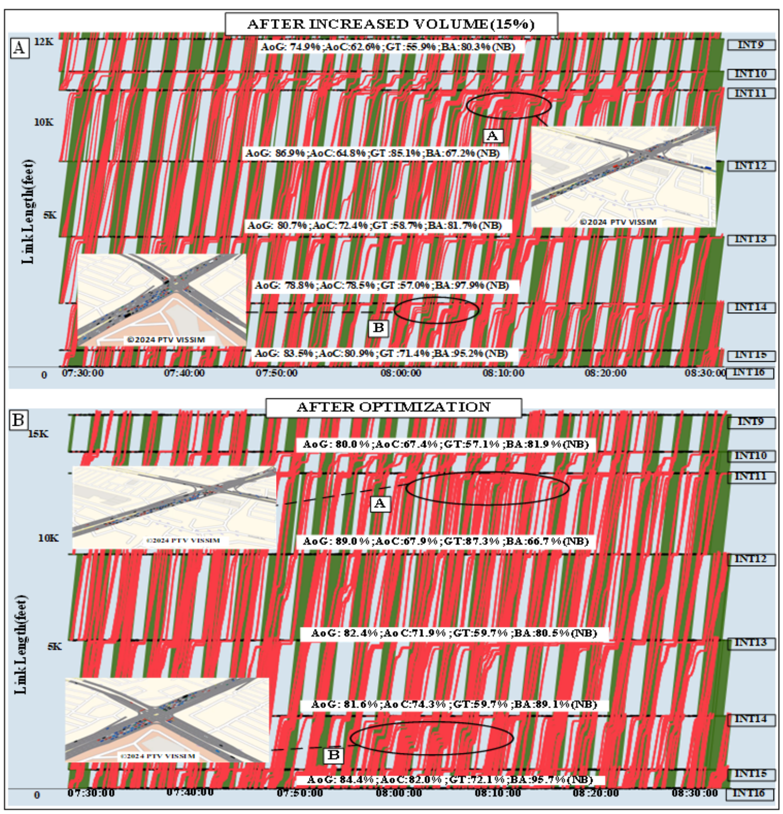


Figure 6-24: Comparison of the Time Space Diagram Before and After Signal Optimization.

6.4. DISCUSSION

- This chapter reveals that the UDOT-ATPSM tends to overestimate the AoG and GT for long queues when advance detectors are occupied. Additionally, vehicles arriving during the red signal phase are not considered until the queue is cleared.
- Increasing the volume resulted in an increase in queue length, control delay, and travel time compared to the original volume under the same traffic signal timing.
- To address this issue, the HCM methods were employed to optimize the cycle length for the critical volume. The cycle length was adjusted from 144 seconds to 160 seconds, aiming to improve signal timing and regulate the smooth flow of traffic.
- Going forward, a case study will be conducted to analyze complex traffic conditions, such as the presence of preemption (rail and emergency vehicles), freight signal priority. This study aims to assess the effectiveness of signal timing strategies and optimization techniques in managing these challenging traffic situations.

CHAPTER 7:
A CASE STUDY OF USING ATSPM-IN-THE-LOOP SIMULATION FOR COMPLEX TRAFFIC SIGNAL TIMING DESIGN

The project team conducted a case study using ATSPM-In-The-Loop simulation for complex traffic conditions, such as preemption, TSP, and emergency vehicles. This task is crucial for assessing the potential negative impacts of special signal requests on general traffic, such as temporary queue spillback. VISSIM default traffic signal emulator, the RBC controller, lacks the necessary fidelity. Therefore, the project team adopted the advanced software-in-the-loop simulation (SILS) package to integrate the VISSIM simulation engine with a hardware signal controller at specific intersections to address this issue. In these chosen intersections, traffic simulation will transmit real-time detector calls via the special interfacing program to the SILS package and receive feedback on the decisions made by the SILS control software. The control decision will then be implemented to control simulated traffic. The adopted MAXTIME SILS package was developed by Q-Free Inc® and can output the corresponding events for the ATSPM-In-The-Loop framework for the evaluation and design of complex traffic timing strategies. The case study aims to demonstrate new opportunities to evaluate the complex traffic signal operations (e.g., preemption, TSP, etc.) under congested traffic conditions with the ATSPM systems.

7.1. SETTING UP MULTIMODAL TRAFFIC SIGNAL OPERATIONS IN THE Q-FREE'S MAXTIME CONTROLLER

Multimodal traffic operations in this context mean preemption for emergency vehicles, trains, as well as signal priorities such as transit/freight signal priority operations. This section outlines a step-by-step procedure for setting up preemption and Freight Signal Priority in MAXTIME and VISSIM.

7.1.1. Configuring Preemption in the MAXTIME Controller

Step 1: In the simulation model, each intersection in need of complex traffic signal operations must be coupled with a separate copy of the MAXTIME SILS signal emulator. On MAXTIME's home webpage, users need to first navigate to the Controller tab on the left side, select Preempt Configuration, and then Preempts. For a better view, choose the table format option on the far right. Refer to Fig. 7-1 for visual guidance.

Figure 7-1: Setting up preempts in MAXTIME Controller.

Step 2: Different types of preempt can be selected in MAXTIME, including Rail Road and Emergency Vehicle. For a train preemption, it is needed to first ensure enabling that preempt and then select "***Rail Road"*** from the types.

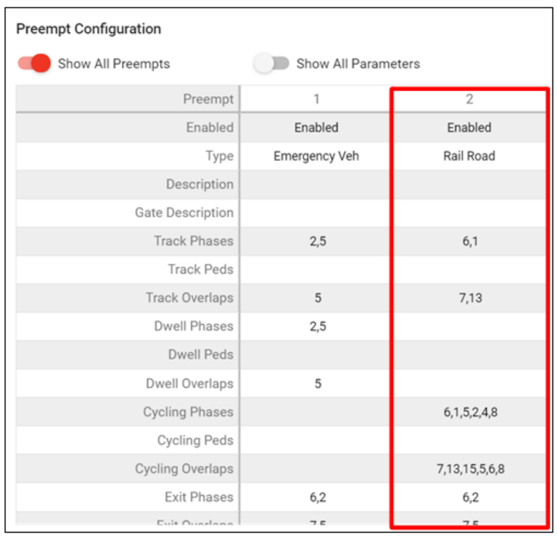


Figure 7-2: Enabling multiple preempts in the MAXTIME controller.

Step 3: Set up three phases for a preemption: Track Phase, Dwell Phase, and Exit Phase.

- Track Phase: Refers to phases that need to be clear because they have a direct conflict with the railroad. For example, in Fig. 7-3, the Track Phase includes phases 6 (Southbound thru) and 1 (Southbound left), which start as soon as the preemption is activated. This ensures no vehicle is on the track and any vehicle queued on the track clears before the train arrives and the rail gate closes.
- Dwell Phase: Refers to movements allowed while the train is passing the intersection. The intersection transitions to Dwell Phases once the gates are down. Dwell Phases use a similar approach to Cycling Phases but differ in that Cycling Phases use the current pattern's base phase plan and adjust timing based on vehicle demand, while Dwell Phases serve based on

the minimum green time or the dwelling time, whichever is longer. Fig. 7-2 shows that cycling phases 6, 5, 1, 2, 4, and 8 are used.

- Exit Phase: Begins once the train has passed. Certain phases are prioritized based on the design to ensure smooth traffic flow. For example, the design prioritizes phases 6 and 2 since Cooper Street is a major street, and the train blocks through traffic for a while, making it important to prioritize through traffic on this major street.

Refer to Fig. 7-3 for the phase diagram of Main Street & Cooper intersection with the preemption setup. This intersection is unique with 16 phases and multiple overlaps, and it employs a double-cycle approach where the left turn phase on Cooper Street is skipped every second cycle.

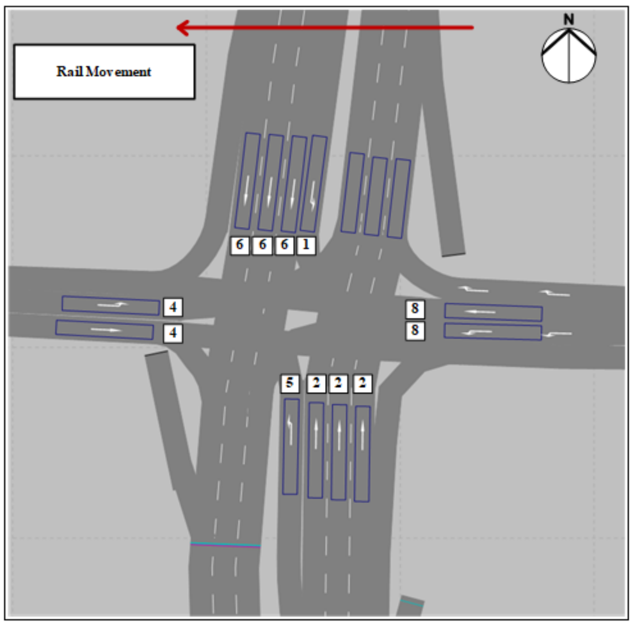
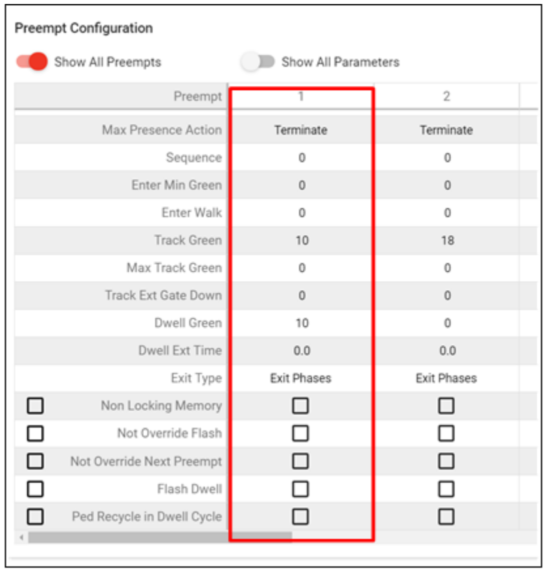


Figure 7-3: Cooper & Main St. intersection layout.

Step 4: Once the phases are configured, minimum timing can be set for both Track Green and Dwell Green. Refer to Fig. 7-4.

- Track Green: Refers to the fixed time in seconds for the Track Phase selected in the preempt settings. The green light will end after this time has expired.
- Dwell Green: Refers to the time in seconds during which the Dwell Phases will be served. If the dwell time is set to 0, the minimum green time for those phases will be used. If a specific dwell time is set, the phases will be served for that duration, and the green light will be extended based on the vehicle detector.

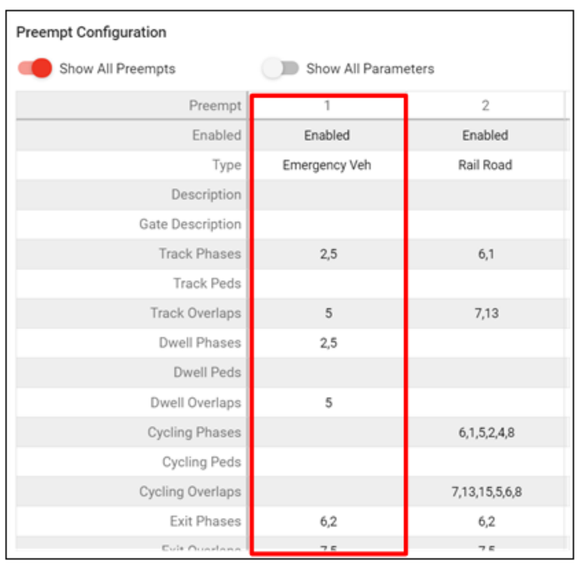

Preempt Configuration

Show All Preempts | Show All Parameters

Preempt	1	2
Max Presence Action	Terminate	Terminate
Sequence	0	0
Enter Min Green	0	0
Enter Walk	0	0
Track Green	10	18
Max Track Green	0	0
Track Ext Gate Down	0	0
Dwell Green	10	0
Dwell Ext Time	0.0	0.0
Exit Type	Exit Phases	Exit Phases
☐ Non Locking Memory	☐	☐
☐ Not Override Flash	☐	☐
☐ Not Override Next Preempt	☐	☐
☐ Flash Dwell	☐	☐
☐ Ped Recycle in Dwell Cycle	☐	☐

Figure 7-4: Dwell and track green setup.

Step 5: Setting up preemption for emergency vehicles. It is like train preemption except that track phases, dwell phases must be set differently because they are on the same approach. As shown in Fig. 7-5, phases 2 (Northbound Thru) and 5 (Northbound Left) are prioritized. This prioritization is because, in the simulation model, the emergency vehicle is placed in the northbound direction on Cooper Street. To ensure high-priority green time is provided to this movement, phases 2 and 5 are selected. Refer to Fig. 7-5.


Preempt Configuration

Show All Preempts | Show All Parameters

Preempt	1	2
Enabled	Enabled	Enabled
Type	Emergency Veh	Rail Road
Description		
Gate Description		
Track Phases	2,5	6,1
Track Peds		
Track Overlaps	5	7,13
Dwell Phases	2,5	
Dwell Peds		
Dwell Overlaps	5	
Cycling Phases		6,1,5,2,4,8
Cycling Peds		
Cycling Overlaps		7,13,15,5,6,8
Exit Phases	6,2	6,2

Figure 7-5: Emergency Vehicle Preemption Setup in MAXTIME.

7.1.2. Configuring Priority (a.k.a., low-priority preemption) in MAXTIME

Step 1: Go to Controller > Prioritor Configuration. Ensure that the Prioritor Configuration is in active mode. Refer to Fig. 7-6.

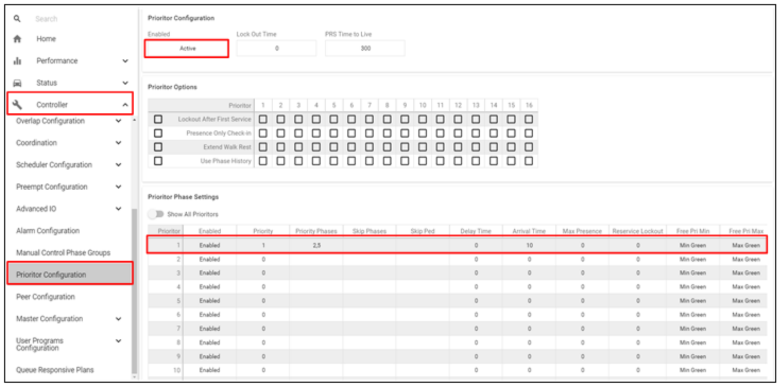


Figure 7-6: Prioritor setup in MAXTIME controller.

Step 2: Depending on the number of prioritor, different prioritor can be configured. In Fig. 7-6, one prioritor is configured. For vehicles that need priority, such as buses and freight trucks in the northbound direction, phases 2 and 5 should be considered. Therefore, these phases are placed in the Priority Phases.

Step 3: Set up the arrival time based on how long it will take for the vehicle to reach the intersection once it is detected and the prioritor is activated. This setup ensures the system is enabled to consider priority for the vehicle.

7.1.3. Configuring preemption/priority Detectors.

Setting up preemption and priority detector is crucial as if and only if set up correctly will request for right calls. The following steps can be followed to ensure that the pri/pre detectors are correctly set up.

Step 1: Go to Controller > Detector Configuration > Pri/Pre-Detectors. For a better view, choose the table format option on the far-right corner. Refer to Fig. 7-7 for visual guidance.

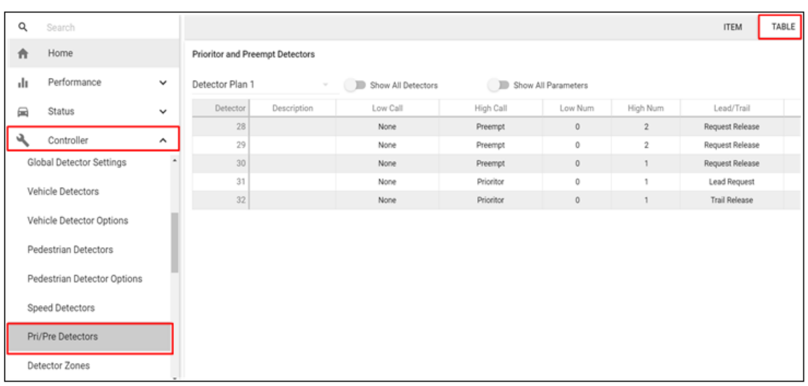


Figure 7-7: Pri/Pre detector setup in MAXTIME controller.

Step 2: Utilize the non-stop bar detector numbers to establish preempt/Prioritor calls.

- In the "High Call" field, select "Preempt."
- For the corresponding "High Num," use the index value for that preempt. If multiple preempts are present, separate detectors should be used. For instance, if two preempts are

configured where 1 is for emergency vehicles and 2 is for railroad preempt, then different detectors should be set up for different "High Num" values.

- From Fig. 7-7, it is evident that five different detectors are set up for preempt and prioritization. For example, detector 28 has a "High Call" set to "Preempt," meaning it is used for preemption with "High Num" 2. This indicates that preemption type 2 will be called if detector 28 is activated. Fig. 7-2 shows that preempt 2 refers to the Rail Road. Detector 28 is placed on the westbound railroad track, as demonstrated in Fig. 7-8A.
- Similarly, detector 30 is also used for preempt with "High Num" 1. Fig. 7-5 shows that preempt 1 is for emergency vehicles. The placement of detector 30 is in the northbound direction, as shown in Fig. 7-8B.
- Additionally, detectors 31 and 32 are used to call the prioritor with "High Num" 1. Detector 31 is placed right after the preempt detectors and activates only for the vehicle type "Freight." Detector 31 serves as the check-in detector for the prioritor, with its state as "Lead Request," while detector 32 serves as the check-out detector with its state as "Trail Release." Further explanation of these detector states is provided in the following section.

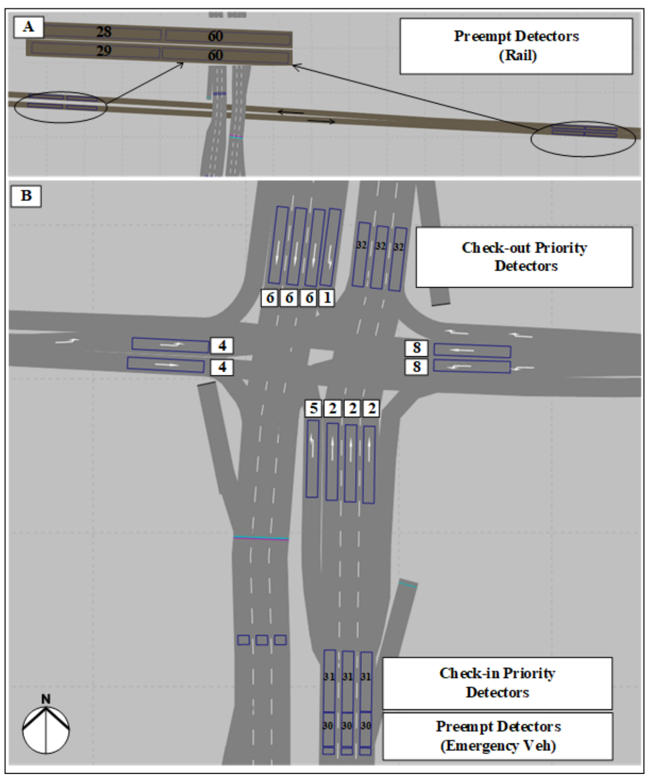


Figure 7-8: Pri/Pre detector setup layout in VISSIM.

In setting up the prioritor, distinct detectors should be allocated for each prioritor call to differentiate between prioritization requests effectively. The behavior of the detector depends on the Lead/Trail setup, which determines the state of the input that activates the detector. There are five different types of states:

- Lead Request: When the lead end of the detector is activated, it requests the prioritor plan (i.e., the request to call the prioritor is triggered when the vehicle's front bumper enters the detection zone).
- Lead Release: When the lead end of the detector is activated, it releases the prioritor plan (i.e., the call to release/drop the prioritor is triggered when the vehicle's front bumper enters the detection zone).
- Trail Request: When the trail end of the detector is activated, it requests the prioritor plan (i.e., the request to call the prioritor is triggered when the vehicle's rear end exits the detection zone).
- Trail Release: When the trail end of the detector is activated, it releases the prioritor plan (i.e., the call to release/drop the prioritor is triggered when the vehicle's rear end exits the detection zone).
- Request Release: The detector requests the prioritor or preempts timer when activated and releases it when deactivated. This means the prioritor/preempt request is triggered as soon as the vehicle's front bumper enters the detection zone and is held as long as the vehicle is within the zone. The request is dropped, and the prioritor/preempt is released as soon as the vehicle's rear end leaves the detection zone. For preemption, the request release state is considered the default, and all other states are also treated as request/release.

Step 3: Navigate to Controller > Advance IO > Cabinet Configuration > Input Points. Change the input control type for the index similar to the detector number used for Pri/Pre-Detector to "Prioritor/Preempt Detector". This ensures that the input points are configured correctly to correspond with the Prioritor/Preempt detectors set up in the system. Refer to Fig. 7-9.

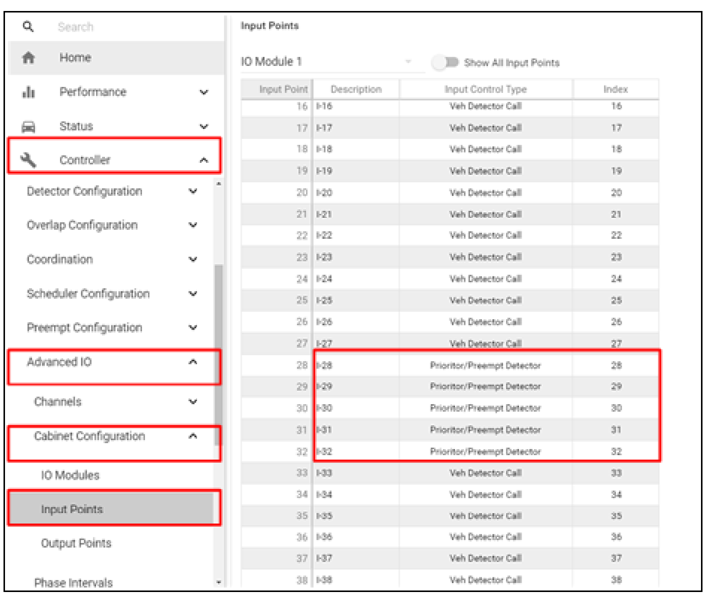


Figure 7-9: Cabinet configuration for input points in MAXTIME.

7.2. EVALUATION OF THE BASELINE TRAFFIC SIGNAL PERFORMANCE USING THE ATSPM-IN-THE-LOOP TRAFFIC SIMULATION MODEL

In this case study, all sixteen intersection traffic volumes, signal timing, and routing decisions were kept the same as in TM-6 for the baseline scenario. Since our main motive was to evaluate the traffic signal performance in complex traffic scenarios, preemption (Rail and Emergency vehicles) and TSPs were added to the traffic simulation model. Preemptions are designed primarily to prevent fatalities by overriding any signal priority and terminating an extended green light in favor of a green light for the vehicle requesting a green (Urbanik et al. 2015). This mechanism aids emergency vehicles and rail by reducing the time for them to reach their destinations, as it clears a path through traffic signals. Preemption systems work by communicating with traffic signals to request priority or clearance, thus reducing the need for emergency drivers to slow down, stop, or maneuver around other vehicles at intersections. These systems can be activated by various methods, such as radio, infrared, GPS, or cellular signals. Fig. 7-10 shows the configuration of rail preemption (Fig. 7-10A) and emergency vehicle preemption (Fig. 7-10B) in the model. The preemption path is cleared giving green signals to the approaching intersection.

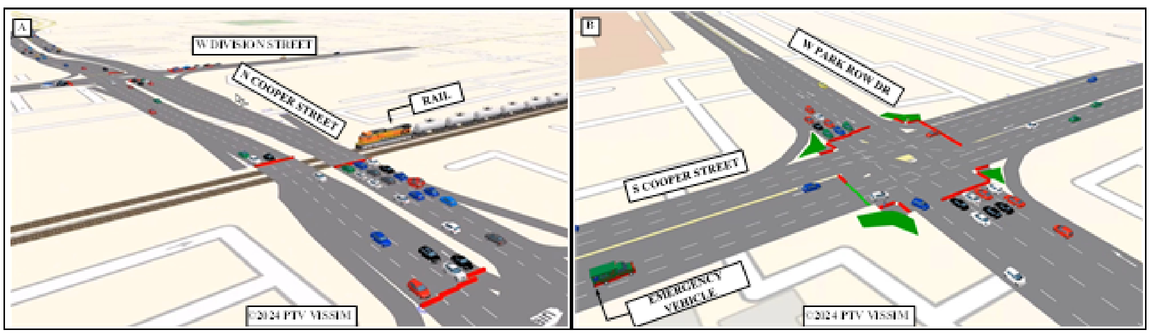


Figure 7-10: Preemption (A) Rail and (B) Emergency vehicle setup in PTV VISSIM.

On the other hand, TSP is used to provide extra green time at intersections for larger, street-running transit vehicles such as freight trucks, buses, light rail, and streetcars. By preventing these vehicles from being stopped at red lights, this added green time can help those favored vehicles arrive at their destinations on time and prevent traffic delays and congestion. Nonetheless, the TSP works differently from the preemption. The green extension or red truncation is conditional and may not be granted every time. As a result, the favored vehicle may still have to stop for the red light at certain intersections even though a TSP request is placed. Fig. 7-11 shows the same freight truck's priority request was granted at one intersection but declined at another intersection. In contrast, preemption requests are unconditional.

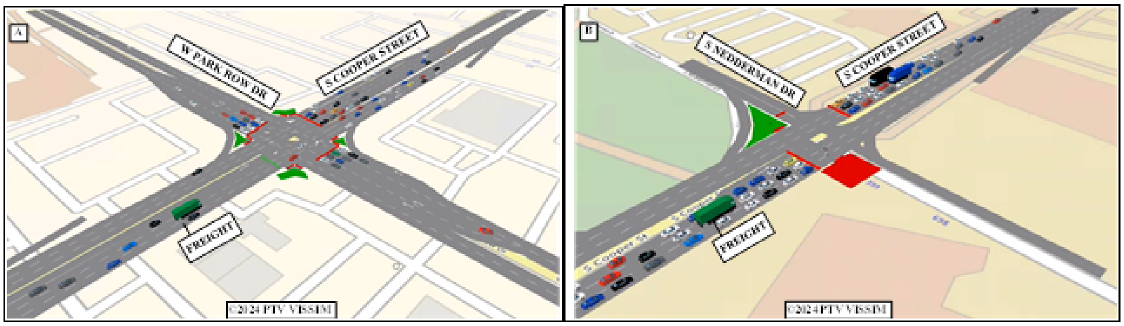


Figure 7-11: Demonstration of granted or declined traffic signal priority at intersections.

In this project, a rail preemption was added at Main Street in the westbound direction to clear vehicles on the rail track when a train is approaching. Another two emergency vehicles and eleven trucks with enabled TSP capability were added from Pioneer Pkwy at Cooper Street to the north. These add-on special vehicles necessitate preemptions and traffic signal priorities. A background clock-based traffic signal coordination plan was employed and so all intersections share a common cycle length. A few intersections along Cooper Street were selected for evaluation of those complex traffic signal operations on the background traffic.

7.2.1. Analysis of Performance Metrics from VISSIM Outputs

The simulation parameters for MOE evaluation were kept consistent with TM-6. By examining the nodes, metrics like total queue length, average queue length, total vehicle delay, average vehicle delay, and LOS were automatically calculated for each intersection, considered a *"pseudo ground truth"*. Table 7-1 indicates that the average queue length and vehicle delay at Park Row Road and Pioneer Pkwy are longer. This can be attributed to the movement of emergency vehicles and TSP from Pioneer Pkwy to other intersections on Cooper, leading to queue development in Pioneer and Park Row due to their proximity. These results are further justified with the LOS rating of D and C respectively compared to B of Main Street & UTA Blvd.

Table 7-1: Results obtained from PTV VISSIM

Intersection	**Main St**		**UTA Blvd**		**Park Row Dr**		**Pioneer Pkwy**	
Movement	NBT	SBT	NBT	SBT	NBT	SBT	NBT	SBT
Queue length	8.60	5.82	17.72	20.67	109.81	69.24	31.31	48.45
Average Queue Length	3.42	2.88	6.76	15.39	79.32	36.01	44.23	44.87
Vehicle Delay	14.28	9.31	12.65	14.02	32.99	43.15	20.37	29.41
Average Vehicle Delay	14.23	13.81	11.30	19.51	39.32	43.23	32.30	34.10
LOS	B		B		D		C	

7.2.2. Analysis of Performance Metrics from UDOT-ATSPM

The approach of Pioneer Pkwy needs a split time of 125.20 seconds to clear queues. This relatively long split time can be attributed to the need to accommodate higher traffic volumes or longer cycle lengths to ensure efficient progression and reduce the likelihood of congestion. Conversely, Park Row Dr has a Purdue Split score of 17. This is relatively high which suggests that the traffic signal phase is not effectively serving the demand, leading to poor utilization of the green time. At the intersection of Main Street, traffic showed maximal and average waiting times, likely due to the high traffic volumes, and frequent signal transitions, causing bottlenecks. The presence of the rail track between Division Road and Main Street further complicates the traffic flow, leading to additional delays. The rail track introduced periodic interruptions to surrounding traffic and increased waiting times contributing to overall congestion. The results are summarized in Table 7-2.

Table 7-2: Performance Metrics for the Intersection.

Intersection	**Main St**		**UTA Blvd**		**Park Row Dr**		**Pioneer Pkwy**	
Performance Metrics	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)
Approach Delay (sec)	2	2	4	6	8	20	7	16
Total Delay (hr.)	1.90	1	3.40	3.40	9.3	11.4	5.20	10.40
Approach Volume	3061	2234	3354	2271	4409	2090	2293	2417
Arrivals on Red (AoR-%)	22	13	20	24	20	44	21	42
Red Time (RT-%)	25	23	39	38	27	59	44	47
Arrivals on Green (AoG-%)	78	87	80	76	80	56	79	58
Green Time (GT- %)	75	77	61	62	48	41	56	53
Platoon Ratio (PR)	1.04	1.13	1.31	1.23	1.67	1.37	1.4	1.09
Average Split	94.30	97.07	75.60	68.30	86.50	86.10	125.20	125.20
Purdue Split (PR)	3	3	1	1	17	0	10	2
Average Wait(sec)	163.40	11.60	26.50	39.30	63.90	72.40	50.70	54.20
Max Wait(sec)	272.00	25.90	68.60	64.60	77.30	85.70	78.70	64.70

The PCD is critical to help users understand signal timing by showing vehicle arrivals alongside the cycle length. Fig. 7-12 illustrates the PCD at Inwood Drive, highlighting the key MOEs like AoG, GT, and PR. These parameters allow users to assess intersection efficiency and adjust traffic performance to reduce delays and congestion. The PCD at Inwood Drive shows a higher AoG for Phase 2 (northbound) at 81% compared to Phase 6 (southbound) at 74%. This implies that a greater amount of time is allotted for traffic heading north. High AoG values may not accurately reflect actual traffic conditions due to long queue lengths occupying detectors, skewing the data. The statement above highlights observations from TM-6. This shows the need for the approach of using trajectory-based TSD from UTAIM, which can be integrated with the PCD. This integrated approach allows for a more thorough evaluation of whether elevated AoG values correlate with poor intersection performance. By analyzing both PCD and TSD data, one can better understand the impact of long queue lengths on detector occupancy and AoG values, providing a clearer picture of actual traffic conditions and intersection efficiency.

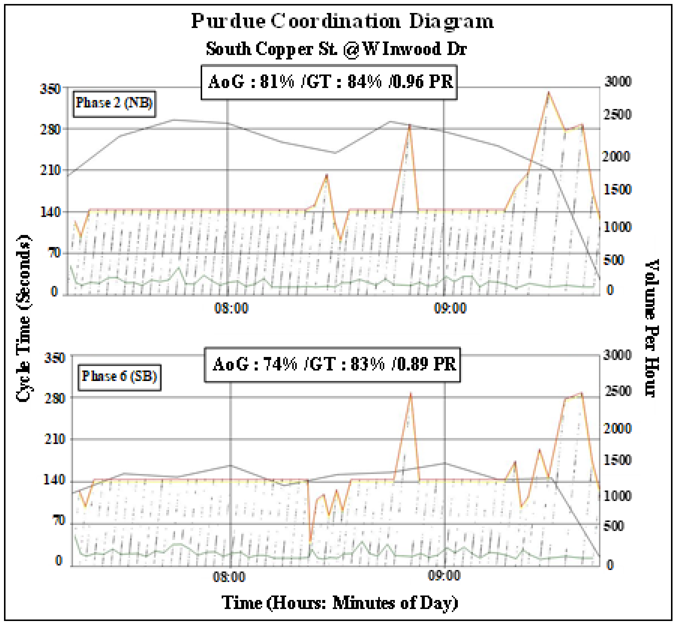


Figure 7-12: Purdue Coordination Diagram for W Inwood Dr.

Fig. 7-12 shows that the PCD diagram remains consistent with a cycle length of more than 140 seconds until 8:15 am, as designed in the simulation. However, the effect of preemption becomes evident thereafter, demanding more green time than usual in the northbound phase. This adversely affects the southbound phase, resulting in a deficit of green time compared to the regular scenario. Additionally, the preemption effect is validated further after 9:15 am, demanding even more green time. However, this graph only shows the fluctuation of the cycle length, which is a limitation, as it does not indicate the reasons behind these changes, such as preemption, ped call, or priority requests, which can be observed in UTAIM.

Fig. 7-13 shows the Preemption details at Main Street where preemption was requested, received, dwell time, and the time to service. The Dwell time will be activated until the Minimum Duration time has been satisfied. The Preemption Detail Graph (David Bremer 2019) offers a comprehensive analysis of how preemption events affect traffic signal operations, prioritizing emergency vehicles, trains, or public transit by temporarily overriding the normal signal sequence. It depicts time intervals when preemption occurs along the x-axis and signal phases on the y-axis, using distinct colors to differentiate preemption from regular operations. The graph details how normal signal phases are interrupted, altering green, yellow, and red times. Additionally, it provides insight into the duration and frequency of preemption events, crucial for understanding their impact on traffic flow. This gives a potential opening to use this data to optimize signal timings, balance priority traffic with regular flow, and coordinate across intersections, enhancing emergency response and minimizing disruptions to regular traffic.

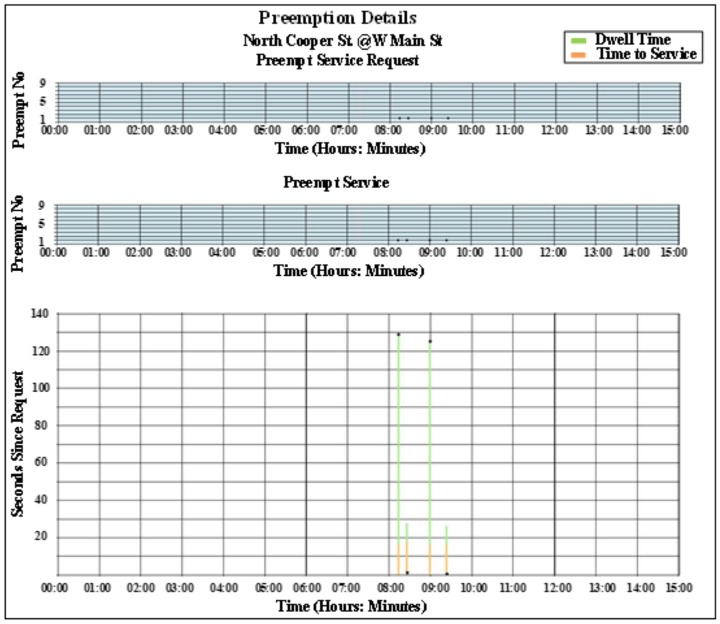


Figure 7-13: Preemption Details for W Main St.

The Purdue Phase graph (David Bremer 2019) is a tool used to evaluate the efficiency of traffic signal phases by categorizing how and when they terminate. It displays phase terminations as Gap Outs (indicating no traffic demand), Max Outs (indicating traffic demand exceeds max green time), and Force Offs (indicating signal coordination limits). By analyzing these terminations, traffic engineers can identify inefficiencies in signal timings, optimize green times, and improve traffic flow coordination across intersections. Fig. 7-14, a distinct number of pedestrians and Force Off terminations can be observed. Increased pedestrian activity and a higher number of Force Off terminations indicate that pedestrian phases are consuming more green time, potentially impacting vehicular traffic flow. This suggests that the signal timings might be heavily influenced by pedestrian crossings, which can reduce the green time available for vehicular phases and necessitate adjustments to better balance pedestrian and vehicle needs. The yellow dots represent unknown reasons for terminating the phases, which, in our case, can be due to the arrival of preemption and TSP.

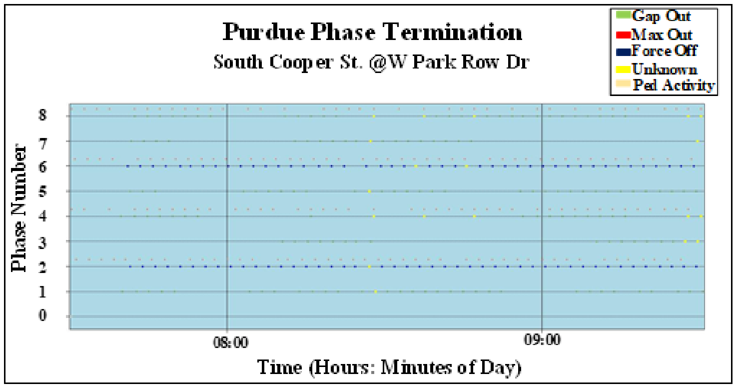


Figure 7-14: Purdue Phase Termination for W Park Row Dr.

The Purdue Split Failure diagram includes a comprehensive overview of split failures, reporting counts, percentages, and classifications (David Bremer 2019). It uses stop bar presence detection to calculate the green occupancy ratio (GOR) and red occupancy ratio (ROR). When both ratios exceed 80%, the phase is considered a split failure for that cycle. Figure 7-15 illustrates a total split failure of 15 for Park Row Dr. This signifies that during those cycles, the actual signal timing deviated significantly from the planned timing, potentially leading to traffic congestion, delays, or inefficiencies at the intersection. The Purdue Split Failure analysis evaluates the performance of signal timing plans by comparing planned and actual splits for each phase. It highlights discrepancies, which are crucial for identifying issues in signal timing that need adjustment to align with traffic demand.

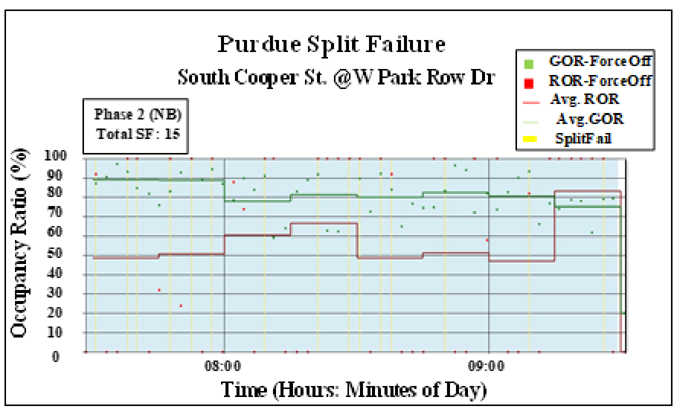


Figure 7-15: Purdue Split failure for W Park Row Dr.

The Left Turn Gap Analysis measure evaluates the safety and efficiency of left turn movements at intersections without requiring any special configuration (David Bremer 2019). This analysis focuses on assessing the availability and adequacy of gaps in oncoming traffic that allow vehicles to safely make left turns, considering factors such as traffic volume, speed, and driver behavior. Fig. 7-16 shows the average gap between oncoming vehicles typically ranges from 40 to 60 seconds at Mitchell Street. This duration of gaps is crucial because it provides sufficient time for vehicles to safely complete left turns without causing delays or posing safety risks. The Left Turn Gap Analysis figure visually presents detailed information about the gaps available for left-turning vehicles at intersections, showing time intervals when left-turn movements occur on the x-axis and different signal phases on the y-axis.

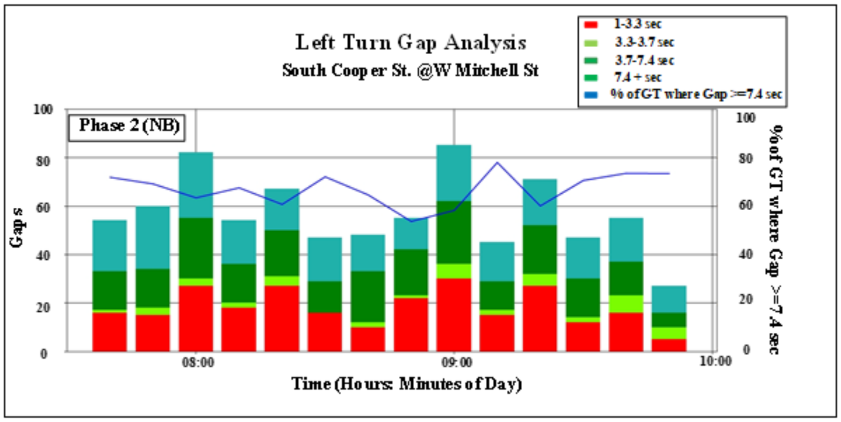


Figure 7-16: Purdue Phase Termination for W Mitchell St.

The Wait Time measure provides a detailed graphical representation of how long vehicles wait between green indications at the intersection (David Bremer 2019). It calculates the wait time as the duration (in seconds) between the first detector call on a phase during red and when the indication for that phase turns green. Additionally, data on the termination type of the previous phase and the volume per hour for the phase are included for reference. Fig. 7-17 shows the Wait Time for Park Row Dr, where the average wait time is 72.4 seconds and the maximum wait time is 85.7 seconds. These values signify the typical and worst-case scenarios for vehicle wait times at this intersection. A higher average and maximum wait time suggest potential periods of congestion and delays, which can impact traffic flow efficiency and driver satisfaction. The graph uses time intervals on the x-axis to show when vehicles arrive at the intersection and displays wait times in seconds on the y-axis. This visual representation allows the opportunity to assess the effectiveness of signal timings, identify congestion patterns, and make informed decisions to optimize traffic flow.

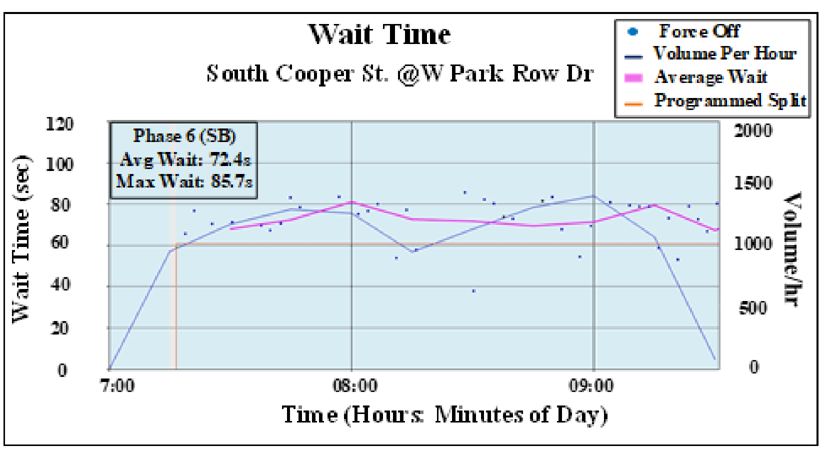


Figure 7-17: Wait time for W Park Row Dr.

7.2.3. Analysis of Performance Metrics from UTAIM

In TM-7, a similar approach was taken to compare outputs from UDOT-ATSPM and UTAIM systems for traffic signal performance evaluation. Further analysis of Fig. 7-18 reveals that from approximately 7:30 to 8:15 am, the cycle length remains constant at an average of 144 seconds, validating that pedestrian calls do not disturb the cycle length. However, a preemption call (causes a sudden increase in the cycle length to up to 250 seconds. This preemption occurs when emergency vehicles need priority access, affecting the regular cycle sequence. Additionally, the Prioritor extends the green phase, accommodating the transit vehicle's passage through the intersection. This extension of the green phase can be seen in the figure. Likewise, it also results in a disturbance of the cycle length afterward, as the signal timing adjusts to accommodate the prioritized vehicle. These actions illustrate how both preemption and TSP affect the cycle length, demonstrating their impact on intersection operations and traffic flow.

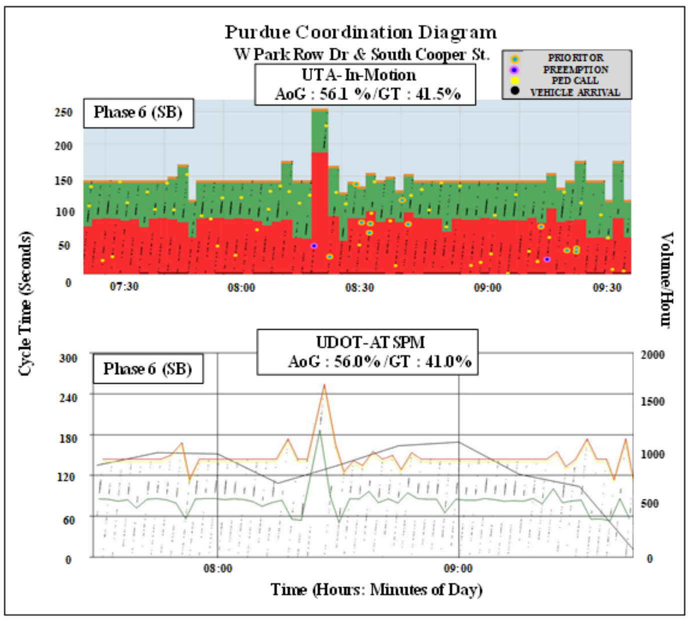


Figure 7-18: Purdue Coordination Diagram for Phase 6 (SB) W Park Row Dr & South Cooper St.

Fig. 7-19 illustrates the PCD for Pioneer Pkwy. The cycle length remains constant throughout the intersection at 144 seconds, except for certain fluctuations due to preemption and TSP (prioritor). As mentioned earlier, the limitations of UDOT-ATSPM in showing these fluctuations in cycle length are further clarified by UTAIM, which provides an actual representation of the calls made at the intersection, whether they are preemptions or TSP. The PCD in Fig. 7-19 demonstrates how preemption can terminate an extended green light in favor of providing green light time for the vehicle requesting preemption. On the other hand, Prioritor when a vehicle arrives on red, must join the queue until the next cycle of green is started. In this case, the prioritor only extends the available green time to allow the vehicle to pass through the intersection. This analysis signifies the operational impacts of preemption and TSP on intersection control. Preemption, used by emergency vehicles and rails, causes interruptions to regular traffic flow by extending green phases. Meanwhile, TSP optimizes traffic flow for priority vehicles like freight, ensuring minimal disruption to traffic while allowing these vehicles to move efficiently through the intersection. The comparison between UDOT-ATSPM and UTAIM PCDs highlights the importance of accurately representing these signal control events for effective traffic management and intersection performance optimization.

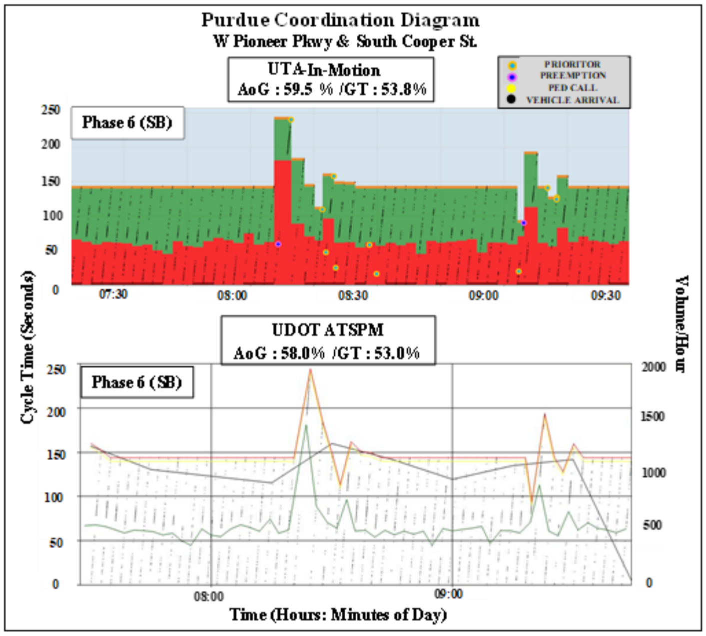


Figure 7-19: Purdue Coordination Diagram for Phase 6 (SB) W Pioneer Pkwy & South Cooper St.

Fig. 7-20 illustrates a coordinated TSD for Division Street to Arkansas Ln intersections. In Fig. 7-20A, the diagram shows a preemption call made by the rail. Yellow dots in Figure represent pedestrian calls at Intersections 4 and 7, where pedestrian crossings are located. Fig. 7-20B displays preemption caused by emergency vehicles, represented by dark blue trajectory lines. These vehicles receive green signals at every intersection, illustrating their ability to terminate other cycles to clear their path. Fig. 7-20C represents TSP, indicated by vehicles mostly receiving green signals but needing to join the queue upon arrival at red lights. This highlights the difference between preemption and TSP. Upon closer examination of the northbound approach at Intersection 11 (Arkansas Ln shown as Region D in Fig. 7-20, it's apparent that some vehicles are moving slowly, leading to delays and queues. Although the queue is not prominently visible in the diagram, it's evident that certain vehicles are stopping multiple times before crossing the intersection. This situation indicates split failures, where vehicles are unable to cross in the next cycle and must wait through multiple cycles to pass the intersection. While notable congestion isn't observed due to the coordination of the entire corridor, there is an expectation of increased queues in scenarios where vehicle demand rises without corresponding adjustments to the signal timing plan. This could be exacerbated by preemption and TSP fulfilling their demand and adding more queues in the coordinated Cooper Street Corridor.

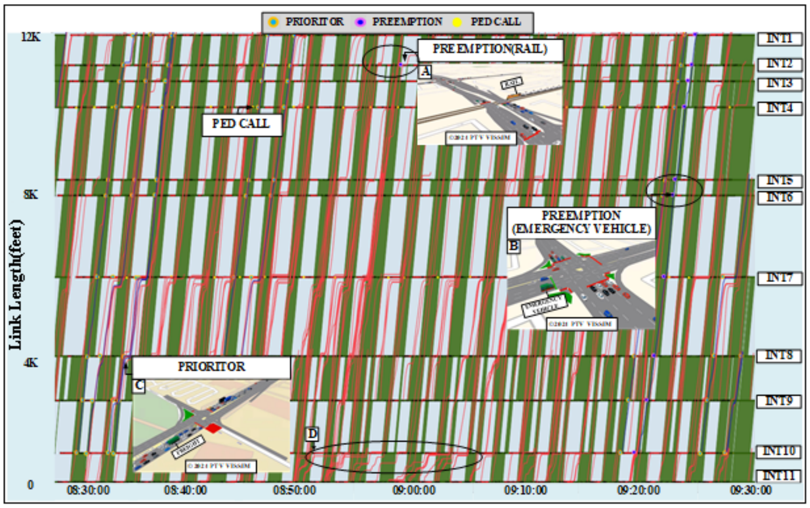


Figure 7-20: Time Space Diagram for Northbound Cooper St.

7.3. ANALYZING TRAFFIC SIGNAL PERFORMANCE WITH PROJECTED TRAVEL DEMAND GROWTH USING ATSPM-IN-THE-LOOP SIMULATION

The main goal is to demonstrate the potential of the ATSPM-In-The-Loop simulation framework in designing and evaluating complex traffic signal operations. The objective is to manage increasing traffic volumes effectively while ensuring the traffic signal system functions optimally. To achieve this, traffic volumes on all approaches were simulated to reflect the projected increase in travel demand over the next five years. Using a growth factor of $(1 + 3\%)^5 = 1.15$ which represents the anticipated new travel demand in five years, the traffic volumes were adjusted accordingly. Additionally, the performance of the traffic signal system was evaluated using the ATSPM-In-The-Loop traffic simulation model in TM-7. This evaluation is crucial to understand the impact of anticipated traffic demand and identify any necessary adjustments or optimizations needed for traffic signal timings. By conducting this evaluation, traffic engineers and planners can gain valuable insight into how the traffic signal system will perform under future conditions, allowing them to make informed decisions and implement measures to ensure smooth traffic flow and efficient signal operations.

7.3.1. Analysis of Performance Metrics from VISSIM ("ground truth")

The rise in traffic volume has significantly impacted the performance of intersections, notably increasing both average queue lengths and vehicle delays across most routes. Specifically, at the intersections of Main Street and UTA Blvd, the LOS has declined from B to C, indicating a transition from a stable flow with acceptable delays to a more congested flow with noticeable delays. The data from Table 7-3 shows that average queue lengths and vehicle delays have increased by notable percentages, reflecting the overall trend of congestion. These increases are further exacerbated by preemption and TSP, which, while designed to prioritize certain vehicles such as emergency and transit vehicles, disrupt normal signal operations and contribute to longer waits and delays for general traffic.

Table 7-3: Results obtained from PTV VISSIM.

Intersection	Main St		UTA Blvd		Park Row Dr		Pioneer Pkwy	
Movement	NBT	SBT	NBT	SBT	NBT	SBT	NBT	SBT
Queue Length	17.39 **(96%)**	7.83 **(34%)**	24.34 **(37%)**	29.53 **(43%)**	180.02 **(64%)**	77.11 **(11%)**	45.76 **(46%)**	68.22 **(41%)**
Average Queue length	6.56 **(92%)**	3.66 **(27%)**	9.37 **(39%)**	22.43 **(46%)**	127.91 **(61%)**	37.77 **(5%)**	60.35 **(36%)**	56.28 **(25%)**
Vehicle Delay	16.17 **(14%)**	10.35 **(11%)**	15.00 **(19%)**	17.12 **(22%)**	38.85 **(18%)**	41.45 **(-4%)**	31.25 **(53%)**	32.27 **(10%)**
Average Vehicle Delay	20.67 **(45%)**	15.06 **(9%)**	13.11 **(16%)**	24.69 **(-27%)**	43.38 **(10%)**	40.53 **(-6%)**	44.14 **(37%)**	36.23 **(6%)**
LOS	C		C		D		C	

Furthermore, a graph illustrating the average travel time of vehicles was plotted before and after increasing the volume based on the VISSIM results. In Fig. 7-21, it is clear that the average travel time has significantly increased. A notable observation is the sharp rise in average travel time after 8:00 am, where the travel time from one end of the corridor to the other saw a drastic increase of up to 60%. This substantial increase indicates that despite the Cooper Street Corridor being in a coordinated phase, the higher vehicle volume is not aligned with the previous signal timing. Consequently, this discrepancy leads to vehicle delays, congestion, and longer travel times. Moreover, it's important to note that the travel time before and after peaked after 8:50 am, correlating with the increased volume. The impact of complex traffic signals such as preemption and TSP becomes more prominent as the day progresses, with time differences of up to 20% observed. So as per TM-6, a detailed node analysis based on the VISSIM database was conducted to further confirm the deterioration of traffic conditions. The findings of this analysis are detailed in the subsequent section.

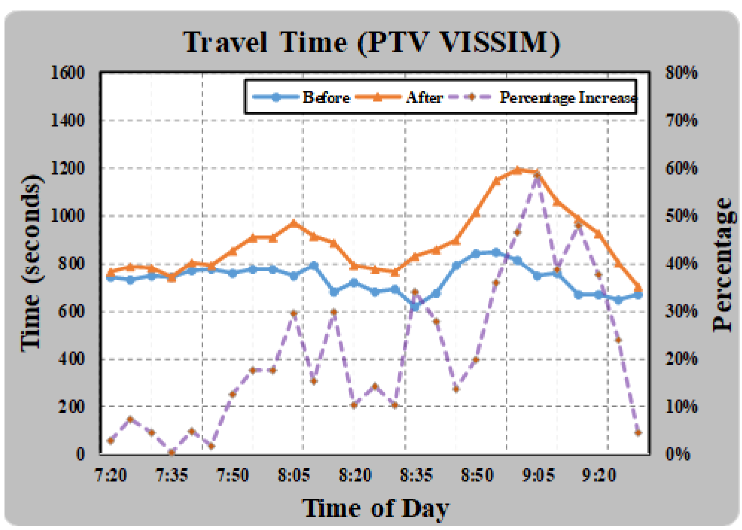


Figure 7-21: Comparison of the Travel Time with Increased Volume.

Using the node analysis data from the VISSIM simulation, the impact of preemption and TSP on vehicle average delay and queue lengths was assessed. The study revealed that preemption and TSP caused vehicle delays to increase across nearly every approach of the intersection, with Pioneer Pkwy experiencing over a 35% rise in delays. Queue lengths also surged by more than 60% for northbound Pioneer Pkwy. Notably, the northbound direction experienced a more significant impact than the southbound direction which further aligns with the fact relating to CBD as mentioned in TM-6. These increases are attributed to preemption and TSP, which prioritize specific vehicles and disrupt normal signal timings, leading to higher congestion and longer waits. This necessitates signal timing adjustments, which may not be sufficient to handle the increased traffic volume, further exacerbating delays and queue lengths. The findings highlight that preemption and TSP have significantly impacted the Cooper Street Corridor, underscoring the need for optimized signal timing to improve traffic flow.

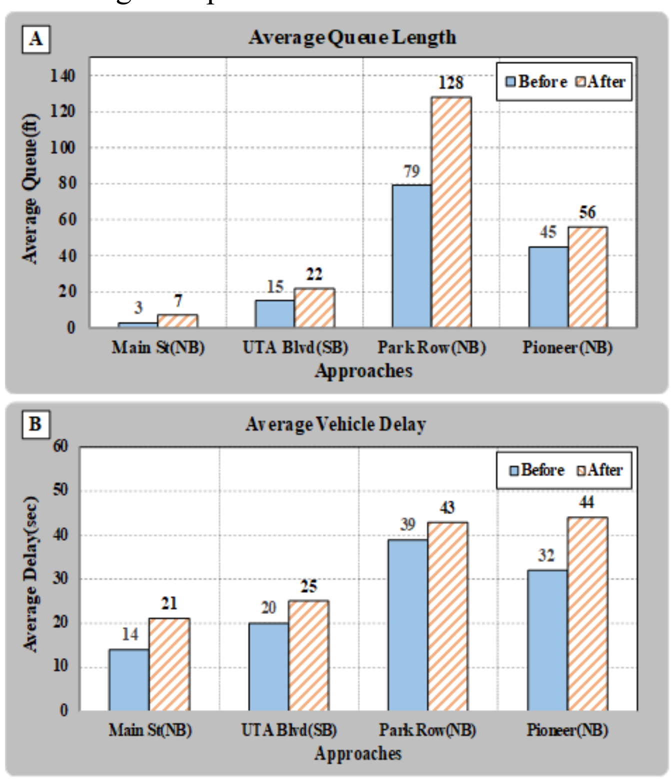


Figure 7-22: Comparative study of Average Queue Length (A)and Vehicle Delay(B).

Table 4 reveals a decline in the LOS following an increase in volume for five intersections: Abram St, UTA Blvd, Pioneer Pkwy, California Ln, Mayfield Rd, and Interstate 20, out of a total of 16 intersections. The implementation of TSP and preemption has significantly affected the Cooper Street Corridor, particularly at these key intersections has disrupted the normal signal cycle to prioritize specific vehicles. This disruption cascades through the interconnected intersections, causing increased delays and longer queues for regular traffic. The findings underscore that while the LOS ratings remain within acceptable limits, the increased congestion and delays due to TSP and preemption call for optimized signal timing and traffic management strategies to mitigate their impact and maintain efficient traffic flow throughout the corridor.

Table 7-4: LOS obtained from PTV VISSIM.

Intersection	Baseline Scenario	Increased Volume Scenario
Division St	C	C
Main St	C	C
Abram St	B	C
UTA Blvd	B	C
Nedderman Dr	B	B
Mitchell St	C	C
Park Row Dr	C	C
Inwood Dr	C	C
Snooty Fox Dr	C	C
Pioneer Pkwy	C	D
Arkansas Ln	C	C
California Ln	B	C
Mayfield Rd	C	D
Arbrook Blvd	C	C
Pleasant Ridge Rd	C	C
Interstate 20	B	C

7.3.2. Analysis of Performance Metrics from UDOT-ATSPM

All comparable graphs were generated to analyze traffic signal performance with increased volume, highlighting the effects of preemption and TSP. While approach delays decreased, total delays increased, indicating disruption to overall traffic flow. Red signal time increased and green signal time decreased, favoring specific vehicles at the expense of general traffic flow. The platoon ratio increased, suggesting tighter vehicle groups. For instance, at Park Row Dr, with a Purdue Split of 29%, the signal allocates 29% of the cycle to vehicles, potentially disrupting optimal green time distribution for vehicles and pedestrians. This disruption underscores the importance of managing signal timing adjustments to balance the needs of all road users and optimize traffic flow efficiency, particularly in intersections with pedestrian volume. Additionally, increases in Purdue Split Average and maximum waiting times indicate coordination disruptions influenced by preemption and TSP, which may not be optimal for existing coordinated cycle lengths.

Table 7-5: Performance Metrics obtained from UDOT-ATSPM.

Intersection	Main St		UTA Blvd		Park Row Dr		Pioneer Pkwy	
Performance Metrics	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)	Phase 2 (NB)	Phase 6 (SB)

Approach Delay(sec)	2 **(0%)**	2 **(0%)**	4 **(0%)**	8 **(33%)**	7 **(-13%)**	16 **(-20%)**	8 **(14%)**	14 **(-13%)**
Total Delay (hr.)	2.4 **(26%)**	1.6 **(60%)**	4.7 **(38%)**	5.7 **(68%)**	10.5 **(13%)**	16 **(40%)**	7.8 **(50%)**	10.8 **(4%)**
Approach Volume	3825 **(25%)**	2865 **(28%)**	3865 **(16%)**	2693 **(19%)**	3636 **(22%)**	5466 **(24%)**	3524 **(56%)**	2970 **(26%)**
Arrivals on red (AoR-%)	20 **(-9%)**	14 **(23%)**	22 **(10%)**	26 **(4%)**	39 **(-10%)**	36 **(-14%)**	22 **(5%)**	37 **(-10%)**
Red Time (RT-%)	24 **(-4%)**	27 **(17%)**	41 **(5%)**	41 **(8%)**	59 **(0%)**	46 **(94%)**	46 **(5%)**	47 **(0%)**
Arrivals on green (AoG-%)	80 **(3%)**	86 **(-3%)**	78 **(-3%)**	74 **(-1%)**	82 **(3%)**	61 **(11%)**	82 **(-1%)**	63 **(7%)**
Green Time (GT- %)	76 **(1%)**	73 **(-5%)**	59 **(-3%)**	59 **(-5%)**	48 **(-2%)**	41 **(-2%)**	48 **(-4%)**	43 **(0%)**
Platoon Ratio (PR)	1.05 **(1%)**	1.18 **(2%)**	1.32 **(1%)**	1.25 **(3%)**	1.71 **(4%)**	1.49 **(13%)**	1.44 **(3%)**	1.19 **(7%)**
Average Split	94.80 **(1%)**	85.80 **(-12%)**	90.00 **(19%)**	87.70 **(28%)**	74.60 **(-14%)**	65.60 **(-24%)**	85.0 **(-32%)**	84.7 **(-32%)**
Purdue Split (PR)	4 **(33%)**	3 **(0%)**	5 **(40%)**	0 **(-10%)**	29 **(71%)**	0 **(0%)**	30 **(20%)**	10 **(40%)**
Average Wait (sec)	109.10 **(-33%)**	12.70 **(9%)**	32.80 **(24%)**	40.90 **(4%)**	62.00 **(-3%)**	73.80 **(2%)**	59.10 **(17%)**	57.10 **(5%)**
Max Wait (sec)	271.50 **(0%)**	25.90 **(0%)**	93.00 **(36%)**	68.30 **(6%)**	73.20 **(-5%)**	94.70 **(11%)**	76.50 **(-3%)**	70.20 **(9%)**

Likewise, approach delay graphs for Park Row Dr were generated. The comparison between the before and after scenario graphs is presented in Fig. 7-23. It is evident from the figure that the AD per vehicle has decreased from 8 seconds to 7 seconds, while the TD has increased from 9.3 seconds to 9.7 seconds, as shown in Fig. 7-23. This increase in delay, particularly evident at 8:45 am, may be attributed to the effects of TSP and preemption, which prioritize specific vehicles and disrupt normal traffic flow. These additional graphs aid in validating our conclusion regarding the worsening conditions at this intersection due to the expected increase in vehicle demand and the implementation of TSP and preemption.

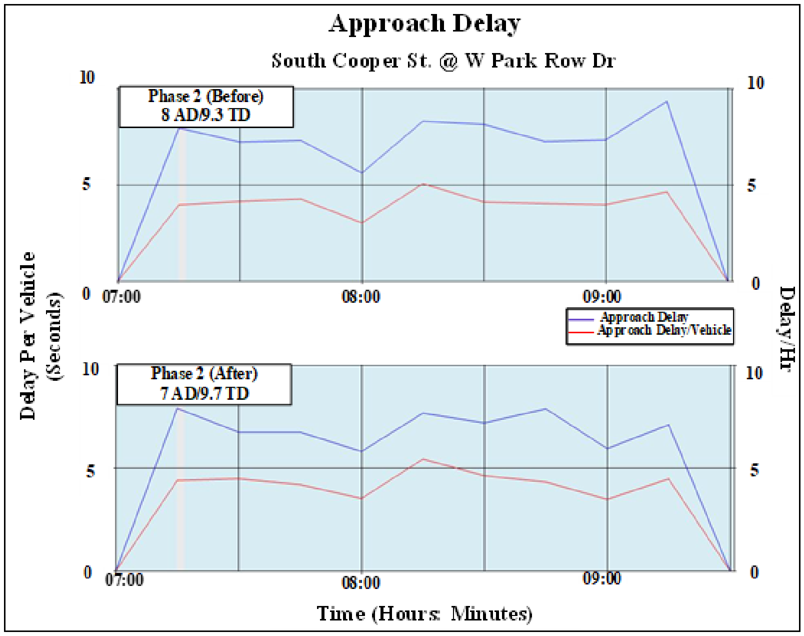


Figure 7-23: Approach Delay at W Park Row Dr.

Following the increase in volume, the AoR percentage for Phase 6 (southbound) notably decreased from 44% to 39% at the Park Row intersection, as illustrated in Fig. 7-24. Meanwhile, the RT percentage remained constant at 59%. These changes indicate a shift in traffic patterns, potentially influenced by the implementation of TSP and preemption. Without adjustments to signal timing in response to the volume increase, these fluctuations in arrival rates across intersections suggest challenges in efficiently accommodating all vehicles, potentially leading to instances of stop-and-go traffic along the corridor.

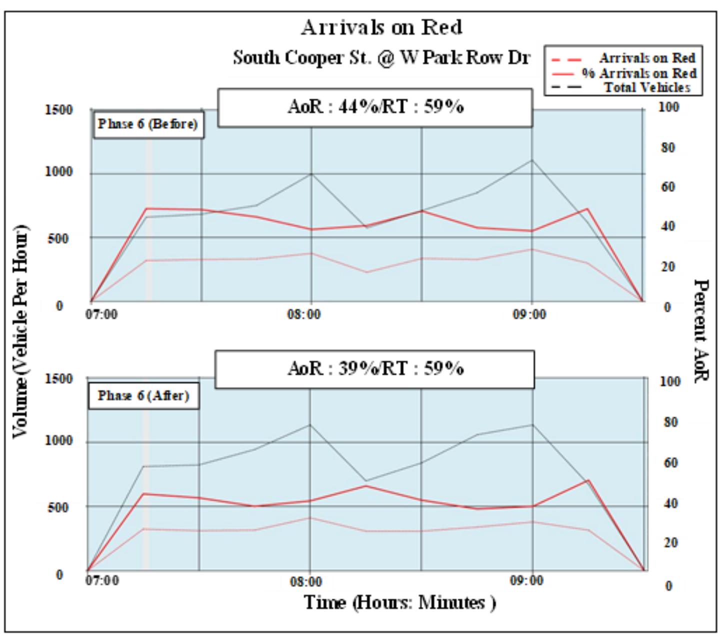


Figure 7-24: Arrivals on Red at W Park Row Dr.

In Fig. 7-25, The PCD reveals that for Phase 2 (northbound), the AoG has increased to 63%, indicating an improvement in the proportion of green time allocated to vehicles. However, despite traffic congestion, achieving optimal AoG remains challenging, highlighting how prolonged queue lengths can keep detectors occupied and contribute to elevated AoG values even during congested traffic conditions. This scenario exemplifies the impact of TSP and preemption, which can lead to less predictable traffic patterns. The data also shows a decrease in fluctuation after 9:00 AM in the after graph, with more variability around 8:30 AM, demonstrating the randomness introduced by TSP and preemption. These values should not be interpreted in isolation, but rather in conjunction with trajectory-based TSD, which provides a more accurate representation of queue lengths and traffic conditions.

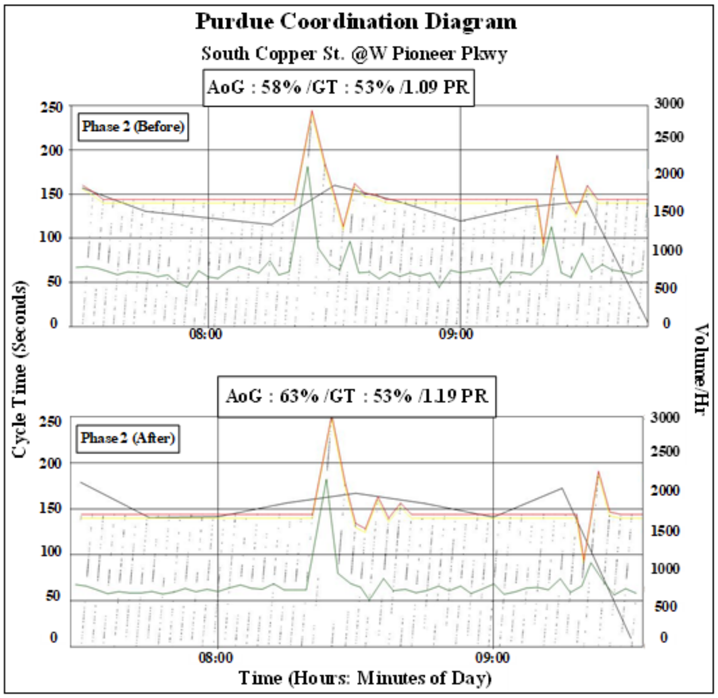


Figure 7-25: Purdue Coordination Diagram for W Pioneer Pkwy.

7.3.3. Analysis of Performance Metrics from UTAIM

In Table 7-6, most intersections showed a decrease in AoG, with only Intersection 11 (Arkansas Ln) showing a positive increase of 10%. This increase may be due to the detection bars being occupied by vehicles in the queue, which can lead to an overestimation of AoG as the traffic signal system interprets continuous vehicle presence during high traffic volumes. Similarly, GT also decreased in most cases. This decrease can be attributed to the prioritization of specific vehicles with preemption and TSP, leading to reduced GT allocation for general traffic. In the southbound case, most AoG and GT values decreased, with certain anomalies. Notably, there was a decrease of up to 13% in Park Row Dr and 21% in Pleasant Ridge, highlighting the interconnected nature of intersections and the significant impact seen at Intersection 15 (Pleasant Ridge) compared to Intersection 7 (Park Row Dr). For GT, Main Street, where rail is set up, showed the most significant decrease, up to 34%, reflecting the adjustments made for rail preemption.

Table 7-6: Performance metrics obtained from UTAIM.

	Northbound		Southbound	
Intersection	**AoG %**	**GT %**	**AoG %**	**GT %**
Division St	68.80**(-1%)**	55.50**(-5%)**	68.10**(0%)**	52.70**(-6%)**
Main St	81.60**(1%)**	76.70**(-1%)**	82.50**(2%)**	76.30**(-34%)**
Abram St	90.10**(-2%)**	69.30**(-2%)**	88.20**(3%)**	67.60**(-3%)**
UTA Blvd	79.80**(-3%)**	61.60**(-4%)**	77.40**(5%)**	58.90**(-4%)**
Nedderman Dr	89.90**(-2%)**	73.50**(0%)**	88.40**(-5%)**	73.20**(8%)**
Mitchell St	94.30**(-2%)**	74.50**(-3%)**	92.00**(0%)**	72.50**(-1%)**
Park Row Dr	80.80**(3%)**	48.10**(-1%)**	83.40**(-13%)**	47.50**(-10%)**
Inwood Dr	83.10**(-2%)**	84.20**(-3%)**	81.20**(2%)**	81.70**(0%)**
Snooty Fox Dr	80.30**(-1%)**	83.00**(-1%)**	79.50**(-1%)**	82.20**(-7%)**
Pioneer Pkwy	79.30**(0%)**	56.00**(-3%)**	79.20**(-1%)**	54.30**(7%)**
Arkansas Ln	61.60**(10%)**	62.30**(-12%)**	67.80**(-5%)**	55.00**(-2%)**
California Ln	85.20**(-4%)**	86.20**(-2%)**	82.00**(-8%)**	84.50**(0%)**
Mayfield Rd	84.10**(-3%)**	61.00**(-6%)**	81.40**(0%)**	57.10**(6%)**
Arbrook Blvd	81.90**(-4%)**	56.30**(-4%)**	78.90**(3%)**	54.00**(4%)**
Pleasant Ridge Rd	80.10**(2%)**	72.80**(-3%)**	82.10**(-21%)**	70.70**(-3%)**
Interstate 20	86.60**(1%)**	83.30**(1%)**	87.20**(-1%)**	84.00**(1%)**

The PCD for northbound at Park Row Dr was analyzed in Fig. 7-26, showing an increase in AoG from 80.8% to 83.4%, and a decrease in GT from 48.1 seconds to 47.5 seconds with an increase in volume. However, the cycle length remained nearly constant at approximately 144 seconds, with sudden spikes due to preemption and certain extensions of green time due to TSP. The analysis also showed no significant effect due to pedestrian calls on cycle length. This scenario signifies that despite the increase in traffic volume, the traffic signal system effectively adjusted the green time allocation and cycle length to accommodate the traffic demand, with occasional adjustments for preemption and TSP. The lack of significant effects due to pedestrian calls on cycle length suggests that the signal timing adjustments prioritize vehicle flow efficiency, ensuring minimal disruption from pedestrian crossings while maintaining traffic flow.

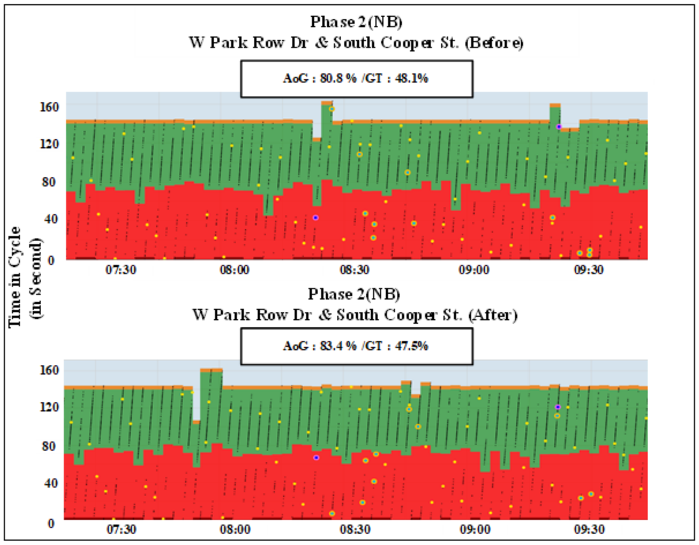


Figure 7-26: Purdue Coordination Diagram for Phase 2 (NB) W Park Row Dr. & South Cooper St.

The TSD for Cooper Street in the northbound direction, depicted in Fig. 7-27, provides a comprehensive view of traffic dynamics influenced by preemption and TSP. The diagram highlights significant operational differences between preemption and TSP. Additionally, the TSD illustrates instances where vehicles must stop for pedestrian crossings, affecting traffic flow. With a 15% increase in volume, the diagram shows that a notable number of vehicles are required to make at least three stops before passing through certain sections, particularly in region D. This observation indicates that higher traffic volumes lead to more frequent stops and potentially increased travel times. The TSD is validated against a VISSIM snapshot, ensuring accuracy in depicting real-world traffic conditions. Overall, the diagram underscores the impact of preemption, TSP, and pedestrian calls on traffic operations, highlighting the challenges of managing increased traffic volumes while maintaining efficient flow through signal timing adjustments.

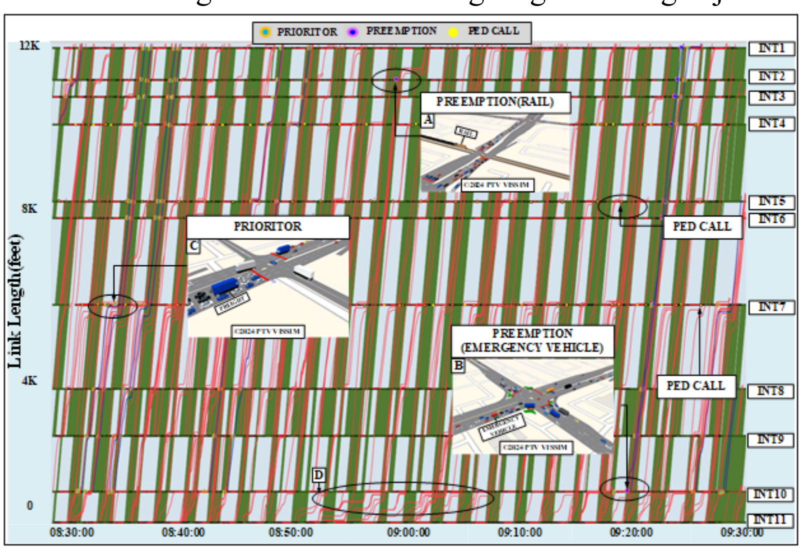


Figure 7-27: Time Space Diagram for Northbound Cooper St.

7.4. DISCUSSION

- This chapter demonstrates the potential of the ATSPM-In-The-Loop simulation framework for complex traffic signal design to the TAC and delivers a detailed guideline for setting up the SILS software tool.
- Building on the findings from TM-6, where it was revealed that UDOT-ATSPM tends to overestimate the AoG and GT for long queues when advance detectors are occupied, TM-7 further justifies this by conducting additional simulations. These simulations confirmed that vehicles arriving during the red signal phase are not considered until the queue is cleared, reinforcing the need for accurate performance measures.
- The analysis in TM-6 showed that increased traffic volume results in longer queue lengths, higher control delays, and extended travel times compared to original volumes under the same signal timing. TM-7 further validates these findings with additional data, demonstrating the significant impact of traffic volume on signal performance and travel efficiency.
- This chapter explains how preemptions override existing signal priorities, terminating extended green lights to provide immediate green lights for vehicles requesting preemption. TSP provides extra green time at intersections for certain vehicles (e.g., freight) to improve transit efficiency and reliability.
- Furthermore, this chapter also demonstrates how using UDOT-ATSPM and UTAIM, coupled with simulation, can help understand the impact of incorporating different transportation modes. For instance, it could potentially explore whether changing rail schedules can improve overall traffic flow during peak hours, or if increasing or decreasing the volume of freight vehicles will significantly affect traffic flow in a specific corridor. This case study can be adapted to any scenario with sufficient data. Additionally, it aids city planners in understanding signal performance in future scenarios. Overall optimizing a pre-established network system is a complex task, which may require multiple iterations.

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APPENDIX A: THE QUESTIONNAIRE OF THE SURVEY (CONVERTED FROM ITS ONLINE VERSION)

Hello: You are invited to participate in our survey to understand the traffic signal stakeholders' experiences and/or expectations on Automated Traffic Signal System Metrics (ATSPM). In this survey, you will be asked to complete a survey that asks questions about your opinions on ATSPM. It will take approximately 10 minutes to complete the questionnaire. Your participation in this study is completely voluntary. There are no foreseeable risks associated with this project.

However, if you feel uncomfortable answering any questions, you can withdraw from the survey at any point. We need to learn your opinions. Your survey responses will be strictly confidential and data from this research will be reported only in the aggregate. Your information will be coded and will remain confidential. If you have questions at any time about the survey or the procedures, you may contact Slade Wang by email at "peirong.wang@mavs.uta.edu" This survey is part of a research project sponsored by the TxDOT Research and Technology Innovation (RTI) division, titled "Improving traffic signal system planning, design, and management with big-data-enhanced Automated Traffic Signal performance metrics (ATSPM) system “Thank you very much for your time and support. Please begin the survey by clicking on the START button below.

Q1: What’s your current employer?
1. Federal agencies
2. State DOTs
3. Municipal Traffic agencies
4. Public Transit officials
5. Consulting Firms
6. Academia /Research institutes
7. Other __________

Q2: Did you deploy, use, or consider the ATSPM system in traffic management?
1. Yes
2. No

Q3: Have you ever used an ATSPM system before?
1. Yes
2. No

Q4: Which ATSPM system are you using or considering?
1. UDOT-ATSPM
2. INRIX
3. Variants of UDOT-ATSPM
4. Q-free Maxview’s ATSPM module
5. Iteris Cloud-based ATSPM
6. Other __________

Q5: Coverage of your ATSPM system (deployed or considered)?
1. Small (1 - 10 intersections)
2. Medium (11 - 30 Intersections)
3. Large (Above 30 intersections) __________

Q6: Which best describes your current exercise with the ATSPM system? (Please select one)
1. Not installed but plan to do so soon.

2. Just trying out
3. Supplemental to ATMS or ATIS (Occasional usage)
4. Active Usage (regularly)

Q7: On a scale of 1-5 (1. Not useful. 2. Kind of Useful; 3 It's OK; 4 Very useful; 5. Must have), what is the importance of the following commonly used ATSPM MOEs?

	1	2	3	4	5	N/A
PCD	❑	❑	❑	❑	❑	❑
Spilt Monitor	❑	❑	❑	❑	❑	❑
Ped Delay	❑	❑	❑	❑	❑	❑
Preemption Details	❑	❑	❑	❑	❑	❑
Timing and Actuation	❑	❑	❑	❑	❑	❑
Purdue Split Failure	❑	❑	❑	❑	❑	❑
Yellow and Red Actuations	❑	❑	❑	❑	❑	❑
Turning Movement Counts	❑	❑	❑	❑	❑	❑
Approach Volume	❑	❑	❑	❑	❑	❑
Approach Delay	❑	❑	❑	❑	❑	❑
Arrive on Red	❑	❑	❑	❑	❑	❑
Approach Speed	❑	❑	❑	❑	❑	❑
Left-Turn Gap	❑	❑	❑	❑	❑	❑
Wait Time	❑	❑	❑	❑	❑	❑
Time-of-Day Volume / Capacity Plot	❑	❑	❑	❑	❑	❑
Time-space-diagram	❑	❑	❑	❑	❑	❑
Multimodal	❑	❑	❑	❑	❑	❑

Q8: Please evaluate the ATSPM's benefits that you perceive.

	Strongly Disagree	Disagree	Neutral	Agree	Strongly Agree
Reduce delay and emission	❑	❑	❑	❑	❑
Signal Health monitoring	❑	❑	❑	❑	❑
Causes for congestion	❑	❑	❑	❑	❑
Safety Analysis	❑	❑	❑	❑	❑

Q9: Have you deployed ATSPM, or do you consider deploying ATSPM at a large scale (>30 intersections)?

1. Yes
2. No

Q10: What inspires you to consider upscaling your ATSPM system? (Please rank)

	Strongly Disagree	Disagree	Neutral	Agree	Strongly Agree
Comprehensive MOEs	❑	❑	❑	❑	❑
Easy and straightforward	❑	❑	❑	❑	❑
Available budget and affordable	❑	❑	❑	❑	❑
Required by administration	❑	❑	❑	❑	❑

Q11: What concerns you about the large-scale deployment of the ATSPM system? (Please rank)

	Strongly Disagree	Disagree	Neutral	Agree	Strongly Agree
Futureproof? (e.g., frequent major upgrades)	❑	❑	❑	❑	❑
Rather different from current practice	❑	❑	❑	❑	❑
Need a lot of training	❑	❑	❑	❑	❑
Hardware/Software Cost (Detection, Solution, database, etc.)	❑	❑	❑	❑	❑
Overlapped functions with other ITS/big-data solutions	❑	❑	❑	❑	❑

Q12: What are your wishes and expectations for the ATSPM system in the future?

	Strongly Disagree	Disagree	Neutral	Agree	Strongly Agree
Introduce emerging big data, such as connected vehicles, crowdsource traffic data, and create innovative MOEs	❑	❑	❑	❑	❑
Introduce more safety MOEs	❑	❑	❑	❑	❑

Consider multimodal traffic	❑	❑	❑	❑	❑
Integrate inclusive sensing data such as pedestrian/bicyclist data	❑	❑	❑	❑	❑
AI-empowered automated problem identification and decision support for traffic signal management	❑	❑	❑	❑	❑
Consider corridor-level traffic signal performance metrics	❑	❑	❑	❑	❑

Q13: Based on your understanding, please rank your opinions about ATSPM and its features.

	Strongly Disagree	Disagree	Neutral	Agree	Strongly Agree
Multimodal metrics	❑	❑	❑	❑	❑
Automation (visualization and decision support)	❑	❑	❑	❑	❑
Future Prediction	❑	❑	❑	❑	❑
Easy and straightforward	❑	❑	❑	❑	❑
Robust and resilient	❑	❑	❑	❑	❑

Q14: What are your concerns about the ATSPM system? (Please rank)

	Strongly Disagree	Disagree	Neutral	Agree	Strongly Agree
Cost	❑	❑	❑	❑	❑
Training efforts	❑	❑	❑	❑	❑
long-term Maintain	❑	❑	❑	❑	❑
Reliability	❑	❑	❑	❑	❑

Q15: What's the current traffic signal management platform that you are working with?

1. ATMS
2. Open-source software (e.g., UDOT-ATSPM)
3. Research-spinoff systems by academia.
4. Commercial ATSPM platform
5. Other ___________

Q16: Rank the importance of the following performance metrics in traffic signal management? (1: Least important, 5: Most important)

	1	2	3	4	5
PCD	❑	❑	❑	❑	❑
Spilt Monitor	❑	❑	❑	❑	❑
Ped Delay	❑	❑	❑	❑	❑
Preemption Details	❑	❑	❑	❑	❑
Timing and Actuation	❑	❑	❑	❑	❑
Purdue Split Failure	❑	❑	❑	❑	❑
Yellow and Red Actuations	❑	❑	❑	❑	❑
Turning Movement Counts	❑	❑	❑	❑	❑
Approach Volume	❑	❑	❑	❑	❑
Approach Delay	❑	❑	❑	❑	❑
Arrive on Red	❑	❑	❑	❑	❑
Approach Speed	❑	❑	❑	❑	❑
Left-Turn Gap	❑	❑	❑	❑	❑
Wait Time	❑	❑	❑	❑	❑
Time of Day Volume / Capacity Plot	❑	❑	❑	❑	❑
Multimodal PCD	❑	❑	❑	❑	❑
Corridor PCD	❑	❑	❑	❑	❑

APPENDIX B: VALUE OF RESEARCH

The UTA team conducted a value of research (VOR) analysis of TxDOT Research Project 0-7160 to produce an estimate of the benefit that the project will likely yield for TxDOT. The temporal scope for this analysis is an 11-year period (labeled as years 1–11), starting with the beginning of the 2-year project. The value of the project is described in terms of NPV and cost-benefit ratio (CBR), which are computed using economic discounting formulas.

The primary objective of TxDOT Research Project 0-7160 is to promote a new approach to traffic signal design and eventually improve mobility on urban street. congestion of urban and rural freeways in Texas.

METHODOLOGY

The UTA team used a VOR template provided by TxDOT to compute the NPV and CBR measures. The template requires the following items:

- Project budget: $292,033.35.
- Project duration: 2.00 years.
- Expected value duration: 11 years (convention chosen by TxDOT).
- Discount rate: 3 percent (default value assumed by TxDOT).
- Expected value per year: $2,116,000.

The project's expected value per year is estimated based on increased ATSPM deployment and users because of the project deliverables and the ATSPM benefits including operations and mobility benefits.

Concept

To conduct the VOR analysis, the following steps were taken:

1. Estimate the average number of traffic signal projects sponsored by TxDOT every year
2. Estimate the benefits of ATSPM because of this tool
3. Provide approximation of mobility benefits.
4. Provide approximation of Cost-benefit ratios.
5. Apply the procedure to estimate the expected value of the research.

Input Data

The UTA team first identified the TxDOT's mobility projects for the next 10 years using the TxDOT Project Tracker, the total number of planned projects is 1,152 and total budget is $113,596,630. It is assumed that 10% of the mobility projects will be related to traffic signal and ATSPM, which is approximately 115 traffic signal projects in 10 years. It is further assumed that 20% of traffic signal projects will deploy the ATSPM systems (23 projects, 2.3 projects per year) and each project includes 10 intersections on average. Therefore, there are about 23 traffic signal projects related to this tool per year.

Installation cost

The initial installation charge by consultants for an ATSPM system right now is about $100K per solution, including software deployment and a certain period of maintenance. The ATSPM system at each intersection will cost $5,000 per year to maintain.

Project benefit

Using the delivered ATSPM-in-the-loop simulation solution will facilitate stakeholders to understand and use the ATSPM systems. As a result, it will reduce the installation cost which is mostly composed of labor hours. The estimated benefit (in terms of cost reduction) will be 50%. The maintenance cost will not reduce a lot as it involves hardware and software. The user delay benefit is estimate as $87K per signal per year2 (page 58). Therefore, the total benefits of deployment and mobility every year will be:

- $50K*2.3 (installation cost reduction) and.
- $87K*23 (delay reduction)

RESULTS

The UTA team calculated the NPV and CBR according to the above rationale and conclude Figure xxx

Figure B-1 summarizes the VOR calculations. The payback period for Research Project 0-7131 is 0.11 years, and the CBR is 64.

The findings shown in Figure xxx are as follows:

- The benefits included in the VOR calculations include only those incurred by TxDOT. Other agencies (e.g., local and county agencies within Texas and other state DOTs) will be able to implement and benefit from the published tools from the project.

- The estimated benefits include installation cost and mobility benefits, based on the assumption of tool and model usage. TxDOT will likely receive additional benefits that are more difficult to quantify.

- The VOR analysis focused on urban signalized intersections. Both rural and urban freeway facilities may also realize similar benefits from the application of these project results. The estimated VOR, NPV, and CBR would increase if these sites were included in the analysis.

Texas Department of Transportation

Project #	0-7160		
Project Name:	Improving traffic signal system planning, design, and management with big-data-enhanced Automated Traffic Signal Performance Monitoring (ATSPM) system		
Agency:	UTA	**Project Budget**	$ 239,186
Project Duration (Yrs)	2.0	**Exp. Value (per Yr)**	2116000
Expected Value Duration (Yrs)	10	**Discount Rate**	3%
Economic Value			
Total Savings:	$ 18,646,645	**Net Present Value (NPV):**	$ 15,250,596
Payback Period (Yrs):	0.113037	**Cost Benefit Ratio (CBR, $1 : $___):**	$ 64

Years	Expected Value
0	-$133,864
1	-$158,169
2	$2,116,000
3	$2,116,000
4	$2,116,000
5	$2,116,000
6	$2,116,000
7	$2,116,000
8	$2,116,000
9	$2,116,000
10	$2,116,000
11	$2,116,000

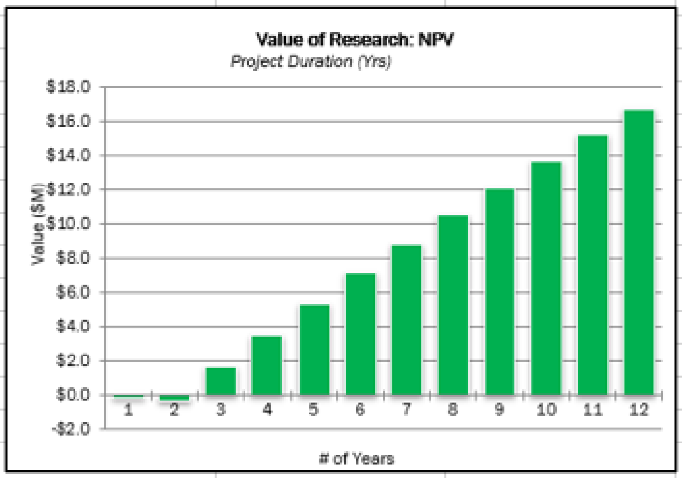

Figure B-1: Analysis Results