RESEARCH & DEVELOPMENT

Delay and User Cost Estimation for Work Zones on Urban Arterials

Bastian J. Schroeder, Ph.D., P.E.
Nagui M. Rouphail, Ph.D., Billy M. Williams, Ph.D., Ali Hajbabaie, Ph.D., Behzad Aghdashi, Ph.D., Sangkey Kim, Ph.D., Kambiz Tabrizi, and Brian Narron

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Delay and User Cost Estimation for Work Zones on Urban Arterials

Final Project Report

Prepared for:

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Prepared by:

B. Schroeder, N. Rouphail, B. Williams, A. Hajbabaie, B. Aghdashi,
S. Kim, K. Tabrizi, and B. Narron

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Abstract:
This is the final project report of NCDOT research project 2013-09: Delay and User Cost Estimation for Work Zones on Urban Arterials. The project seeked to develop a methodology for quantifying delay and user cost impacts of arterial work zones in North Carolina in an analytical framework, supported by NC-specific empirical performance data of arterial work zones. NCDOT recently acquired a similar methodology for the evaluation of significant work zones on freeways, and this research aims to build on that prior effort to develop a companion tool for arterial streets. Just as with the prior effort (NCDOT Research Project 2010-08), the methodology developed in this project would be implemented in a software tool, ARTVAL-WZ, which can be used directly for in-house analyses of these types of work zones to assure seamless technology transfer of these research products. This report summarizes the findings of all project tasks.

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Executive Summary

This documents is the final report for NCDOT research project 2013-09: Delay and User Cost Estimation for Work Zones on Urban Arterials. The research developed a methodology for quantifying delay and user cost impacts of arterial work zones in North Carolina in an analytical framework, supported by NC-specific empirical performance data of arterial work zones. The methodology developed in this project was implemented in a software tool, ARTVAL, which can be used directly for in-house analyses of these types of work zones to assure seamless technology transfer of these research products.

This research presented a statistical approach to model saturation headway at signalized intersection work zones based on work intensity, pavement condition, ledge presence, turn percentages from shared lanes, and the number of closed exclusive turn lanes. The saturation headway, work zone configuration, and prevailing conditions data were collected in North Carolina at six different intersections (representing seven intersection approaches), yielding a total of 19 unique work zone configurations and more than 4,600 headway observations.

The project team fitted four models, with three of the models applying traditional multiple linear regression at three different aggregation levels: a) cycle level aggregation, b) 15 minute aggregation, and c) full aggregation by work zone configuration. For the fourth model, the team used a path analysis based regression approach with a cycle level aggregation that allowed the inclusion of the 2010 HCM default values to adjust saturation headway based on right turn and left turn percentages.

All four models gave good fit and had statistically significant independent variables with logical signs. Overall, the project team recommends adopting the traditional regression model with 15-minute aggregation level, since it has a good statistical fit and is consistent with the HCM analysis period. However, if left turn and right turn percentages are significant and it is important to account for their impacts separately, the analyst may consider using the model based on path-analysis as it allows using the 2010 HCM recommended values to adjust for right and left turn percentages.

In addition to the headway model, the team developed default values needed to apply the analysis framework for urban streets analysis. The framework uses a quick estimation methods available in the Highway Capacity Manual: Quick Estimation Method for Urban Streets (QEM-US). It further uses Webster's method to estimate signal timing parameters in the absence of field timing data. To facilitate the application of these methods, guidance was given on six specific input variables, including: (1) hourly directional demand distribution, (2) total delay due to turns into mid-segment access points, (3) lost time per cycle, (4) proportion of arrivals during green, (5) delay due to mid-segment sources, and (6) turning vehicle percentage estimation.

Of these defaults, the turning movement estimation method represents the most significant contribution. The method allows the estimation of turning percentages using a gravity-model approach, which was tested using over 100 intersection turning movement counts and corresponding AADTs from cities in North Carolina.

All models and methods have been implemented into the ARTVAL computational engine. The engine is based on the Microsoft Excel/Visual Basic platform, and allows estimation of urban street performance in a "data poor" and planning-level context. If available, more detailed operational data can be

substituted in the engine, making the tool very versatile and user friendly. The tool is customized to allow for evaluation of arterial work zone impacts, but also functions well as a non-work zone tool due to its ability to evaluate an extended 24-hour period. This sets the tool apart from more traditional tools that focus on the analysis of the peak 15 minutes. The ARTVAL tool has been compared to two commercially available software tools and performed reasonably well, but with a much reduced data entry burden compared to these other tools.

This final report outlines the research results and methodology, as well as a software tool that can be applied directly by NCDOT. The results of this research will be important to NCDOT to facilitate decision-making on work zone scenario selection for arterial streets, with an effort to balance anticipated impacts to the traveling public with other departmental project objectives and constraints.

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Stuart Bourne, P.E. (Chair)
Lawrence Gettier, P.E.
Steve Kite, P.E.
Dan Thomas, P.E.
James H. Dunlop, P.E.
Randy A. Garris, P.E.
Marsha Sample
Phillip Johnson, P.E.
Mrinmay Biswas, PhD., P.E.
Ernest Morrison, P.E. (Project Manager)

The authors would like to thank all members of the Steering and Implementation Committee for their continued guidance and support throughout this project. We would like to express special gratitude to the committee chair for his time in overseeing this project. Special thanks also goes to Mr. Ernest Morrison of the research unit, who continues to help guide the project along and offered assistance with meeting scheduling and contractual questions.

Without the help of all the above individuals, the project would not be able to be completed in a successful and timely manner.

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1. Introduction

The analysis of operational impacts of work zones is important to estimate travel time, delay, and user cost impacts to the traveling public due to construction activities. While work zone analysis on freeway segments and facilities is discussed quite frequently in the literature, the coverage of arterial work zone analysis and associated methodologies are limited. Specifically, a recently-completed NCDOT research effort developed a methodology and software implementation for freeway work zone analysis, which is currently being used by NCDOT. However, for arterial streets, no work-zone specific analysis methodology exists to date, and empirical data for arterial work zone analysis is limited (especially for North-Carolina specific work zones).

This research seeks to develop a methodology for arterial work zone analysis that is anchored in North-Carolina performance data, and which is implemented in a computational software tool for ready in-house application by NCDOT.

Background

At the present time, the evaluation of work zone impacts on arterial streets is limited by the absence of a methodology that can readily be applied in-house at NCDOT, that is calibrated based on NC data, and that can be applied without the need for extensive simulation-based analyses. Short of the last option, NCDOT is limited to the use of tools like Synchro or HCS, which are restricted to the evaluation of the peak fifteen minutes, as opposed to allowing for analysis of multiple hours up to an entire day. A peak-fifteen minute approach is not suitable for work zone analyses on arterials for two reasons: First, the majority of high-impact work zone activities are deliberately scheduled outside of the peak demand periods; and second, the duration of the work zone activity most certainly will last longer than fifteen minutes. Alternatively, the new Urban Streets methodology in the 2010 Highway Capacity Manual (HCM) now allows for the evaluation of multiple 15-minute periods, but does not consider spill-back to upstream segments. The methodology is also quite computationally intense, relying on an iterative procedure to estimate actuated signal times and performance, which is very data intensive. As such, it can be challenging to apply the HCM method in a planning-level context. So while that method provides key building blocks for work-zone evaluation on arterial streets, further enhancements are needed to model queuing (and the resulting metering of downstream traffic).

As such, it is desirable to develop an analytical methodology that can be used in-house at NCDOT for estimating the impacts of different work zone scenarios in an extended time-space domain. The developed methodology should allow for the consideration of multiple analysis time periods (up to 24 hours), varying capacities of arterial work zones, as well as consideration of congestion spillback effects to upstream segments due to the presence of a work zone, signal-induced queues, or both. The methodology should further be applicable in a planning-context, by minimizing to the extent possible the amount of input data necessary. This can be achieved through the use of default values and/or the use of quick estimation methods and automated procedures to estimate performance.

Objectives

The specific objectives of this research are to:

1) Develop a planning-level methodology for the evaluation of arterial work zones that can be applied (quickly) for in-house analysis of arterial work zone performance impacts, including the impact of signalization;
2) Validate the methodology through targeted field studies at North Carolina work zones to calibrate the method and illustrate its application to North Carolina case studies, and
3) Implement the methodology in a computational software tool for ready application and technology transfer.

The guiding principles for this research are: 1) compatibility with Highway Capacity Manual analysis methodologies for arterial streets, 2) consistency with existing analysis approaches used by NCDOT for work zone evaluation on freeways, 3) consistency with NCDOT practices for non-work zone analyses for arterials, 4) applicability in a planning-level context with limited available input data, and 5) foundation in empirical work zone performance data gathered in North Carolina.

Report Content and Limitations

This report is produced through NCDOT research project 2013-09, which was divided into ten research tasks: 1. Project Work Plan and Schedule, 2. Literature Review, 3. Development of Analysis Framework, 4. Data Collection Plan, 5. Interim Project Meeting and Report, 6. Field Data Collection, 7. Methodology Development, 8. Computational Engine Development, 9. Calibration and Validation of the Methodology, and 10. Final Report and Deliverables. This report represents the deliverable of Task 10: Final Report Project.

The remainder of this report is organized as follows. Chapter 2 presents the literature review summary, Chapter 3 presents the analysis framework for urban street analysis (in a planning context), as well as work zone estimation. Chapter 4 presents the methodology applied in the project, including data collection details based on the analysis framework, and a summary of data reduction and analysis activities. Chapter 5 presents the results of the modeling effort from the collected field data. Specifically, it presents a method for estimating saturation headways in arterial work zones, and discusses additional default values derived through the project. Chapter 6 presents an application of the method to a case study location, and comparison to other software tools used to evaluate the performance of urban streets. Chapter 7 presents a summary of the research, followed by references in Chapter 8.

2. Literature Review

This chapter presents a literature review for work zones on urban arterials. Typical work zone configurations, modeling approaches and data collection methods for arterials are summarized. The literature review is a resource document for generating the models and analytical frameworks for the methodologies to be developed in subsequent research tasks.

Acknowledgement

Part of the material in this chapter was adapted in modified form from existing work, including NCHRP Project 3-107 Work Zone Capacity Methods for the Highway Capacity Manual (Task 1 Working Paper Literature Review), and SHRP 2 Project L08 Incorporation of Non-recurrent Congestion Factors into the Highway Capacity Manual Methods (Working Paper No.6 HCM Urban Streets Methodology Enhancements – Saturation Flow Rate Adjustment Factor for Work Zone Presence).

Urban Street Work Zone Configurations

The Manual on Uniform Traffic Control Devices (MUTCD) identifies seven typical work zone configurations as listed below:

1. Shoulder or Sidewalk Work
2. Signalized Intersection – Through Lane Closure
3. Signalized Intersection – Exclusive Turn Lane Closure
4. Mid-segment – Inside Lane
5. Mid-segment – Outside Lane
6. Lane Shift
7. Median Crossover

A thorough understanding of these work zone configurations, as well as their expected operational impacts is an important part of this project. The analysis framework for this project should accommodate all seven configurations, as well as provide the analyst the ability to compare the impacts of different alternative configurations. A discussion of the seven configurations is presented in the following paragraphs

<u>Shoulder or Sidewalk Work</u>

Exhibit 1 shows typical shoulder or sidewalk closures on urban arterials. This type of work zone is expected to have significant impact on pedestrians or bicyclists, and are believed to also have negative influences on auto travel speeds. While no formal lane closures are in effect in this configuration, the team expects that traffic adjacent to this type of work zone will be impacted due to driver distraction, adjacent work activity, and potential dust and visibility effects.

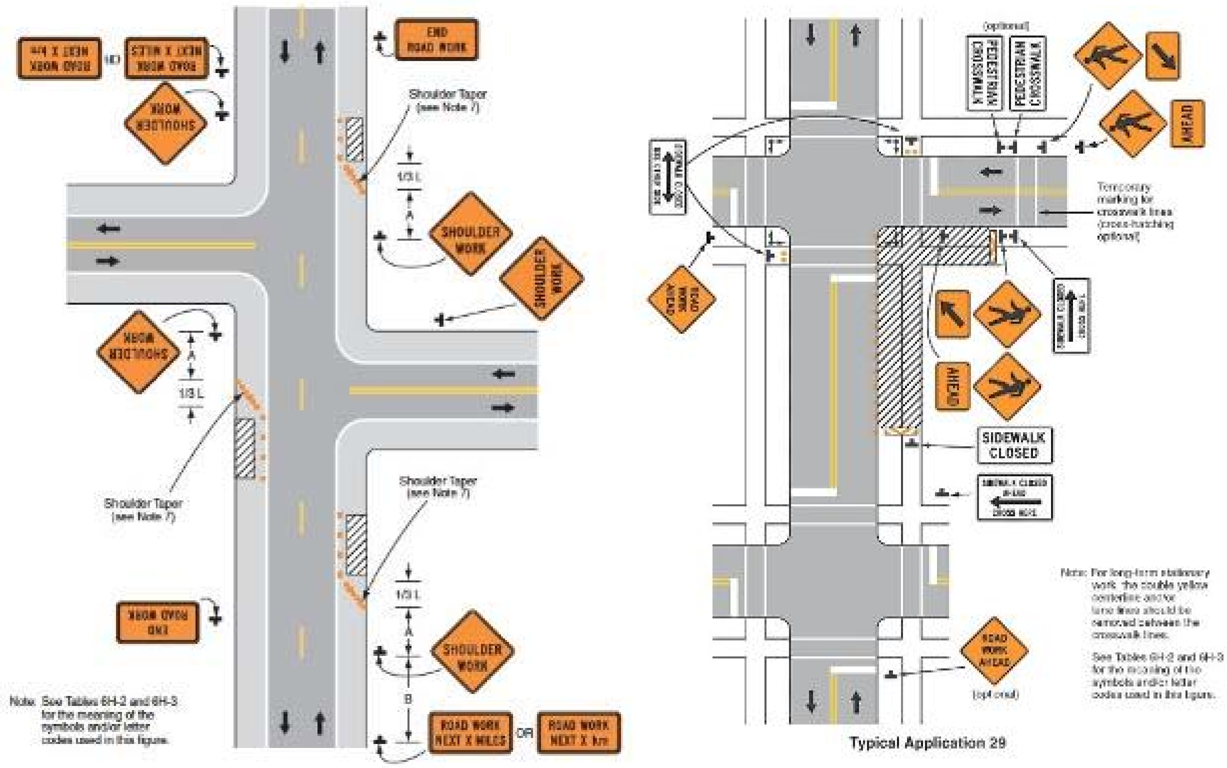


Exhibit 1: Typical Shoulder (Left), and Sidewalk or Crosswalk (Right) Closure

Signalized Intersection – Through Lane Closure

Work zones with lane closures at signalized intersections are expected to have a significant impact on intersection operations. This configuration describes a *through lane closure,* and a schematic of this scenario is shown in Exhibit 2. A through Lane Closure work zone results in the loss of one or multiple through lanes, which may include through and left turn or through and right turn lanes). Depending on the configuration, the lane assignment of open lanes may be modified (for example from an exclusive through lane to a shared through and right turn lane), to offset the loss of capacity. In a macroscopic analysis framework the location of the lane closure is less critical than the resulting capacity impact on the intersection. That capacity effect is postulated to be a combination of the loss of lane, and a supplemental per-lane capacity loss in the open lanes due to friction. Other considerations may be distinction between closures in the outside versus middle lanes, which are expected to impact traffic patterns differently.

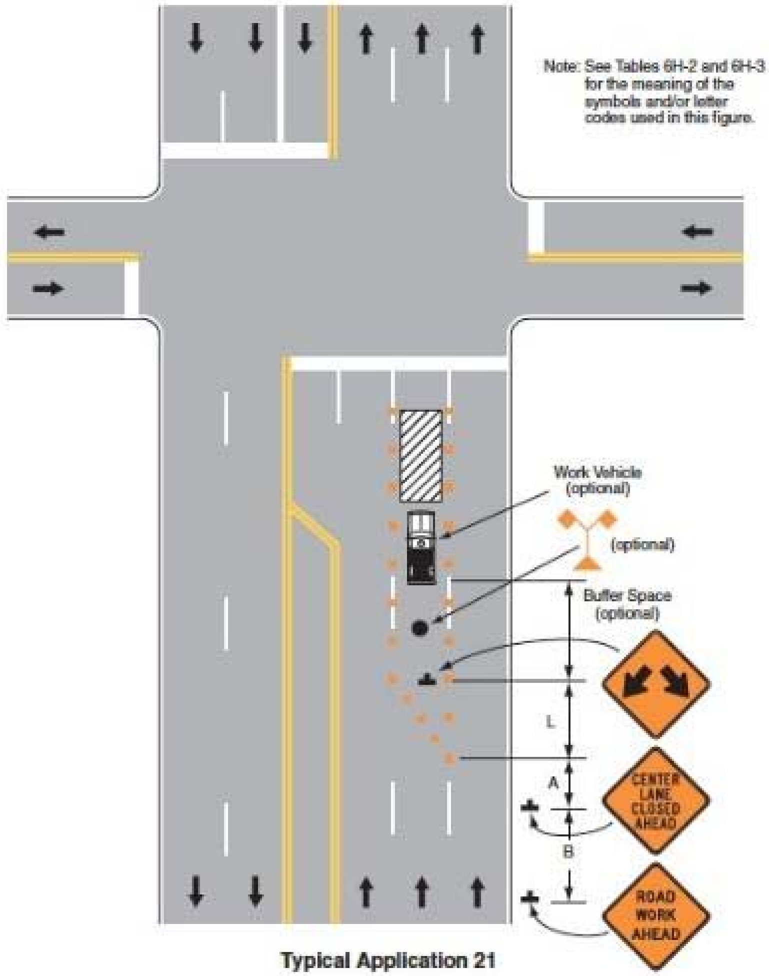


Exhibit 2: Typical Through Lane Closure at Signalized Intersections

Signalized Intersection – Exclusive Turn Lane Closure

A second intersection work zone is classified as an exclusive turn-lane closure. In this case, the exclusive turn lane has been closed and lane functions of other lanes (probably through lanes) need adjustment to accommodate turning maneuvers. In this case, channelization, lane closure, friction, and lane assignment changes would all affect work zone operations.

Mid-segment – Inside Lane

Work zones away from intersections are categorized as mid-segment work zones. Both inside and outside lane closure would be of great interest for this project. A middle lane closure is more common for low traffic areas and is expected to be rare in urban arterials, due to significant impacts on both directions of travel. Exhibit 3 shows the typical configuration of Mid-segment – Inside Lane work zones.

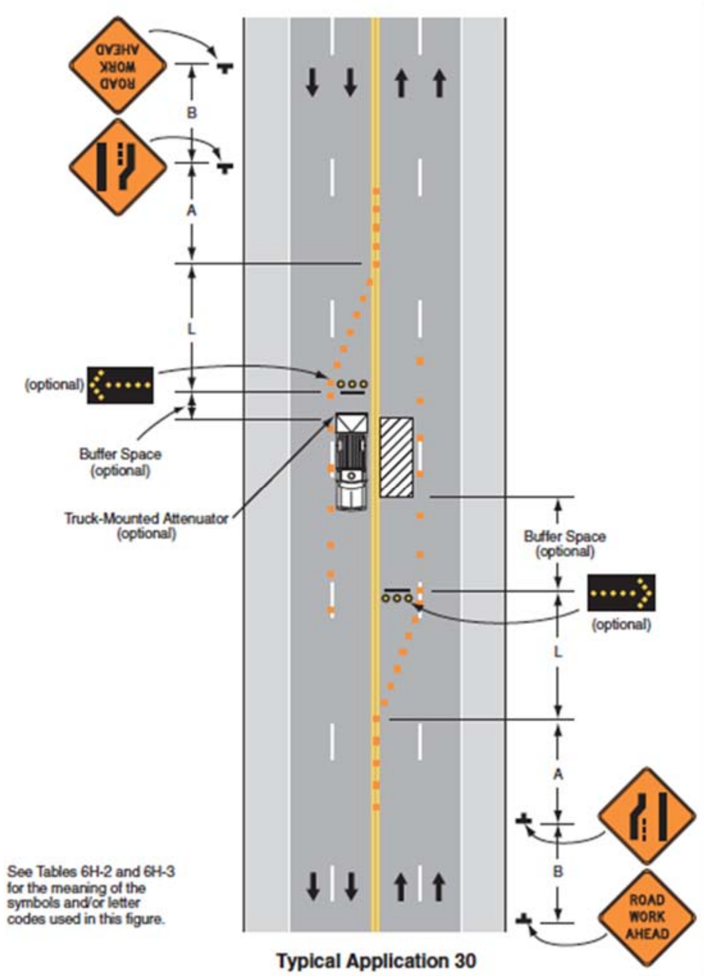


Exhibit 3: Typical Inside Lane Closure at Mid-segment Work Zones

Mid-segment – Outside Lane

Another type of Mid-segment work zone is the Outside Lane Closure. It is also far from intersections and has similar configuration with the Mid-segment – Inside Lane work zones. Mid-segment work zones may be combined with lane Shift configuration (introduced below) to keep the original number of lanes. Channelization and possible lane closure are the major influential factors to operation in this type of work zone.

Lane Shift

Lane shift work zones shift through lanes using the shoulder or median to avoid the work activity area. In this case, channelization and possible lane closure are the main influential factors to traffic operation. An example of a typical lane shift work zone configuration is shown in Exhibit 4.

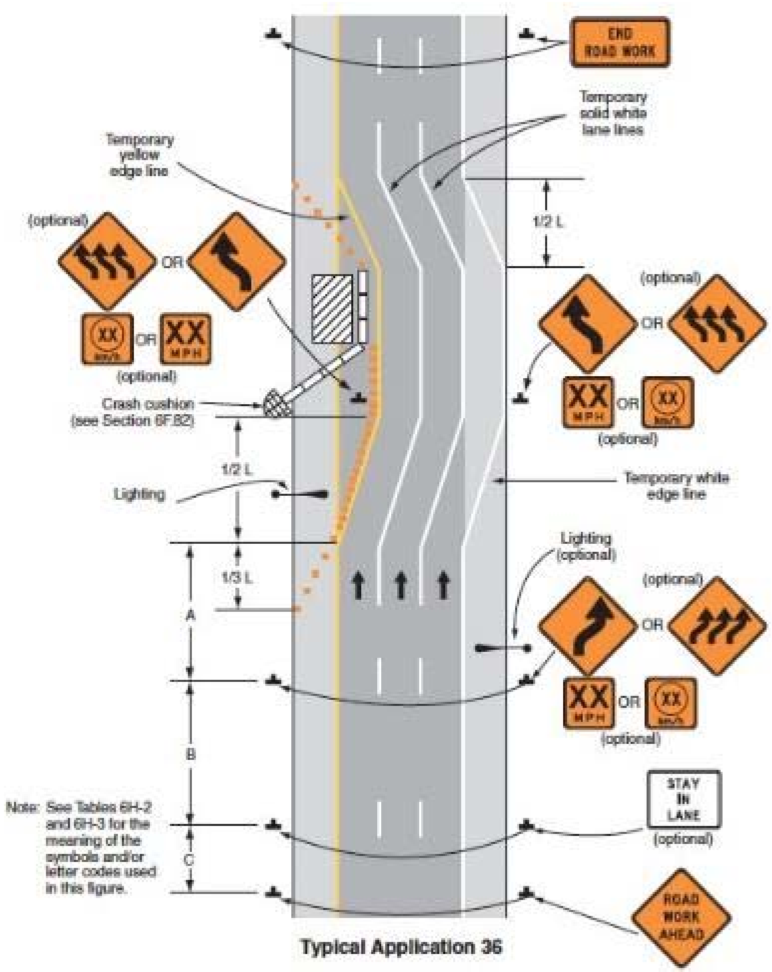


Exhibit 4: Typical Lane Shift Work Zones

Median Crossover

A median crossover work zone configuration is similar to that of a lane shift. The difference is Median Crossover shifts the lane further across the median to the other side of the road because of major work activities at this side, as shown in Exhibit 5. It is likely that a Median Crossover comes with lane closures. This type of work zone could have impacts on traffic operation in both directions.

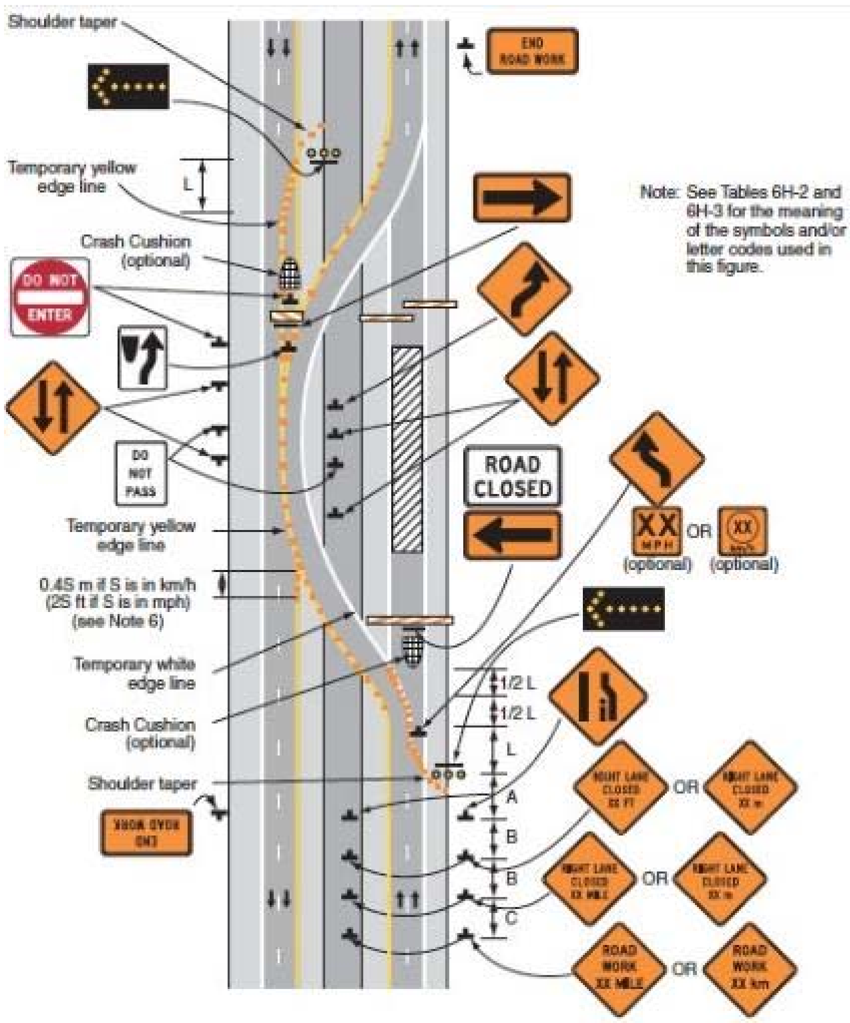


Exhibit 5: Typical Median Crossover Work Zones

Urban Arterial Work Zone Modeling

With an understanding of the seven types of urban arterial work zones, this section presents a review of work zone modeling approaches for urban streets. Most of the work-zone-related publications found in the literature are focused on freeway work zones and their effect on freeway operations. However, a few publications were found that addressed work zone presence on urban streets. These publications about work zone operation, capacity, evaluation and simulation methods are the focus of this section. It provides a general review of factors believed to affect work zones operations, a summary of existing macroscopic modeling approaches, and a review of simulation-based approaches for work zone evaluation.

Factors Impacting Urban Street Work Zones

Several studies have investigated factors that impact work zone operations on urban streets. A study by Hawkins et al. (1992) investigated work zone performance on urban streets in Texas. The authors identified a list of factors that impact work-zone operations at mid-segment locations, including sight distance restrictions, driveway access, narrow lanes, lateral clearance between the work zone and open travel lanes, and pedestrian presence.

In research performed in Florida, Elefteriadou et al. (2008) identified several factors that could describe the operational performance of an intersection work-zone, including the green-to-cycle-length ratio for each lane group, the percent of left-turning traffic, and the relative distance between the work zone and the intersection. Very similar factors were identified in an early simulation-based evaluation of work zone impacts on urban streets by Joseph et al. (1988). The simulation tool showed the effect of the work zone on traffic operations under consideration of signal timing, the percent of platooned arrivals, and the relative distance between the work zone and the signalized intersection.

In a state-of-practice survey of all state DOTs in the US, Kianfar et al. (2010) asked respondents about the factors that were believed to primarily influence work zone capacity. While the survey was primarily focused on freeway work zones, it identified four key factors believed to impact urban street work zone performance, including work zone length, number of open lanes, lane width, and heavy vehicle percentage.

The factors impacting urban street work zone operations were summarized in a recent paper (J. Bonneson and K. Nguyen (2012)), which is summarized in Table 1.

Exhibit 6: Factors Impacting Work Zone Operations (Source: J. Bonneson and K. Nguyen (2012))

Category	Factor [1]
Work zone data	Work zone length
	Location of closed lane (outside, middle, or inside; parking)
	Work intensity (presence of equipment and workers)
	Work duration (number of days since work zone installed)
	Police presence
	Time of work activity (daytime, nighttime)
Geometry	Number of open lanes in the work zone
	Approach grade
	Lane width in the work zone
	Lateral clearance to the work zone and to opposing lanes
	Driveway presence
	Provision/closure of turn lanes at intersection
Traffic characteristics	Traffic demand volume
	Heavy vehicle percentage
	Lane utilization (or lane volume) on intersection approach
	Turn movement percentages
	Pedestrians at intersection and along street, if sidewalk is closed
Traffic control	Speed limit prior to work zone and speed limit in work zone
	Use of flagger or signal control
	Type of devices used to delineate work zone (cones, barrier, other)
	Effective green duration and cycle length, if signalized

The list of factors in Exhibit 6 represents a set of potential explanatory variables in a model used to predict work zone capacity. While it is unlikely that all of these factors will emerge as significant variables in a capacity model, the table represents a reasonable starting point to assist with data collection planning and the development of an analysis framework for arterial work zones.

Work Zone Capacity Estimates

With the factors identified in the previous section, it is desirable to predict their impact on the capacity of a work zone. The capacity of a signalized intersection is generally expressed as the product of the saturation flow rate, and the effective green to cycle length ratio (g/C). The saturation flow rate is intuitively a function of the various factors identified in Exhibit 6 and will be discussed in more detail below.

The g/C ratio is generally a function of signal timing patterns, which at a work zone can be (a) pre-timed control, (b) actuated control, or (c) controlled by flaggers or police. The effective green time is readily available for a pre-timed control, but analyst judgment is required in the case of actuated signals to estimate an average effective green time. In the case of flagger or police control, the estimation of the g/C ratio can be very challenging, and no clear guidance was identified in the literature. However, with flagger-controlled work zones being very common (WSDOT, 2009), an assessment of flagger operations can be an important facet of work zone analysis.

The base saturation flow rate in the 2010 Highway Capacity is 1,900 veh/h/ln for large urban areas and 1,750 veh/h/ln for smaller communities. The saturation flow rate is the inverse of the saturation headway, which can be measured in the field as the time between successive vehicles discharging from a queued state. The saturation flow rate at the intersection is further related to the mid-segment capacity of the work zone approach.

Compared to the HCM defaults, Hawkins et al. (1992) measured a capacity of only 760 veh/h/ln for a mid-segment short term work zone on a two-lane approach to a signalized intersection with a single lane closure. Presumably, the reduced capacity is a function of adjacent work activity and other friction that impacts driver headways.

While other field studies of work zone capacities at signals are scarce, Elefteriadou et al. (2008) used simulation to develop a set of regression equations to predict work zone capacity. The authors estimated a capacity range from 385 to 1,005 veh/h/ln, as a function of signal timing, approach geometry, and the distance between the work zone and the intersection. Chin et al. (2004) estimated the total capacity loss due to a work zone from the number of lanes normally open, the number of lanes closed due to the work zone, and the length of time the lanes were closed. A summary table of arterial work zone capacity estimated values from current available literature are listed in Exhibit 7 below.

Exhibit 7: Arterial Work Zone Capacity Estimated Values from Current Available Literature

Research	Year	Method	Results	Background / Details
Hawkins et al.	1992	Measurement	760 veh/h/ln	Mid-segment short term WZ on a two-lane approach to a signalized intersection with a single lane closure
Chin et al.	2004	Summary	4.1 billion veh/yr capacity loss	0.43 million mile-days (or total of 1,980 miles) work zone activities
Elefteriadou et al.	2008	Simulation / Regression	385 to 1,005 veh/h/ln	A function of signal timing, approach geometry, and the distance between the work zone and the intersection.

Macroscopic Modeling Approaches

In a review of the literature, only few existing modeling approaches for urban street work zones were identified. The primary resource for signalized intersection and urban street analysis in the US, the

Highway Capacity Manual (HCM 2010), presently does not offer any methods for evaluating urban street work zones, or for estimating work zone impacts on saturation flow rate, mid-segment running speed, or other methods. However, a national research project (NCHRP 3-107) is ongoing that is tasked with developing these methods for the Highway Capacity Manual. This team is involved in this national project, and will coordinate efforts between the two projects closely.

Another parallel project funded by the Federal Highway Administration (FHWA) will collect traffic data before, during and after work zone construction at several sites to develop a framework for design, operation and decision-making related to urban arterial work zones (A. Varma, et.al 2012).

J. Bonneson and K. Nguyen (2012) recently proposed saturation flow rate adjustment factors for work zone presence during the SHRP L08 project. This method is further expected to be adopted in the next version of the Highway Capacity Manual (HCM). Using a regression-based modeling approach, the authors developed a new saturation flow rate adjustment factor for work zones at signalized intersections. This study concludes that the work zone impact adjustment factor depends primarily on lane configuration, with independent variables including the number of lanes with and without the work-zone, as well as the approach lane width. The resulting equation is shown below:

$$f_{wz} = 0.858 \times f_{wid} \times f_{reduce} \leq 1.0 \qquad \textbf{Equation 1}$$

$$f_{wid} = 1.0/[1.0\text{-}0.0057(a_w - 12)]$$

$$f_{reduce} = 1.0/[1.0+0.0402(n_o - n_{wz})]$$

where,

f_{wz} = saturation flow rate adjustment factor for work zone presence;
f_{wid} = saturation flow rate adjustment factor for approach width;
f_{reduce} = saturation flow rate adjustment factor for reducing lanes during work zone presence;
a_w = approach lane width during work zone (= total width of all open left-turn, through, and right-turn lanes), ft;
n_o = number of left-turn and through lanes open during normal operation, ln; and
n_{wz} = number of left-turn and through lanes open during work zone presence, ln.

Due to a limited number of sites and work zone configuration, the study was not able to isolate effects of other work zone configuration factors, such as pavement surface, and work activity.

In an earlier approach, the Florida DOT's *Plans Preparation Manual* (2009) offers equations for estimating the capacity of both mid-segment and intersection work zones. The capacity of mid-signal work zones is estimated using equation 2 below.

$$c_{ms} = n \times c_o \times f_o \times f_{wz} \qquad \textbf{Equation 2}$$

where:

c_{ms} = capacity of lanes adjacent to a mid-signal work zone, veh/h;
c_0 = base capacity (= 1,800), veh/h/ln
n = number of open lanes adjacent to work zone, ln;

f_o =obstruction factor (= 1.0 if lateral clearance is 6 ft and lane width is 12 ft; 0.65 if lateral clearance is 0 ft and lane width is 9 ft); and

f_{wz} = work zone factor (= 0.99 if work zone length is 200 ft; 0.72 if length is 6,000 ft).

For work zones that are located at a signalized intersection (or less than 600 ft upstream) the Florida manual offers equation 3 below for estimating the work zone capacity.

$$c_{\text{int}} = c_{ms} \times g / C \qquad \textbf{Equation 3}$$

where:

c_{int} = capacity of lanes adjacent to a work zone on an intersection approach, veh/h;

g = effective green duration, s; and

C = cycle length, s.

It is emphasized that the c_{ms} term estimated in equations 2 and 3 is identical to the saturation flow rate concept used in the Highway Capacity Manual, and as estimated by Bonneson and Nguyen (2012) in equation 1.

Even though modeling approaches specifically designed for arterial work zones are rarely documented except those above, some modeling ideas from freeway work zones may be adapted to arterial applications. J. Weng and Q. Meng (2012) introduce an Ensemble Tree approach to estimating work zone capacity. A bootstrap aggregation method is employed to build an ensemble tree comprising of a set of individual decision trees. More specifically, a set of bootstrap samples is first generated by sampling with replacement from a training sample. With these bootstrap samples, a set of individual trees are constructed by using a tree-learning algorithm, and then combined by averaging the output. The proposed Ensemble tree method was shown to generate better results than either the HCM model or the single decision tree methods do.

Q. Mend and J. Wend (2011) tested an Improved Cellular Automata (ICA) method for work zones on freeways and major arterials at Singapore. This model needs be calibrated with work zone vehicle composition, activity length and transition length, and traffic speeds in and outside work zone. Even though this model could handle such factors like vehicle composition much better than the traditional Cellular Automata model does, it would become very complicated to consider more of other important factors.

Apart from the traffic characters in the work zone, driver diversion before the work zone, at the planning level, may be another important factor related to user cost. Several studies have revealed the pattern of driver diversion before work zones. For instance, H. Lee (2009) studied the route-changing behaviors of road users at urban street work zones. Work zone plans should take these behaviors into account, so that delay or user cost could be reduced.

Simulation-Based Approaches for Urban Street Work Zones

As an alternative to the macroscopic (HCM-style) approaches for work zone analysis, simulation offers another viable approach for evaluating arterial work zone operations and impacts. Microsimulation tools are routinely applied for the evaluation of arterial street systems, and are generally capable of modeling a diverse array of geometric and operational configurations, including various signalized intersection details. In application of simulation tools to work zone analysis, a challenge can be the calibration of work zone capacity and speed inputs to match field expectations. Similar to the capacity estimation described for the macroscopic approaches above, simulation therefore relies on some degree of prior knowledge as to the expected work zone performance.

The literature shows that various researchers have explored the use of simulation for work zone evaluation, while the use by agencies is limited. In a 2006 survey of 19 state DOTs Edara and Cottrell (2006) microsimulation was said to be rarely used for freeway work zone evaluation, and a more recent survey found that 60% of 29 surveyed agencies preferred the HCM procedures to estimate work zone capacity over simulation Kianfar et al. (2010).

The use of simulation in research is more common for freeway work zones, where several authors have used simulation. For example, Benekohal et al. (2003) used FRESIM to model freeway work zone analysis, but found that the tool overestimated speed in the work zone when queuing was present. Similarly, Chatterjee (2008) developed a procedure for calibrating VISSIM microsimulation so it can be used to model freeway work zone accurately. The need for proper calibration to properly model work zones in simulation was also identified by Schnell et al. (2002), who proposed that car-following and lane-change parameters need to be calibrated to assure proper operations, as well as emphasized a careful look at truck characteristics. A review of the tools being used for work zone analysis including microsimulation, mesoscopic simulation, and simpler spreadsheet tools is provided by Hardy and Wunderlich (2008).

An example of applying simulation to urban street work zones is given by Elefteriadou et al. (2008), who used CORSIM to explore the effects of work zone configuration and location on signalized intersection approach capacity. However, the authors noted that most simulation tools lack explicit features for modeling work zones on urban streets, but rather rely on custom configuration by the analyst to approximate work zone performance. The authors identified several desirable features that should be include in work zone simulation models, including rubbernecking behavior, the ability to specify location and severity of the work zone, and the ability to model upstream driver information systems that alert modeled vehicles of the presence of the work zone.

Summary of Modeling Approaches

Available tools from literature for urban arterial work zone modeling have been reviewed in the sections above, and a summary is presented below:

- Work zone saturated flow rate adjustment factors regression models, studied by J. Bonneson and K. Nguyen (2012) and Florida DOT's *Plans Preparation Manual* (2009), are useful in determining capacity at arterial work zones. Factors that affect work zone capacity should be carefully investigated and calibrated. This models, if applied, should be expanded and thus fit with all arterial work zone configurations listed in the chapter above;

- The parallel ongoing NCHRP and FHWA projects are expected result in further guidance, and will be an asset to the modeling approaches in this project;
- Complicated approaches like Ensemble Tree and Improved Cellular Automata (ICA) are available to model arterial work zone, but require extensive efforts for redevelopment and calibration;
- Microsimulation software package, such as VISSIM, FRESIM and CORSIM, are available for work zone modeling. They have the capability to output a rich set of measurement of effectiveness (MOE's), as long as the car-following and lane-changing models are calibrated by the local work zone data.

Urban Arterial Work Zone Data Collection

This section summarizes data collection methods for arterial work zones available from the literature. J. Bonneson and K. Nguyen (2012) described a detailed data collection protocol for development of saturation flow rate adjustment factors for arterial work zones used in SHRP-2 project L08. The authors focused their study on one intersection approach, and collected data separately for each lane group following HCM2010 definitions. The authors used a "during-after" experimental design, that collected the same data while the work zone was ongoing, as well as after all work activity had completed.

Site Selection Criteria

The selection of suitable study sites by Bonneson and K. Nguyen (2012) were based on the AADT, work zone end date, work zone duration, and number of lanes closed for the work zone. The volume criterion was established as a minimum AADT of 3550 veh/d per lane, to assure that some congestion and queuing would be present during the work zone. The work zone end date criterion was used to ensure that the work zone would be removed in a timely manner, such that the "after" study could be completed within the time schedule of the research project. The other two criteria were used to guide site selection such that a range of values for each criterion were represented in the database.

In addition to the site selection attributes used by Bonneson and K. Nguyen (2012), several other factors emerged from the literature that should be considered:

- Pavement condition, including travel on milled pavement surface, or travel across a ledge that results in a reduction in speed,
- Work activity, including the presence of heavy equipment or lights that may impact driver behavior,
- Daytime vs. nighttime conditions, to explore the performance of the work zone at night.
- Horizontal and vertical design details, such as work zones located in sharp curves or on hill crests that cause sight distance constraints,
- Approach grade, and the impacts of work-zones located on non-flat geometries

A general data collection approach for arterial work zones is the use of video cameras, with a preference to overhead locations. Video recordings are readily used to count throughput through the work zone, and more specifically, measure saturation headways of various configurations. Video is also useful to confirm the type of work activity and to isolate different scenarios.

Video can be limited in its ability to show queue lengths generated by work zone congestion, as well as estimate vehicle delay or travel time through the work zone system. Video is further inadequate in most cases to estimate speeds through the work zone. As such, alternate field studies including manual tally sheets, floating car runs, or Bluetooth travel time studies are desirable supplements to the video data collection.

Work Zone Capacity Measurements

In conducting field studies of mid-segment work zone capacities, a common difficulty is identifying periods of congestion that coincide with work activity. Oftentimes, work activity on urban streets is limited to short-term construction during off-peak periods, with a requirement to the contractor that any lane closures be removed during peak hours (WSDOT, 2009). As a result, several researchers have documented the challenge of observing urban street work zones that operate at capacity (Hawkins et al., 1992; Elefteriadou et al., 2008).

In terms of performing actual capacity measurements, mid-segment work zones are evaluated using similar techniques to that used to measure freeway work zone capacity. The video recordings (or field observation) would measure the throughput past the work zone under periods of sustained upstream demand. Most commonly, these measurements are performed near the center of the work activity (Kianfar et al., 2010).

For work zones at signalized intersections, the HCM computes capacity for each approach lane group based on its saturation flow rate, lost time, lane utilization, number of lanes, and signal timing. This computational approach is typically more useful than field-measuring the actual capacity. Instead, field studies at signalized intersections focus on measuring the various component of the HCM capacity estimation procedure, with the most important factor being the saturation flow rate.

The approaches for measuring saturation flow rate or other factors at signalized intersections are not unique to work zones, and can be borrowed from standard intersection studies as described in the ITE Manual for Transportation Engineering Studies (ITE, 2010).

3. Analysis Framework

Introduction

This chapter introduces an analysis framework to estimate the capacity of work zones on arterial streets in a macroscopic, planning-level context. The team identified several key factors to be considered in the models as the main determinants of arterial work zone capacity. Data will be collected to identify the impacts of different work zone configurations (i.e. number of open and closed lanes), pavement condition, effect of pavement ledges, intensity of work activity, and the heavy vehicle percentage. In addition, the impacts of other factors such as lighting conditions (i.e. day vs. night), and weather conditions are likely to be captured while data are being collected.

The discussion on the analytical framework is presented in four parts: (1) urban street framework, (2) work zone framework, (3) user cost framework, and (4) computational engine framework. In the first section, the framework of modeling urban street segments is introduced. This process is based on the HCM2010 quick estimation approach for urban street analysis. In the second section, the analysis framework for modeling work zones in urban streets is explained. The third section summarized the approach for user cost estimation from a prior NCDOT research effort, followed by a description of the proposed computational engine structure in the fourth section.

Urban Streets Framework

The urban streets analysis framework is itself divided into three sections: (A) estimation of signal timing parameters, and (B) evaluation of undersaturated conditions. The section concludes with a summary of input needs of the various methods, as well as a discussion of defaults and estimation methods for the various inputs.

For signal timing, the 2010 Highway Capacity Manual (HCM2010) offers a Quick-Estimation Method (QEM) for the estimation of signal timing parameters in Chapter 31. This method will be referred to in this report as *QEM for signal timing, or QEM-ST*. In undersaturated conditions, where the traffic demand level is below the capacity of the urban street segment, the team proposes to use a second QEM as explained in Chapter 30 in HCM2010. This method uses traffic flow model equations to estimate the capacity and running time of an urban street segment in undersaturated conditions. This method will be referred to as the *QEM for urban streets, or QEM-US*, to distinguish it from QEM-ST. For oversaturated conditions, there is no suitable method available in the HCM2010 for urban street estimation with queuing. While a new methodology is being proposed for the next HCM update, it is not included here due to its computational complexity.

The QEM-ST method is presented here to document the initial analysis framework. However, over the course of the project, and in developing the ARTVAL computational engine, the team decided to forgo the use of the QEM-ST method, and instead use Webster's method to estimate signal timing for intersections for which no field data are available. QEM-US is used in the final computational engine as described below.

Method for Estimating Signal Timing – QEM-ST

Chapter 31 of the 2010 HCM provides details on QEM-ST. The main objective of this method is to estimate volume-to-capacity ratio, signal timing, and control delay at a signalized intersections in cases where detailed timing data are not available. While the team expects the framework to be flexible and customizable to enter facility-specific timing data, it is expected that for many facilities the signal timing details may not be readily available. QEM-ST thus allows the method to be broadly applicable to include even such data-poor facility.

The QEM-ST method has the following five key steps:

1- Determine left-turn treatment,
2- Determine lane volume,
3- Determine signal timing,
4- Determine critical intersection volume-to-capacity ratio, and
5- Determine control delay.

Step (1): Determine Left-turn Treatment

The quick estimation method for signal timing can be used to determine left-turn treatments. The method goes through four checks to identify the left turn treatment. As soon as it is determined that a protected left turn phase is required, the rest of checks are unnecessary. A left turn is recommended when:

a) the number of left turn lanes is more than one,
b) the unadjusted left turn volume exceeds 240 veh/h
c) the cross-product of the unadjusted left turn volume and opposing mainline volume exceeds 50,000, 90,000, or 110,000 for 1, 2, or 3 opposing lanes respectively, and
d) either the unadjusted left-turn volume exceeds the sneaker capacity or the equivalence factor exceeds 3.5.

In this project, this step may be skipped by the user if the left-turn treatment is known.

Step (2): Determine Lane Volume

This step determines lane volumes for right-turn movements, left-turn movements, through movements, through movements with exclusive turn lane, and through movements with shared lanes. Details on determining the lane volumes are available in Chapter 31 of the HCM 2010. That chapter offers Exhibit 8, which can be used to estimate lane volumes. In this project, the lane-volume step is largely automated, to allow the estimation of signal timing parameters.

Exhibit 8 Quick Estimation Lane Volume Worksheet (Source: HCM2010 Exhibit 31-39)

QUICK ESTIMATION LANE VOLUME WORKSHEET			
General Information			
Description/Approach			
Right-Turn Movement			
	Exclusive RT Lane		Shared RT Lane
RT volume, V_R (veh/h)			
Number of exclusive RT lanes, N_{RT}			use 1
RT adjustment factor,[1] f_{RT}			
RT volume per lane, V_{RT} (veh/h/ln) $V_{RT} = \frac{V_R}{(N_{RT} \times f_{RT})}$			
Left-Turn Movement			
LT volume, V_L (veh/h)			
Opposing mainline volume, V_o (veh/h)			
Number of exclusive LT lanes, N			
LT adjustment factor,[2] f_{LT}			
LT volume per lane,[3] V_{LT} (veh/h/ln) $V_{LT} = \frac{V_L}{(N_{LT} \times f_{LT})}$	Permitted LT, use 0 ___	Protected LT ___	Not Opposed LT ___
Through Movement			
	Permitted LT	Protected LT	Not Opposed LT
Through volume, V_T (veh/h)			
Parking adjustment factor, f_P			
Number of through lanes, N_{TH}			
Total approach volume,[4] V_{TA} (veh/h) $V_{TA} = \frac{V_{RT}(\text{shared}) + V_T + V_{LT(\text{unopposed})}}{f_P}$			
Through Movement with Exclusive LT Lane			
Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{TA}}{N}$			
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{TH}, V_{RT} (exclusive), V_{LT}]			
Through Movement with Shared LT Lane			
Proportion of left turns, P		Does not apply	Does not apply
Equivalence factor, E_{LT} Shared lane LT adjustment factor, f_{SL}		Does not apply	Does not apply use 1.0
Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{TA}}{(N_{TH} \times f_{SL})}$			
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{RT} (exclusive), V_{TH}]			
Notes			
1. For RT shared or single lanes, use 0.85. For RT double lanes, use 0.75. 2. For LT single lanes, use 0.95. For LT double lanes, use 0.92. For a one-way street or T-intersection, use 0.85 for one lane and 0.75 for two lanes. 3. For unopposed LT shared lanes, N_{LT} = 1. 4. For exclusive RT lanes, V_{RT} (shared) = 0. If not opposed, add V_{LT} to V_T and set $V_{LT(\text{unopposed})}$ = 0. 5. V_{LT} is included only if LT is unopposed. V_{RT} (exclusive) is included only if RT is exclusive.			

Step (3): Determine Signal Timing.

In this step a feasible signal timing plan is estimated for the intersection. The procedure includes five steps as follows:

a) develop phasing plan,
b) compute critical phase volume and lost time,
c) compute critical sum and cycle lost time,
d) compute cycle length, and
e) compute green time

The method recommends several phase plans for the user to choose the appropriate plan for the intersections. Phase selection is based on the left turn treatment identified in step 1. The team expects to develop simple default phasing plans such as “full eight-phase control”, “six-phase split phasing”, or “five-phase plan for minor side-streets”.

In the next step, the critical phase volume and its associated lost time is determined. Default values for lost time are available in Chapter 31 of HCM 2010 and will be incorporated into the methodology in this project. In addition, for each phase plan, critical volumes are identified in Exhibit 31-44 of HCM2010. Based on the critical phase, the critical sum and cycle lost time are determined. Finally, the effective green time is allocated to each phase proportional to the contribution of its critical phase volume to the critical sum.

At the end of this step, the method has estimated sufficient timing details to apply the quick-estimation method for urban streets, QEM-US. The next two steps of QEM-ST are presented for completeness, but will not be needed in the application to the arterial work zone analysis method used in this project.

Step (4): Determine Critical Intersection Volume-to-Capacity Ratio

This step estimates the critical intersection volume-to-capacity ratio for isolated intersections. In this project, this step is replaced with a similar estimation in the QEM-US method described below. For completeness, the volume-to-capacity ratio for QEM-ST would be determined using the following equation:

$$X_c = \frac{CS}{1700 * PHF * f_a \left(1 - \frac{L}{C}\right)}$$

Equation 4

Where:

X_c: Critical intersection volume-to-capacity ratio,

CS: Critical sum,

PHF: Peak hour factor,

f_a: Adjustment factor for area type,

L: cycle lost time, and

C: cycle length.

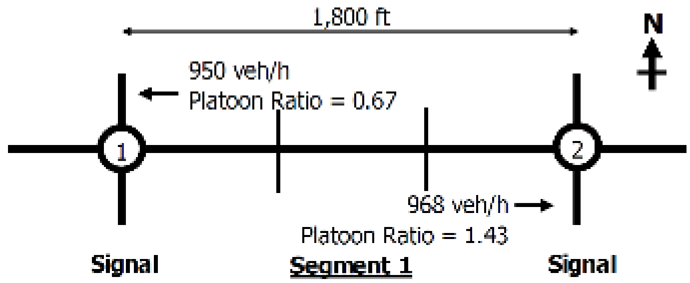
Step (5): determine Control Delay

In this step, using the results of all previous steps, the control delay is determined. In the proposed approach of this study the control delay is determined as a part of QEM-US, as described in the next section.

Urban Streets Framework for Undersaturated Conditions – QEM-US

The Quick Estimation Method for Urban Streets (QEM-US) is fully described in Chapter 30 in HCM2010. The QEM-US method is developed to evaluate the operation of an undersaturated coordinated urban street segment with signalized boundary intersections. An example of an urban street segment is shown in Exhibit 9. The main focus of the approach is to analyze the performance of the through traffic movement at the boundary intersections. It is important to note that the comprehensive “Urban Street Method” described in Chapter 17 of HCM2010 will not be used due to its extensive input data requirements, as well as additional runtime.

Exhibit 9 Example of an Urban Street (Source: HCM2010, Exhibit 30-13)

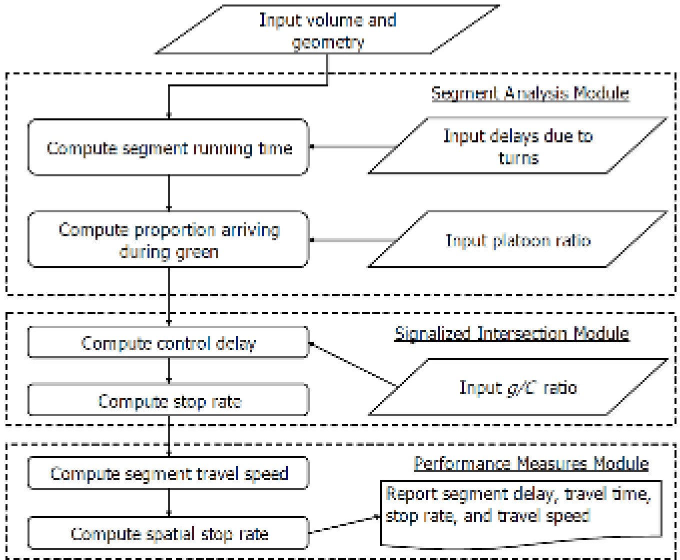


In the example in Exhibit 9, the urban street segment is shown with an intersection spacing of 1,800 feet between two adjacent intersections. The performance approaching signal 1 from the east and signal 2 from the west is primarily a function of the arrival volume, and the platoon ratio. In this framework, the team proposes to develop defaults for estimating the platoon ratio for different facility types (e.g. downtown, principal arterial, minor arterials, etc.).

For the traffic volume input, the team envisions allowing the use of Average Annual Daily Traffic (AADT) values for the facility, and then using a default distribution of hourly factors (as well as directional and peak-hour factors) to estimate traffic demands in the analysis periods. This approach is consistent with the existing analysis methodology for work zones on freeways in North Carolina (implemented in FREEVAL-WZ), but will be customized here to reflect traffic distributions on arterial streets.

The QEM-US method consists of five computational steps to determine (1) segment running time, (2) proportion arriving during green indication, (3) through movement control delay, (4) through movement stop rate, and (5) travel speed and spatial stop rate. Exhibit 10 presents the process flow of the QEM-US method for urban street segments. As shown in the exhibit, the process includes three main modules corresponding to the five computational steps. Each step is also associated with several input variables. The team expects to provide default values for input variables whenever possible; however, the user will have the option of overwriting all or some of the default values if data are available.

Exhibit 10 Process Flow of the QEM-US for Urban Street Segments (Source: HCM2010, Exhibit 30-7)



After entering volume and geometric input data, the segment analysis module computes (1) segment running time and (2) proportion of traffic arriving during green. The signalized intersection module uses g/C input data to compute (3) control delay, from which (4) stop rate is calculated. Finally, the performance measure module computes (5) segment travel speed and spatial stop rate. In the remainder of this section, each computational step is described in more detail. A summary of all required input data for the QEM-US method is presented in Exhibit 11, followed by additional detail on the five computational steps in Exhibit 12 through Exhibit 15.

Exhibit 11 Input Data Required for QEM-US

Data Category	Location	Input Data Element
Traffic characteristics	Boundary intersection	Through-demand flow rate
		Through-saturation flow rate
		Volume-to-capacity ratio of the upstream movements
	Segment	Platoon ratio
		Midsegment flow rate
		Midsegment delay
Geometric design	Boundary intersection	Number of through lanes
		Upstream intersection width
	Segment	Number of through lanes
		Segment length
		Restrictive median length
		Nonrestrictive median length
		Proportion of segment with curb
		Number of access point approaches
Signal control	Boundary intersection	Effective green-to-cycle-length ratio
		Cycle length
Other	Segment	Analysis period duration
		Speed limit

Step (1): Computing the running time

The input data and equations required to estimate the running time are summarized in Exhibit 12.

Exhibit 12 QEM-US Running Time Worksheet (Source: HCM2010 Exhibit 30-8)

RUNNING TIME WORKSHEET			
General Information		**Site Information**	
Analyst	JME	Street	Texas Avenue
Agency or Company	ACME Engr.	Jurisdiction	
Date Performed	9/30/10	Analysis Year	2010
Analysis Time Period	5:30 p.m. to 5:45 p.m.	Analysis Level	planning

Input Data				
	Segment 1		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Segment Data				
Number of through lanes for length of segment (N_{th}), ln	2	2		
Speed limit (S_{pl}), mi/h	35	35		
Midsegment volume (v_m), veh/h	1,150	1,150		
Total delay due to turns into access points (Σd_{ap}), s/veh	0.52	0.52		
Delay due to other midsegment sources (d_{other}), s/veh	0	0		
Length of segment (L), ft	1,800	1,800		
Width of upstream boundary intersection (W_i), ft	50	50		
Length of segment with restrictive median (L_{rm}), ft	0	0		
Length of segment with nonrestrictive median (L_{nr}), ft	0	0		
Start-up lost time (l_1), s	2.0	2.0		
Access Data				
Proportion of street with curb on right-hand side (p_{curb})	0.70	0.70		
Number of access points on right hand side (N_{ap})	4	4		
Running Time Computation				
Adjusted segment length (L_{ad}), ft $L_{ad} = L - W_i$	1,750	1,750		
Proportion of segment length with restrictive median (p_{rm}) $p_{rm} = L_{rm} / L_{ad}$	0	0		
Speed constant (S_0), mi/h $S_0 = 25.6 + 0.47\ S_{pl}$	42.1	42.1		
Adjustment for cross section (f_{CS}), mi/h $f_{CS} = 1.5\ p_{rm} - 0.47\ p_{curb} - 3.7\ p_{curb}\ p_{rm}$	-0.3	-0.3		
Access point density (D_a), access points/mi $D_a = 5280\ (N_{ap,s} + N_{ap,o}) / L_{ad}$	24.1	24.1		
Adjustment for access points (f_A), mi/h $f_A = -0.078\ D_a / N_{th}$	-0.9	-0.9		
Base free flow speed (S_{fo}), mi/h $S_{fo} = S_0 + f_{CS} + f_A$	40.8	40.8		
Segment length adjustment factor (f_L) $f_L = 1.02 - 4.7\ (S_{fo} - 19.5)/\max(L, 400) \le 1.0$	0.96	0.96		
Free-flow speed (S_f), mi/h $S_f = S_{fo}\ f_L$	39.3	39.3		
Proximity adjustment factor (f_v) $f_v = \frac{2}{1 + \left(1 - \frac{v_m}{52.8\ N_{th}\ S_f}\right)^{0.21}}$	1.03	1.03		
Running time (t_R), s $t_R = \frac{6.0 - l_1}{0.0025\ L} + \frac{3600\ L}{5280\ S_f}\ f_v + \Sigma d_{ap} + d_{other}$	33.7	33.7		

**numbers are examples*

As mentioned before, default values will be provided some all input values shown in Exhibit 12 based on the facility type. While some inputs will always be facility-specific (e.g. number of lanes, and speed limit), others will be provided as defaults based on the facility type. For example, a downtown street would be expected to have curb throughout, but no restrictive median. On the other hand, a suburban arterial would be expected to have tighter access control and a higher proportion of restrictive median.

The team has some experience with developing these defaults from a recently-completed NCDOT project on providing defaults for the NCLOS software tool, and expects to provide consistent defaults in this project.

Using the various inputs, the procedure determines adjusted segment length, proportion of segment length with restrictive median, speed constant, adjustment factor for cross section, access point density, adjustment factor for access points, based free-flow speed, adjustment factor for segment length, free flow speed, and the finally running time. All equations are listed in Exhibit 12 in the same order they are supposed to be used.

Step (2): Computing the proportion arriving during green indication

This step is used to compute the proportion of traffic arriving during the green indication to the downstream boundary intersection. Effective green-to-cycle-length ratio and platoon ratio are required as input data. The proportion of traffic arriving during green indication is found by multiplying these two input variables. The team expects to provide defaults for both of these variables based on the functional classification of the facility. For example, a principal arterial (like Capital Blvd.) or a downtown one-way street are expected to have high platoon ratios, while the downtown street would likely have a higher g/c ratio. A collector street would be expected to have a lower g/c ratio. The team will further explore the defaults in this step as the project continues, and will coordinate guidance with the Steering and Implementation Committee. The analyst will further be allowed to customize all defaults if data are available.

Step (3): Computing through movement control delay

The control delay for through movements has two components: a) uniform delay and b) incremental delay. Exhibit 13 lists all input data required to compute uniform and incremental delays for through movements. In addition, all required equations are available in the Exhibit. Using the input data, the analyst computes capacity, volume to capacity ratio, uniform delay, the adjustment factor for upstream filtering, incremental delay, progression adjustment factor, and finally control delay.

Exhibit 13 QEM-US Control Delay Worksheet (Source: HCM2010, Exhibit 3010)

CONTROL DELAY WORKSHEET				
General Information				
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.			
Input Data				
Analysis period (T), h: 0.25	**Segment 1**		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Signal Timing Data				
Cycle length (C), s	100	100		
Effective green to cycle length ratio (g/C)	0.47	0.47		
Traffic Data				
Through-lane group volume (v_{th}), veh/h	968	950		
Lane group saturation flow rate (s), veh/h/ln	1,800	1,800		
Proportion of arrivals during green (P)	0.67	0.31		
Volume-to-capacity ratio (X_u) of the upstream movements	0.57	0.57		
Geometric Design Data				
Number of through lanes (N_{th}), ln	2	2		
Delay Computation				
Capacity (c), veh/h $c = N_{th}\, s\, g/C$	1,692	1,692		
Volume-to-capacity ratio (X) $X = v_{th}/c$	0.57	0.56		
Uniform delay (d_1), s/veh $d_1 = \frac{0.5\,C\,(1 - g/C)^2}{1 - [\min(1, X)\,g/C]}$	19.2	19.1		
Upstream filtering adjustment factor (I) $I = 1.0 - 0.91\,X_u^{2.68} \geq 0.090$	0.80	0.80		
Incremental delay (d_2), s/veh $d_2 = 900\,T\left[(X - 1) + \sqrt{(X - 1)^2 + \frac{4\,I\,X}{c\,T}}\right]$	1.13	1.08		
Progression adjustment factor (PF) $PF = (1 - P)/(1 - g/C)$	0.62	1.30		
Control delay (d), s/veh $d = d_1\,(PF) + d_2$	13.0	25.8		

Step (4): Computing through movement stop rate

The input data and equations that are required to compute the stop rate are summarized in Exhibit 14. This procedure computes effective green time, effective red time, capacity, volume-to-capacity ratio, average speed, threshold acceleration-deceleration delay, deterministic stop rate, second-term back-of queue size and finally full stop rate.

Exhibit 14 QEM-US Through Movement Stop Rate Worksheet (Source: HCM2010 Exhibit 30-11)

STOP RATE WORKSHEET				
General Information				
Project Description: Texas Avenue, 5:30 p.m. to 5:45 p.m.				
Input Data				
Analysis period (T), h: 0.25	**Segment 1**		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Signal Timing Data				
Cycle length (C), s	100	100		
Effective green-to-cycle-length ratio (g/C)	0.47	0.47		
Traffic Data				
Through-lane group volume (v_{th}), veh/h	968	950		
Lane group saturation flow rate (s), veh/h/ln	1,800	1,800		
Proportion of arrivals during green (P)	0.67	0.31		
Speed limit (S_{pl}), mi/h	35	35		
Incremental delay (d_2), s/veh	1.13	1.08		
Geometric Design Data				
Number of through lanes (N_{th}), ln	2	2		
Stop Rate Computation				
Effective green time (g), s $g = C(g/C)$	47	47		
Effective red time (r), s $r = C - g$	53	53		
Capacity (c), veh/h $c = N_{th}\, s\, g/C$	1,692	1,692		
Volume-to-capacity ratio (X) $X = v_{th}/c$	0.57	0.56		
Average speed (S_a), mi/h $S_a = 0.90\,(25.6 + 0.47\,S_{pl})$	39.9	39.9		
Threshold acceleration–deceleration delay, s $(1 - P)\,gX$	8.6	18.1		
Acceleration–deceleration delay (d_a), s $d_a = 0.393\,(S_a - 5.0)^2/S_a$	12.0	12.0		
Deterministic stop rate (h_1), stops/veh $h_1 = \frac{1 - P(1 + d_a/g)}{1 - PX}$: if $d_a \le (1-P)gX$; $h_1 = \frac{(1-P)(r - d_a)}{r - (1-P)gX}$: if $d_a > (1-P)gX$	0.30	0.74		
Second-term back-of-queue size (Q_2), veh/ln $Q_2 = c\, d_2/(3{,}600\, N_{th})$	0.26	0.25		
Full stop rate (h), stops/veh $h = h_1 + 3{,}600\, N_{th}\, Q_2/(v_{th}\, C)$	0.32	0.76		

Step (5): Computing travel speed and spatial stop rate

Required input data and equations to compute travel speed and spatial stop rate are presented in Exhibit 15. Travel speed is determined by dividing the length of the segment by its travel time. Similarly, spatial stop rate is determined by dividing the total number of stops in the segment by its length, see in Exhibit 15.

Exhibit 15 QEM-US Travel Speed and Spatial Stop Rate Worksheet (Source: HCM2010 Exhibit 30-12)

TRAVEL SPEED AND SPATIAL STOP RATE WORKSHEET				
General Information				
Project Description: Texas Avenue, 5:30 p.m. to 5:45 p.m.				
Input Data				
	Segment 1		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Length of segment (L), ft	1,800	1,800		
Base free-flow speed (S_{fo}), mi/h	40.8	40.8		
Running time (t_R), s	33.7	33.7		
Control delay (d), s/veh	13.0	25.8		
Full stop rate (h), stops/veh	0.32	0.76		
Full stop rate due to other midsegment sources (h_{other}), stops/veh	0	0		
Travel Speed Computation				
Travel time (T_T), s $T_T = t_R + d$	46.7	59.5		
Travel speed ($S_{T,seg}$), mi/h $S_{T,seg} = \frac{3{,}600\,L}{5{,}280\,T_T}$	26.3	20.6		
Spatial Stop Rate Computation				
Total stop rate (h_T), stops/veh $h_T = h + h_{other}$	0.32	0.76		
Spatial stop rate (H_{seg}), stops/mi $H_{seg} = \frac{5{,}280\,h_T}{L}$	0.94	2.22		
Level of Service Computation				
Travel speed as a percentage of base free flow speed (R) $R = 100\,S_{T,seg} / S_{fo}$	64.5	50.6		
Level of service (Exhibit 17-1)	C	C		

More detailed information on quick estimation method is available in Chapter 30 of HCM2010. In addition, the Urban Street computational method is described in Chapter 17 of the HCM2010. For this project, the team plans to automate all equations and computational steps described in this section, and provide defaults for as many input variables as possible. While customization and user override will be allowed for all input variables, the team believes that defaults are needed to assure that the method can readily be applied in a planning-level context.

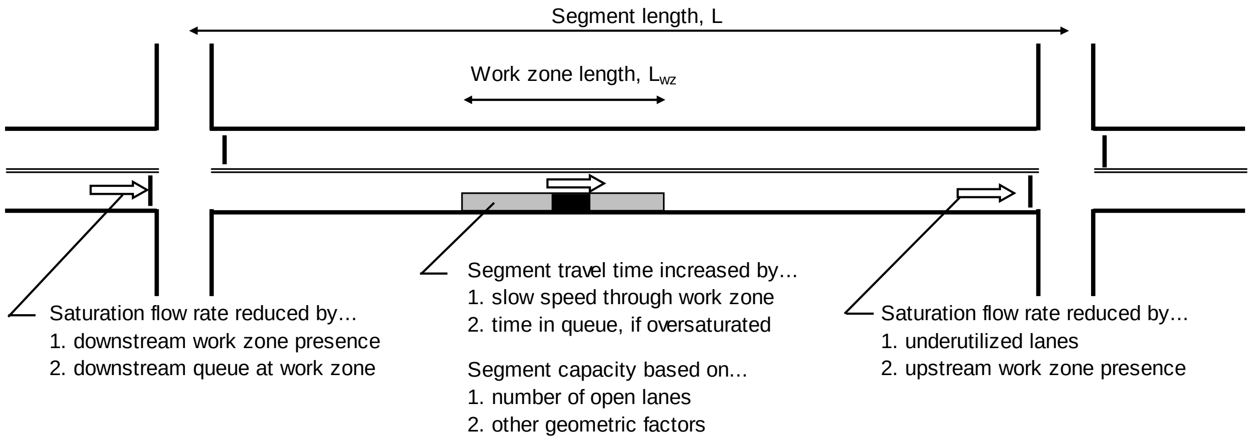
Work Zone Framework

The team proposes a work zone capacity framework based on the urban street method introduced in the previous section. The framework relies on predicting the intersection saturation flow rate when a work zone is active upstream of the intersection, at the intersection, or downstream of it. Intersection capacity is then computed by accounting for the impacts of signals (g/C).

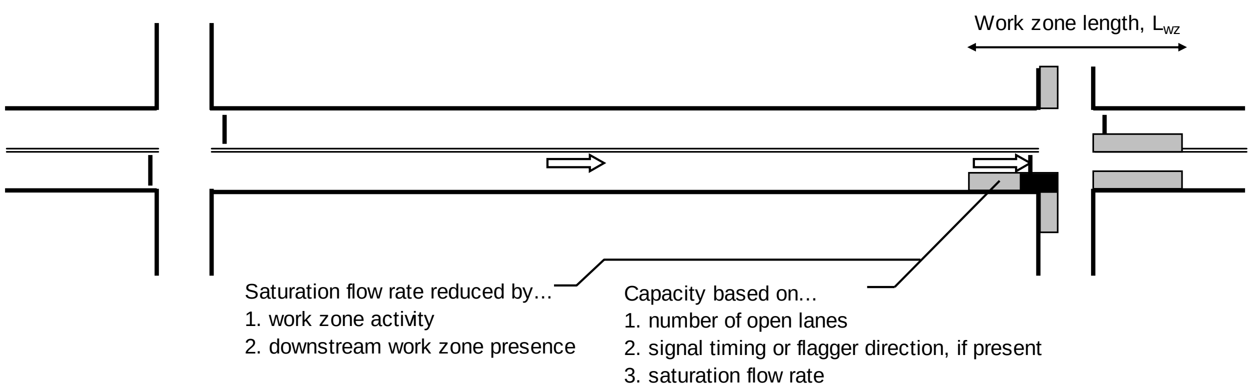
A work zone that is located at a distance from an intersection is called mid-segment arterial work zone, as seen in Exhibit 16-a. These work zones can potentially reduce the capacity of their downstream intersection. This happens when work zone capacity is lower than intersection capacity and as such, starvation occurs. In addition, an arterial work zone may reduce the capacity of its upstream intersection due to queuing. This occurs when the capacity of the work zone is less than the capacity of the upstream intersection, and when the traffic demand level is high-enough to activate the work zone bottleneck.

A work zone that is located at an intersection approach is called intersection work zone, as seen in Exhibit 16-b. This work zone also reduces the capacity of the intersection where it is located at. This capacity reduction is due to the decreased number of lanes, pavement condition, ledge effect, work activity, heavy vehicle presence, etc.

Exhibit 16 Arterial Work Zone (Source: NCHRP 3-107 Working Paper 2)



a) **Mid-segment Work Zone (Influences the capacity of upstream and downstream intersections)**



b) **Intersection Work Zone (Influences the capacity of the intersection where it is located)**

The impacts of arterial work zones on traffic flow can be summarized as follows:

1- Intersection work zone effect on saturation flow rate;
2- Intersection work zone effect on intersection lane capacity;
3- Intersection work zone effect on signal timing;
4- Mid-segment work zone effect on free flow speed;
5- Mid-segment work zone effect on running speed;
6- Mid-segment work zone effect on downstream intersection saturation flow rate;
7- Mid-segment work zone effect on upstream intersection saturation flow rate; and
8- Mid-segment work zone effect on segment lane capacity.

In the rest of this section, all eight impacts of work zone on traffic will be discussed in more detail. In addition, the methodology of quantifying and accounting for these impacts will be discussed.

Intersection work zone effect on saturation flow rate

Intersection work zones can impact the saturation flow rate at the intersection and consequently change its capacity. Items such as the reduced number of lanes, non-ideal pavement conditions, presence of ledges, intensity of work activity, percentage of heavy vehicles, and percentage of turning movement are key contributing factors to changes in saturation flow rate. Regression models can be used to predict the saturation headway in intersection work zones based on the work zone configurations and prevailing conditions. The saturation headway is then converted to saturation flow rate, and is adjusted by g/C ratios to determine capacity.

Therefore, after collecting saturation headway data in several work zones with different configurations, regression models are fitted. Equation 5 is an example of such models.

$$\boldsymbol{h_s = f(N_{CL}, N_{OP}, L_w, PC, L_p, W_a, P_t, DN, Weather, Inc)}$$

Equation 5

Where:

h_s: Saturation headway
N_{CL}: number of closed lanes
N_{PL}: number of open lanes
L_w: lane width
PC: Pavement Condition
L_p: Ledge presence
W_a: Intensity of work activity
P_t: Heavy vehicle percentage
DN: 1 for daytime, 0 for nighttime
$Weather$: 0 for clear, 1 for rain, 2 for snow, and
Inc: 0 for no incident, 1 for incident.

As mentioned above, the saturation flow rate is computed using saturation headway. This is shown in Equation 6 as follows:

$$S = \frac{3600}{h_s}$$

Equation 6

Where:
S: saturation flow rate.

Finally, the capacity of the through movement is determined by accounting for the effect of green time, see Equation 7 below:

$$c = S * \frac{g}{C}$$

Equation 7

Where:
c: capacity of the through movement
g: effective green allocated to the through movement
C: cycle length.

It is important to note that the saturation headway is determined based on a cycle-by-cycle approach. The saturation headway estimate is converted to saturation flow rate, which in turn is used along with the effective green and cycle length to estimate through movement capacity.

Two different model formats can be used to predict saturation headway: (1) additive, or (b) multiplicative. In additive models, a base saturation headway (for non-work zone conditions) is adjusted by subtracting or adding values corresponding to different work zone configurations and prevailing conditions. Equation 8 shows an example of additive models for mid-segment arterial work zones discharge flow rate:

$$\boldsymbol{h_s = h_{s_B} - h_{s_{conf}} - h_{s_{L_w}} - \mathbf{h_{sPC}} - h_{sL_p}}$$

Equation 8

where:

h_s : Saturation headway at the intersection under prevailing conditions
h_{s_B} : Base saturation headway (saturation flow at the intersection without the work zone)
$h_{s_{conf}}$: Saturation headway change due to work zone configuration
$h_{s_{L_w}}$: Saturation headway change due to lane width
h_{sPC} : Saturation headway change due to heavy vehicle percentage
h_sL_p : Saturation headway change due to ledge presence

Multiplicative models are used in HCM 2010 for estimating the adjusted saturation flow rate at signalized intersections (without work zone presence). The model would adjust a base saturation headway for non-work zone intersections by multiplying adjustment factors corresponding to different work zone configurations and prevailing conditions. Equation 5 above introduced an example of such model with a listing of work zone factors that would impact the saturation flow rate.

Intersection work zone effect on intersection lane capacity;

An intersection work zone may be associated with a loss of the number of lanes due to work activity. In addition to the saturation flow rate and capacity per lane, the total capacity is therefore likely to be reduced. In the methodology, this reduction of lanes may impact the capacity of a given lane group (e.g. reduction of three to two through lanes), but oftentimes is associated with a reconfiguration of the approach lane groups. For example, oftentimes work zones may impact an exclusive right or left turn lane, resulting in the creation of shared lanes. In the HCM2010, a shared lane is always evaluated as its own lane group, resulting in an overall modification in the approach capacity estimation. The team proposes to automate the selection of lane groups based on the user-specified work zone configuration.

Also, a temporary flagger control would affect the effective green time and cycle length, and thus the capacity.

Intersection work zone effect on signal timing:

The per lane capacity of a signalized intersection is a function of the aforementioned saturation flow rate, and the effective green to cycle length ratio (g/C) for the approach. The g/C ratio is a direct function of the intersection signal timing. Different work zones may have different impacts on signal timing including (a) use of base timing without adjustments, (b) retiming of intersection under consideration of revised lane configuration, and (c) use of police or flagger control to manage intersection operations. The latter two result in adjustments to the g/C ratio, which needs to be estimated. The team proposes to the development of some defaults in this area, although customization may be needed on an intersection-by-intersection basis.

Mid-segment work zone effect on free flow speed

Presence of a mid-segment work zone can potentially reduce segment free flow speed. This reduction may be due to several reasons, such as non-ideal pavement conditions, narrow lanes, intensity of the work activity, and work zone configurations. The team proposes to use default values for free flow speed under different conditions. The team will explore the feasibility of developing a model for estimating a factor for the reduction in segment free-flow speed under work zone conditions. Such model would require the field-measurement of work zone free-flow speed under variable conditions.

Mid-segment work zone effects on running speed

Not only does a mid-segment work zone reduce the free flow speed, it may further reduce the segment running speed due to additional mid-segment delays. The Chapter 30 QEM-US method currently allows the consideration of *other* mid-segment delay sources in the estimation of segment running speed. The team expects to develop guidance on how this d_{other} factor can be estimated for a given work zone. Conceptually, this added delay is a function of volume. At low volumes, no additional delay (beyond that resulting from the lower free-flow speed) is expected; at higher volumes, the work zone is expected to result in added mid-segment delay. The team will explore options for estimating this additional delay. One simple option may be to adjust the running speed proportional to the adjustment in free flow speed. In other words, the running speed for a certain work zone under prevailing conditions would be adjusted proportional to the free flow speed adjustment for that configuration. It is likely that a more complicated approach is necessary to truly capture the relationship between increasing volume and "other" mid-segment delay resulting from the work zone, but that the detailed data collection necessary for such model is beyond the scope of this research.

Mid-segment work zone effect on downstream intersection saturation flow rate

A mid-segment work zone may potentially reduce the saturation flow rate of a downstream intersection. This happens when the capacity of the work zone is low enough to meter traffic to the downstream intersection. The urban street framework, as discussed previously, is capable of quantifying the impacts of a mid-segment work zone on the saturation flow rate at a downstream intersection by tallying the number of vehicles that are released from one segment to the next. Therefore, there is no need for further customization of this effect specific to work zones.

Mid-segment work zone effect on upstream intersection saturation flow rate

A mid-segment work zone can also reduce the saturation flow rate of an upstream intersection. This happens when a work zone queue spills back to the upstream intersection. The urban street framework is capable of tracking the back of queue and quantifying its impact on the upstream intersection. As a result, collecting data and fitting statistical models are not required to quantify the impacts of mid-segment work zone on an upstream intersection.

Mid-segment work zone effect on segment capacity

Queue discharge flow can be reduced due to the presence of a work zone in mid-segment. This reduction occurs mainly due to reasons such as reduced number of lanes, non-ideal pavement conditions, intensity of work activity, and the presence of heavy vehicles.

Regression models are used to predict the queue discharge flow in mid-segment arterial work zones. For this purpose, queue discharge data needs to be collected in several mid-segment work zones with different configurations and under different prevailing conditions. Regression models are used to identify the key parameters influencing queue discharge flow as well as to quantify their impact. In other words, regression models will be fitted with several independent variables including work zone configuration and prevailing condition to predict the value of the dependent variable being queue discharge. Equation 9 is an example of such models:

$$\boldsymbol{q = f(N_{CL}, N_{OP}, L_w, PC, L_p, W_a, P_t, DN, Weather, Inc)}$$

Equation 9

Where

q: queue discharge flow rate at the mid-segment work zone, and all other parameters are explained previously.

To capture queue discharge flow rate, data needs to be collected when traffic volume is high enough to result in queue formation in the work zones. It is worth mentioning that it is extremely important to accurately estimate the location of the bottleneck to set up data collection devices to capture the break-down and discharging flows. Finally, similar to saturation headway models, queue discharge models may have an additive or a multiplicative form.

Summary of Modeling Approaches

In summary, the proposed analytical framework consists of two main parts: (1) an urban street analysis framework and (b) a work zone analysis framework. The urban street framework is used to estimate signal timing parameters, and evaluate undersaturated conditions. The work zone framework is used to estimate the impacts of an arterial work zone at the intersection and in mid-segment. The combined method allows detailed input of work zone impacts at the intersection, as well as for mid-segment locations.

There are several input variables required in the urban street framework. Exhibit 17 provides a list of this input variables as well as the possibility of providing default values or estimations for each. The table highlights that only very few variables are actually required as user input (bold rows), making the proposed method highly applicable in a "data poor" and planning-level context. But in all cases, customization is possible if the user has access to more detailed data.

Exhibit 17 Summary of Input Variables for Arterial Methodology

Data Item	QEM US[1)]	QEM ST[1)]	Defaults[2)]	Estimation Possibility[3)]	User Input[4)]
Average Annual Daily Traffic (AADT)	**-**	**-**	**No**	**No**	**Yes**
Facility Type	**-**	**-**	**No**	**No**	**Yes**
Hourly Volumes	X	X	No	Yes	Opt
Turn percentages	X	X	No	Yes	Opt
Number of Lanes	**X**	**X**	**No**	**No**	**Yes**
Adjusted saturation flow rate	X	X	Yes	Yes	Opt
Left turn phasing treatment	X	X	No	Yes	Opt
Cycle length	X	X	No	Yes	Opt
Lost time	X	X	Yes	Yes	Opt
Green times	X	X	No	Yes	Opt
Coordination settings	X	X	No	Yes	Opt
Peak hour factor	X	X	Yes	Yes	Opt
Parking	-	X	No	Yes	Opt
Area type	**X**	**X**	**No**	**No**	**Yes**
Speed Limit	X	-	Yes	No	Opt
Segment Length	X	-	No	No	Yes
Mid-segment volume	X	-	NO	Yes	Opt
Total delay due to turns into access points	X	-	Yes	No	Opt
Delay due to other mid-segment sources	X	-	Yes	No	Opt
Width of upstream boundary intersection	X	-	No	Yes	Opt
Length of segment with restrictive median	X	-	Yes	No	Opt
Length of segment with nonrestrictive median	X	-	Yes	No	Opt
Proportion of street with curb on right-hand side	X	-	Yes	No	Opt
Number of access points on right-hand side	X	-	Yes	No	Opt
Effective green-to-cycle-length ratio	X		No	Yes	Opt
Through-lane group volume	X	-	No	Yes	Opt
Proportion of arrivals during green	X	-	No	Yes	Opt
Base Free Flow Speed	X	-	Yes	Yes	Opt

1) An "X" Symbol signifies that this data item is needed for the respective analysis method (QEM-US or QEM-ST)
2) Default values for items with "yes" are developed and provided through this project
3) For items with "yes" this project provides a methodology for estimating the required input
4) Items with "yes" have to be entered by the users, items listed as "opt" are optional for data entry

User Cost Framework

For the user cost estimation, the team proposes to directly adopt the procedure developed for a prior NCDOT research project: NCDOT 2010-08: Corridor-Based Forecasts of Work-Zone Impacts for Freeways. That project resulted in an analysis methodology and computational engine for freeway work zones. The procedure for arterial work zones in this project is intended to be directly compatible to that prior work, and consequently the user cost performance measure estimation should be consistent. The following paragraphs summarize the existing user cost approach for those not familiar with the prior project.

The economic impacts of traffic congestion are directly related to the vehicle-hours of delay (VHD) on the facility, which is an output of the freeway method, and is expected to also be estimated in this project for an arterial streets. The VHD is estimated for each time period and is largely calculated by multiplying the difference of free-flow and congested travel time by the number of vehicles in a given segment.

Based on a review of literature on user-cost estimation, such as the "Red Book" (24), work zone analysis guidance used in the state of Texas (25), and benefit-cost analyses completed for the NCDOT (26), the team derived an approach for user cost estimation that is compatible with the HCM procedures, and which correlates with the literature.

Specifically, user cost is modeled as the summation of two independent values: (1) the Total User Delay Cost (UDC_{total}) and (2) Total Vehicle Operating Cost (VOC_{total}). The UDC is based primarily on the economic impacts from lost wages for the drivers (and passengers) delayed by the work zone. UDC further distinguishes between wages (hourly salary) of standard passenger cars and commercial vehicles. The second component, VOC is determined by estimating the economic impacts of the transported (truck) goods being delayed in traffic. It estimated from assumptions of the monetary value of the average truck load and the economic amortization cost of those goods while delayed in traffic. The VOC concept assumes that the operating agency of the truck has to take on a loan (at an average interest rate) to cover the value of the loaded goods for each hour that the truck is delayed in traffic. The detailed calculations are beyond the scope of this report, but can be referenced in the "Red Book" (24) or guidance for benefit-cost analysis in NC (26).

In the implementation in this method, UDC and VOC are calculated by multiplying the default UDC and VOC rates (per hour) by the vehicle-hours of delay (VHD) for each analysis time period, and summing over the entire analysis period. The proposed formulas for user cost modeling are given below:

$$UDC_{time\ period\#} = VHD_{time\ period\ \#} \times (P_c \times UDC_{cars} + P_t \times UDC_{trucks})$$

$$VOC_{time\ period\#} = VHD_{time\ period\ \#} \times (P_c \times VOC_{cars} + P_t \times VOC_{trucks})$$

$$UC_{time\ period\ \#} = UDC_{time\ period\#} + VOC_{time\ period\#}$$

$$UDC_{total} = \sum_{i=1}^{Number\ of\ Time\ Periods} UDC_i$$

$$VOC_{total} = \sum_{i=1}^{Number\ of\ Time\ Periods} VOC_i$$

$$UC_{total} = UDC_{total} + VOC_{total}$$

Where,

$UDC_{time\ period\#}$ = User Delay Cost at Specific Time Period

$VOC_{time\ period\#}$ = Vehicle Operating Cost at Specific Time Period

$TVHD_{time\ period\ \#}$ = Travel Vehicle Hours of Delay at Specific Time Period

P_c = Percent Cars

P_t = Percent Trucks

UDC_{cars} = User Delay Cost for Cars

UDC_{trucks} = User Delay Cost for Trucks

VOC_{cars} = Vehicle Operating Cost for Cars

VOC_{trucks} = Vehicle Operating Cost for Trucks

$UC_{time\ period\ \#}$ = User Cost at Specific Time Period

UDC_{total}= Total User Delay Cost over All Time Periods

VOC_{total} = Total Vehicle Operating Cost over All Time Periods

UC_{total}= Total User Cost over All Time Periods

To facilitate user input, NC defaults have been developed for UDC and VOC rates per hour for cars and trucks in project 2010-08, which are as follows.

UDC_{cars} = $21.07 per hour

UDC_{trucks} = $26.08 per hour

VOC_{cars} = $22.85 per hour (for FFS=65mph)

VOC_{trucks} = $154.73 per hour (for FFS=65mph)

The default values for User Delay Cost (UDC) for passenger cars and User Delay Cost for trucks are based on the recently completed *US 401 User Benefits Analysis* (26), which was assumed as an adequate example for NC. The default values for Vehicle Operation Costs (VOC) are based on estimates from the AASHTO "Red Book" (24) and a recent NCDOT Benefit Cost Report (26). They take into account the average fuel consumption of vehicles per minute of delay. If multiplied by the price of gas, this gives an estimate of the dollar cost of being delayed in traffic. The AASHTO "Red Book" then expresses the cost of fuel as a percentage of VOC. In other words, VOC can be estimated by dividing the cost of fuel by the percentage fuel cost of VOC.

Exhibit 18 gives guidance for how the VOC default values were estimated as a function of free-flow speed. The exhibit uses an estimate of fuel consumption per minute of delay from the literature (24), which is multiplied by 60 to get fuel consumption (in gallons) per hour of delay. That estimate is then multiplied by the assumed cost of fuel, and divided by the parameter for fuel cost as percent of VOC from (26). The resulting estimates in Exhibit 18 should be treated with care, as several assumptions tend to change quickly, due to changing economic conditions.

Exhibit 18: VOC Estimation Guidance (adapted from (24) and (26))

Free-Flow Speed	Fuel Consumption per Minute of Delay (34)		Fuel Consumption per Hour of Delay (gal)		Estimated VOC per Hour of Delay ($)	
	Car	Truck	Car	Truck	Car	Truck
20	0.022	0.102	1.32	6.12	$ 7.62	$ 35.31
25	0.026	0.133	1.56	7.98	$ 9.00	$ 46.04
30	0.03	0.167	1.8	10.02	$ 10.38	$ 57.81
35	0.034	0.203	2.04	12.18	$ 11.77	$ 70.27
40	0.038	0.241	2.28	14.46	$ 13.15	$ 83.42
45	0.043	0.28	2.58	16.8	$ 14.88	$ 96.92
50	0.048	0.321	2.88	19.26	$ 16.62	$ 111.12
55	0.054	0.362	3.24	21.72	$ 18.69	$ 125.31
60	0.06	0.404	3.6	24.24	$ 20.77	$ 139.85
65	0.066	0.447	3.96	26.82	$ 22.85	$ 154.73
70	0.073	0.49	4.38	29.4	$ 25.27	$ 169.62
75	0.08	0.534	4.8	32.04	$ 27.69	$ 184.85

Assumed Cost of Fuel ($/gal)	
Gas	$ 3.00
Diesel	$ 3.00

Fuel Cost as % of VOC
52%

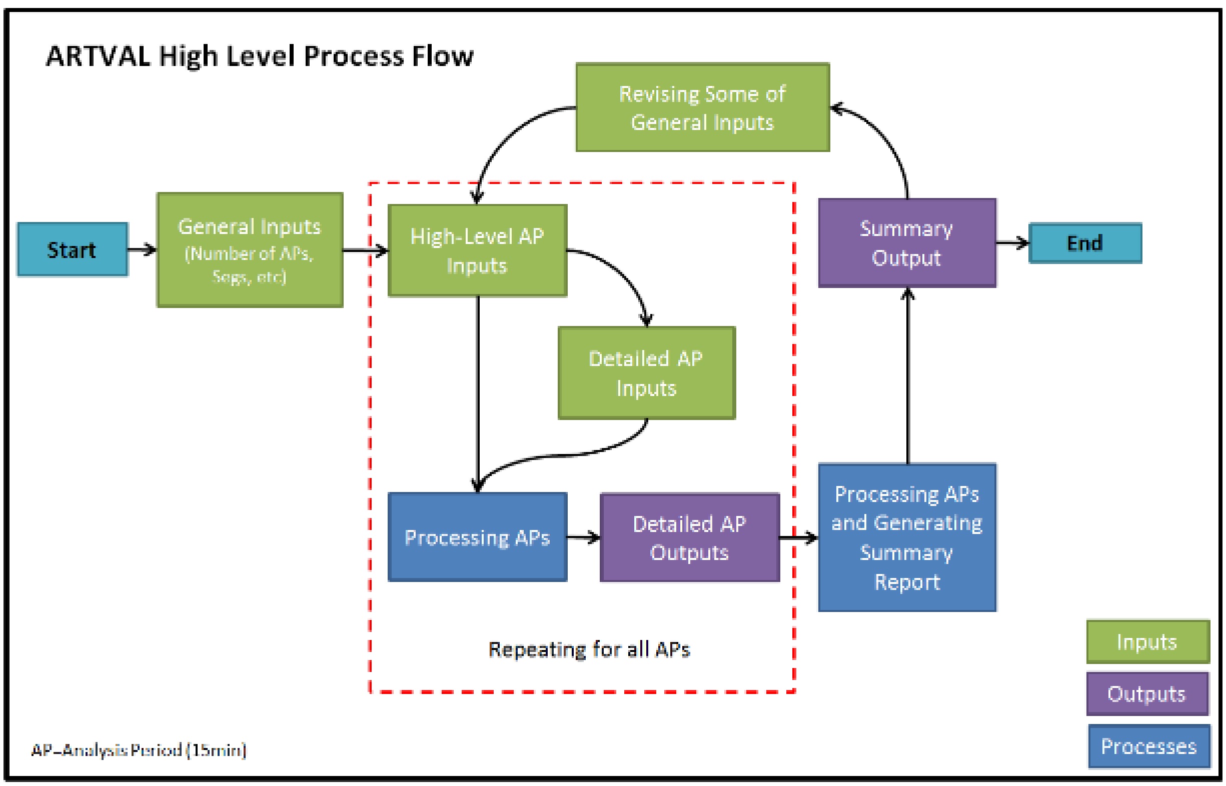
The team will provide defaults for UDC and VOC in the computational engine implementation of the procedure (for example for a free-flow speed of 45 mph). However, all inputs can be customized as necessary and should be changed in the future, as hourly wages, gas prices, and the value of goods increase.

Computational Engine Framework

As part of this project, the team was tasked with developing a computational engine that implements the methodology. The team proposes that this engine for **ART**erial e**VAL**uation (ARTVAL) be designed in a format consistent with the FREEway eVALuation tool (FREEVAL), already being used by NCDOT. The ARTVAL computational engine provides a computational resource compatible with the urban streets methodology in the HCM 2010. The work zone analysis on arterial streets would further be included in the ARTVAL engine, including mid-segment and intersection work zones. The framework comprises several steps to be carried out in a recursive format, which allows the analyst to come back to the beginning and re-run the computation based on new input data.

A high level process flow of the proposed ARTVAL tool is shown in Exhibit 19. The green and purple background boxes depict input and outputs in the ARTVAL process flow, respectively. Dark blue boxes describe the processes that are based on the urban streets methodology in the HCM 2010. The dashed rectangle represents the core analysis modules, which contain the detailed inputs and computations that are repeated for all analysis segments and time periods.

Exhibit 19: ARTVAL high-level process flow



**AP = Analysis period, which is the smallest temporal unit of analysis, is 15 min in duration*

Fundamentally, the ARTVAL process flow is constructed by three basic units, namely “Input”, “Process”, and “Output”. These are color-coded in the exhibits in green, purple, and blue, respectively. The following sections describe these units in more detail.

The Input Unit

This unit in ARTVAL is responsible for collecting all necessary information for computing the defined mobility performance measures for the arterial street. These units are shown in the Process Flow Diagram by green shaded boxes.

The first category of input units captures the *general inputs.* These are general arterial facility information items, such as the number of 15-minute analysis periods considered, the number of segments, and the number intersections. Based on these input data, the appearance of the computational engine will change to account for all the specified information. For example, for each 15-minute analysis period a separate spreadsheet is generated for data entry.

The second category of inputs consists of more specific inputs for each segment and analysis period regarding the segment geometry, traffic demand, work zone configuration, etc. This information should be provided for each 15-minute analysis periods. In ARTVAL, the second category is divided into two sub-categories, named “High Level AP Inputs”, and “Detailed AP Inputs” input.

The high-level analysis input provides default values and configuration for the analyst to be used in a planning-level context, or in the absence of access to detailed information. The engine provides the user with a set of pre-specified facility types and associates a set of default inputs with each. Here, the team relied heavily on recently-developed defaults for a separate NCDOT research project (Project 2012-05) on updating the NCLOS planning-level analysis tool. The specific defaults were coordinated with the Steering and Implementation Committee for this project, and can be customized and overridden should facility-specific detailed data be available. But the vision behind the defaults is to allow for a quick and high-level analysis of the arterial facility and work zone impacts without detailed information on signal timing and other parameters.

If more detailed data are available, the analyst can provide facility-specific data in the “Detail AP Input”, and therefore may override the default values in the high level AP data entry. The concept here is that for high-level planning analyses, the user may decide to skip the detailed AP Input step, and quickly generate results from the high-level AP input only. But if desired, the detailed AP input will be available, enabling customization of any facility-specific input values into the methodology. The detailed input then allows specification of signal timing parameters, platoon ratios, and other detailed parameters. The user can decide to override one, multiple, or all defaults depending on the availability of data.

The Processing Unit

The first processing unit, *Processing APs* in Exhibit 19 is executed for all analysis periods. Here input data are transferred to the computational engine procedure to compute and generate the key performance measures of the urban street facility for each analysis period. The unit produces results for each *node* and each *segment,* where a segment represents the stretch of roadway between two intersection nodes. Each node and segment is represented by a separate column of inputs, and accordingly generates a separate column of outputs. Performance measures for each node contain common metrics of intersections, including delay, queue lengths, and volume-to-capacity ratio, while performance measures at the segment level include travel time, and the average running speed. In addition, the performance measures are aggregated to average facility speed and travel times, along with system performance measures such as Vehicle Miles of Travel (VMT), Vehicle Hours of Delay (VHD), etc. This unit is invoked for all analysis periods defined on the arterial facility.

The second processing unit generates facility and study period output summaries. It uses the outputs of the "Processing APs" unit and summarizes them in the "Summary Output". Within the "Processing APs" unit, the engine invokes procedures for facility analysis. The Processing APs unit represents the heart of the computational engine.

The Output Unit

This unit consists of two elements. The first, "Detailed AP Output", includes the detailed analysis output for each 15-minute analysis period. The first section of the processing unit (Processing APs) generates all entries for this category. The second category is a summary of the performance of the facility across the study period named "Summary Output". The detailed AP output is used to show the facility level output summary. The summary output contains customized reports, summary tables, and graphs that illustrate the performance of the arterial street.

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4. Methodology

This chapter presents the criteria used to select study sites and the plan established for collecting data needed to quantify the effort of various factors on saturation flow rate. The first part of this chapter describes the site selection criteria and data collection plan. The study locations presented in this project consist of data collection from six arterial intersections work zone sites in three municipalities in North Carolina. The data collection plan was coordinated with the Literature Review and the Analysis Framework as described earlier in this report. Consistent with the Analysis Framework, the collected data during work zone activity were used to calibrate and validate saturation flow rate models at intersections, and travel time /delay models at the arterial level. The last part of this chapter describes the data reduction and analysis procedures.

Data Collection Approach

Site Selection Criteria

Site selection criteria were developed based on findings from the literature review, and were refined by the pilot data collection. Adequate data collection is essential to develop and validate saturation flow rate (queue discharge) models at intersection and mid-segment work zones, as well as validating the arterial travel time and delay models. The data collection sites for this project should meet the following criteria:

For both intersection and mid-segment locations, the sites should:

- Have permitted, safe, and easily accessed locations to mount the data collection devices, such as video cameras and Bluetooth units. The video cameras need to be installed at a relatively high position (e.g. signal or utility poles) while Bluetooth devices do not need a lot of height.;
- Have high vehicular volumes (around 400 veh/h/ln during peaks) for an efficient and valid saturation flow rate study;
- Have short-term work zones to enable the team collect before/during or during/after data;
- Ideally have different scenarios that can be observed within the same data collection set-up, such as various lane closure and pavement condition configurations within a one-week period;
- Be located on relatively flat grade to avoid confounding effects of steep grade.

For intersection work zone modeling, the sites should further:

- Be located at or near an intersection along the arterial;
- Have no or very minor queue spill back from other bottlenecks;
- Have exclusive left turn lanes (or very low left turn volume in shared lane); and
- Have exclusive right turn lanes (or very low right turn volume in shared lane).

For mid-segment work zone modeling, sites should further:

- Be located more than 1,000 feet from the intersection of urban arterial (or a longer distance to assure that the work zone will not be affected by the back of the queue from nearby intersections);

- Avoid short blocks like downtown areas; and
- Have limited number of driveways and roadside business access points..

Further criteria for site selection are as follows:

- Sites should be from different cities across the state;
- Sites cover a variety of work zone types; and
- Sites should provide data on important factors (as listed in the literature review and analysis framework sections) in work zone modeling.

The study sites for this project are located in Raleigh, Durham, and Clayton in North Carolina. A Summary table of work zones for which data collection were completed and used in this project provided in Exhibit 20.

Exhibit 20: Summary table of work zones

Study Location	Site Type	Num. Lanes (2 Dir.)	Work Zone Type	Work Zone Extents	Municipality	Comments
Wade Ave EB at Faircloth St	Intersection	4 (Median Divided)	Pavement Resurfacing	Wade Ave (Faircloth St to Ridge Rd)	Raleigh	Various lane closures, milled surfaces, ledge
Wade Ave WB at Faircloth St	Intersection					
Lake Wheeler Rd SB at I-40	Intersection	4 (Median Divided)	Sidewalk Construction SB	Lake Wheeler (Centennial Pkwy to Tryon Rd)	Raleigh	SB right lane closed with exclusive right turn lane
MLK Blvd EB at Hope Valley Rd	Intersection	4 (Median Divided)	Pavement Resurfacing	MLK Blvd (NC 55 to Archdale)	Durham	Various lane closures, milled surfaces, ledge
MLK Blvd EB at Roxboro St	Intersection					
MLK Blvd WB at Roxboro St	Intersection					
US 70 Bus EB at Shotwell	Intersection	5 lane 5 lane (Median Divided)	Pavement coring to remove sensors embedded in pavement at each intersection	US 70 Bus (Shotwell to S Robertson)	Clayton	Various changing lane closures to remove sensors from every travel lane in both directions

Data Collection for Saturation Flow Rate Analysis

The primary data collection focus of this project is to support saturation flow modeling (queue discharge modeling) at intersection and mid-segment work zones, as well as to validate the travel time or delay models at the arterial level. The team relied on video-based data collection for the saturation flow rate studies, while using Bluetooth units to estimate arterial travel time impacts of the work zone. These two data collection activities are detailed below.

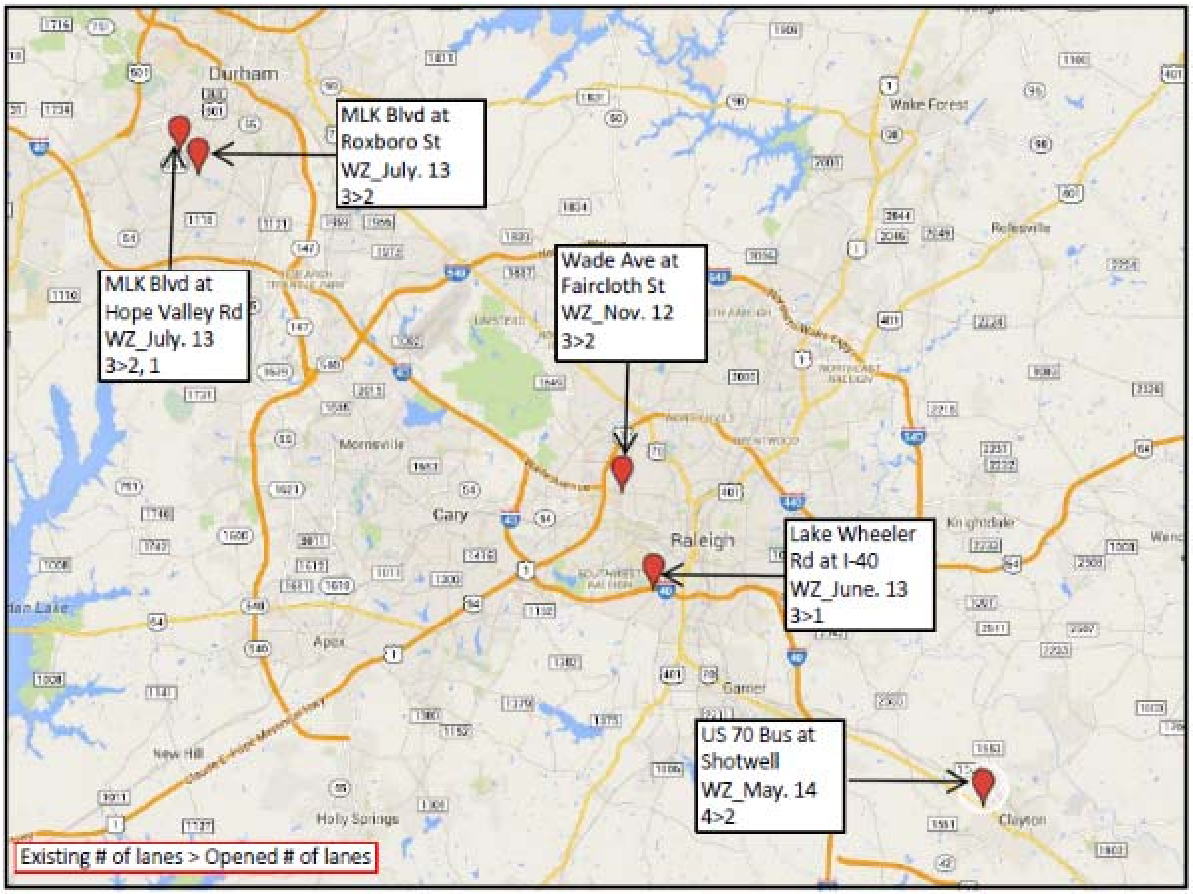
Video-based observation was proposed as the primary tool for saturation flow rate data collection at intersection work zones. Two or more video cameras are typically needed to videotape all four approaches (one camera may cover up to two approaches). Installation instructions have been documented in the Wade Avenue pilot data collection chapter in the next section. The following information is essential to the intersection saturation flow rate model, and needs to be captured:

- Work zone configuration (adjacent activities, lane closure, milled surface, pavement ledge, etc.);
- Work zone schedule, detailed location, number of type of machines;
- Approach, intersection, road, city, county, state identity information;
- Video camera location map, including the camera ID and angle;
- Date, time and weather;
- Geometric and traffic flow factors that would affect saturation flow rate (as documented in the HCM);
- Signal control characteristics (cycle length, phase, green and amber time, etc.);
- Flagger control details (like signal control), if present;
- One to two week video recording before / during or during / after work zone are necessary; a minimum of 30 observations is required for each condition (i.e. work zone type, scenario, site, etc.);

The team considers video data as the most reliable data. Prior research efforts for NCDOT have tested the use of Roadside Traffic Management Sensors (RTMS), but concluded that these units are not reliable and useful for this type of data collection on arterial streets.

Additionally non-work-zone data were also collected after the construction activities were completed. These data used to estimate the incremental impacts of the work zone for different facilities. Data were collected during time periods that are reflective of typical peak traffic periods at each study sites which typically occurred during midweek days. Data were not collected during holidays, incidents, or periods of inclement weather. Exhibit 21 illustrates the geographic location of the data collection sites.

Exhibit 21: Locations of Work Zones



Data Collection Device Installation

Video cameras were installed to capture saturation flow rate data at the study intersections. A small video camera could be attached to a pole or tree by one person using a 12-20 feet ladder. Another person is needed as “spotter” to stabilize the ladder, and assist in fine-tuning the camera angle using a small monitor. The camera is securely attached to the pole with a custom strap and mounting system, and the wire is secured to the pole using tape. The batteries are stored in a weatherproof and lockable box at the base of the pole.

For the first pilot data collection, four video cameras were installed at the Wade Avenue work zone in less than two hours. Approximately, it took ½ hour for two skilled people to install each camera. The locations of video cameras and images of the actual installation are shown in Exhibit 22. Other data collection sites were set up with similar video installations.

Exhibit 22: Video Camera Location and Configuration

Pilot Data Collection

To test the data collection plan, pilot data were collected for an arterial work zone in Raleigh NC during one week in the Fall of 2012 The data collection activities included setting up overhead video cameras to capture saturation flow rate under several work zone configurations: upstream lane closure with adjacent work activities, vehicle running on milled surface or across ledge, and left turns with outside lane closure. Further, Bluetooth devices were installed to measure the travel time during work zone activities. This section presents preliminary analysis results. Further data processing for a larger sample size is recommended for later tasks of this project.

For the pilot data collection, critical work zone activity patterns were extracted from video as shown in Exhibit 23. Note that some other activities may not be visible from video and thus are not listed. The video data extraction efforts were coordinated with these activities, and specific samples obtained for several of the observed conditions.

Exhibit 23: Work Zone Activity Log and Data Extraction for Pilot Study

Day	Intersection*	Activity	Data Collection (Y/N)	Scenario Collected	Sample Size (Cycles)
10/31/2012 Wed	**W-F**	**EB/WB Approach Milled**	**Y**	**Through across Ledge**	**15**
	W-R	**EB Right Ln Half Milled**	**Y**	**Left Turns Before/After Right Ln Closure**	**60**
11/01/2012 Thu	**W-F**	**Remain Milled**	**Y**	**Through on Milled Surface**	**15**
	W-R	EB Upstream Right and Downstream Left Temporary Ln Closure/ Remain Half Milled	N	n/a	
11/02/2012 Fri	W-F	Remain Milled	N	n/a	
	W-R	EB Upstream Right and Downstream Left Temporary Ln Closure/ Remain Half Milled	N	n/a	
11/05/2012 Mon - 11/07/2012 Wed	W-F	Remain Milled	N	n/a	
	W-R	EB Remain Half Milled	N	n/a	
11/08/2012 Thu	**W-F**	**EB/WB Paved**	**Y**	**Through with Upstream Lane Closure and Adjacent Work Activities**	**15**
	W-R	EB Paved	N	n/a	

* 'W-F' indicates the intersection of Wade Avenue and Faircloth St; 'W-R' Wade Avenue and Ridge Rd.

The listing in Exhibit 23 makes evident that a very rich dataset can feasibly be obtained from a single site installation. In particular, the team was able to extract 105 cycles of data in four different work zone configurations, with an additional six different work zone conditions available on the videos.

Site Description

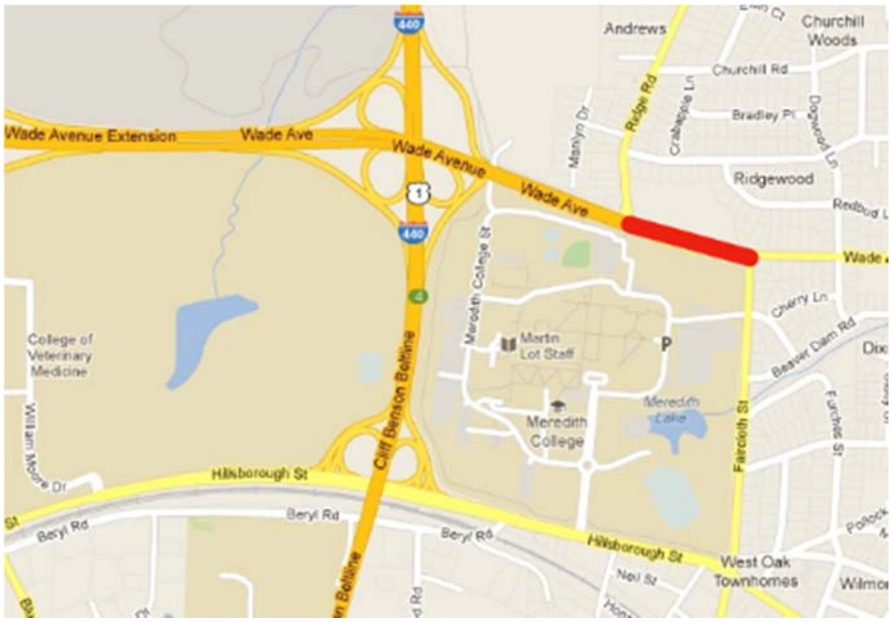
This section describes the data collection sites used in this project.

Wade Ave

For the pilot data collection site, the team selected a section of Wade Avenue from Faircloth Street to Ridge Road, in Raleigh, NC. This section has a total of four lanes in both direction and an AADT of around

31,000 vehicles. It also features heavy commuter traffic volume during peak hours. In this work zone, resurfacing activities started at 9AM on Wednesday 10/31/2012 and lasted for around two weeks. The contractor was only allowed to work during 9AM to 4PM on weekdays. A map of the work zone is shown in Exhibit 24.

Exhibit 24: Wade Avenue Work Zone Location



During the data collection period, resurfacing activities were mainly focused on westbound Wade Avenue and the intersection of Wade Avenue and Faircloth Street. Minor work activities existed on the eastbound shoulder and the outside lane. Generally, contractors controlled traffic using flaggers during the milling operations at the intersection. But the milled surfaces were left overnight for several days before they were repaved, allowing the team to collect data for travel on milled surface. The following sections document the installation of data collection devices, travel time and saturation flow rate study results, and a summary of lessons learned from the pilot study.

Lake Wheeler Road at I 40.

The next site chosen for this study was located on Lake Wheeler Road in the city of Raleigh, NC. The northbound approach has two through lanes and one exclusive right lane as seen in aerial view in Exhibit 25. The camera was installed during work zone in spring 2013 and captured the traffic movements upstream and downstream of the intersection. The work zone was categorized as medium intensity while the exclusive right lane were closed and the one through lane were used for both through and right turning movement. In order to compare the effect of work zone to non-work zone conditions, the cameras were reinstalled after work zone completed in the summer of 2013.

Exhibit 25: Lake Wheeler Road Data Collection Site

Martin Luther King Jr Blvd

Data were collected during and after workzone for two intersections located on Martin Luther King Jr Blvd in Durham, NC. Video data during the work zone was collect at the intersections of MLK and Roxboro St EB, MLK and Roxboro St WB, and MLK and Hope Valley Rd for the project. Video was collected in two rounds for this site, resulting in approximately 14 days of video data from summer 2013. In this study, the focus was on the eastbound approach of the intersection on MLK and Hope Valley Rd. The eastbound approach has one left, one through, and one shared through and right lanes. A short-term work zone was active during the day and the construction equipment was far from the intersection with minimum work intensity. Data were collected when right lane or shared through/right lane were alternatively closed during the work zone.

The study site at intersection of MLK and Roxboro were evaluated for both eastbound and westbound approaches. The eastbound approach of this intersection has one exclusive left, one through, and one shared through/right lanes. Data were collected during none to high work intensity while construction equipment was being operated near the intersection and along the arterial. The data collection included periods when either one or two lanes were closed. This site did not have any pavement ledge and data were collected when pavement surface was milled and smooth.

Similarly, the westbound approach of this intersection had same lanes configuration with same level of work intensity. This approach had a pavement ledge and the pavement surface was in smooth condition during the data collection. The two sites are shown in Exhibit 26 and Exhibit 27.

Exhibit 26: Martin Luther King Jr Pkwy & Hope Valley Rd

Exhibit 27: Martin Luther King Jr Pkwy & S Roxboro St

In order to compare the effect of work zone to non-work zone conditions, the cameras were reinstalled after work zone completed and data were collected without work zones in late 2013.

US 70 Business at Shotwell Rd.

The last study site was located in Clayton, NC. This intersection is located at the intersection of US70 Business and Shotwell road, and the study focused on the eastbound approach. This approach has one left, three through, and one exclusive right lane. The work zone involved removing sensors imbedded in the pavement from the US 70 travel lanes and required lane closure. The data were collected during and before and after work zone in the spring of 2014. The site is shown in Exhibit 28.

Exhibit 28: US 70 Business at Shotwell Rd

Data Reduction and Analysis

The method of data collection was adapted from Chapter 6, Intersection and Driveway Studies of the ITE Manual of Transportation Studies (MTES, ITE 2010). The estimated saturation flow rate from field data is defined as the number of vehicles that pass the stop bar at the intersection with no interruptions in a defined period of time. The MTES describes a method for gathering saturation flow rate observations. In this method, a timer is started when the fourth vehicle in a queue passes the stop bar, as this is the point when the queue will usually start keeping consistent headways, and is after any start-up lost time is incurred. The timer is stopped when either the seventh, eighth, ninth, or tenth vehicle passes the stop bar, whichever was the last in the stopped queue. With a typical standard deviation in saturation flow rate of 140 vehicles per hour, the MTES recommends observing a minimum of 30 valid queues to estimate the mean saturation flow rate within 50 vehicles per hour of the true rate with 95% confidence (ITE, 2010). In other words, if the field estimation shows an average saturation flow rate of 1,600 vehicles per hour per lane (vph/ln), the 95% confidence interval ranges from 1,550 to 1,650 vph/ln.

Saturation Flow Rate Study Results

Saturation flow rate data were collected under the following conditions: (1) upstream lane closure with adjacent work activities, (2) travel across milled surface, (3) travel across a ledge resulting from milling, and (4) right most (non-adjacent) lane closure for left turn. Video screenshots of all four conditions for the Wade Avenue site are shown Exhibit 29 through Exhibit 32 below.

Exhibit 29: Through Movement with Upstream Lane Closure and Adjacent Work Activity

Exhibit 30: Through Movement on Milled Surface

Exhibit 31: Through Movement across Ledge

Exhibit 32: Left Turn Movement with Rightmost (non-adjacent) Lane Closure

The saturation flow rates under these work zone scenarios are expected to be different from each other and lower than the saturation flow rate under the normal or ideal conditions. The saturation flow rate can be estimated from field data using the following formula:

$$\boldsymbol{SF = \frac{3600\,n}{\frac{a}{3}+\frac{b}{4}+\frac{c}{5}+\frac{d}{6}}} \qquad \textbf{Equation 10}$$

where,

SF = mean saturation flow rate (veh/h/ln),

n = total number of queues observed, and

a, b, c, d = sum of the time taken between the 4th vehicle and the 7th, 8th, 9th and 10th vehicles to cross the stop bar over all the observations, respectively.

The results of the saturation flow rate study are detailed in Exhibit 33. The existing work zone saturation flow rate models developed by J. Bonneson and K. Nguyen (2012) and proposed for the HCM 2010 are also shown as a point of comparison to the field estimates from pilot Wade Avenue site.

Exhibit 33: Saturation Flow Rate from Field Data and Models

Movement	Scenario	Saturation Flow Rate (vphpln)	
		Field	HCM
Left	Before Outside Lane Closure	1900	1805
	After Outside Lane Closure	1570	1481
Through	With Upstream Lane Closure and Adjacent Work Activities	1200	1682
	On Milled Surface	1714	1889
	Across Ledge	1440	1889

The pilot data suggests that the saturation flow rates under the tested work zone configurations are different and significantly lower than that during normal conditions (approximately 1,800-1,900 vph/ln). The HCM work zone model proposed by Bonneson and Nguyen (2012) may estimate saturation flow rates with bias when vehicles are running under homogeneous and constant conditions, such as nearby lane closure or on concessive milled surface. These added friction factors, other than lane width and number of lane drops, are not included in the model. As a result, these initial results suggest that pavement conditions (milled pavement and ledge), as well as adjacent work activity, may be important factors to consider in future work zone saturation flow rate models.

Effective Green Time Study Results

A separate study was conducted from video at the Wade Avenue pilot site to investigate the effective green time and cycle length under flagger control at work zones. The results in Exhibit 34 reveal that the cycle length under flagger control is almost twice as much as that under regular signal control. The same is true with the green time. This suggests that estimates of signal timing parameters need to be carefully evaluated under flagger control.

Exhibit 34: Effective Green Time and Cycle Lengths under Flagger Control

Situation	Effective Green Time (sec)	Cycle Length (sec)
On Milled Surface (Signal)	72.2	141.2
Over Ledge (signal)	82.5	155.0
With Adjacent Activity (Flagger)	135.1	235.8

In addition to vehicle's headway and time stamp for all coded vehicles, right turn and left turn vehicle percentage, before and after lane width, heavy vehicle percentage, number of lanes, open lane(s) configurations, closed lane(s) configurations, right turn angle shape, equipment location, work zone work intensity, work activity location, pavement type, and ledge data are also coded into a database.

Data Summary

The objective for each sites was to record a minimum of 30 cycle during the study for non-work zone and during work zone activity. At all sites, the camera were maintained for several months and the average headway data was generated for total of 795 cycles for non-work zone and 677 cycles work zone . The reduced data set does not include any heavy vehicle's headway data, which means that data set consist of pure passenger vehicles. In this section, a summary of data results per side are presented. The data are presented in terms of the average saturation headway in seconds, which is related to the inverse of saturation flow rate. For comparison, a saturation flow rate of 1,800 passenger cars per hour per lane (pc/h/ln) corresponds to a headway of 2.0 seconds. Headways less than 2.0 seconds will correspond to higher saturation flow rates, while longer headways result in reduced saturation flow rates.

Wade Ave EB at Faircloth St

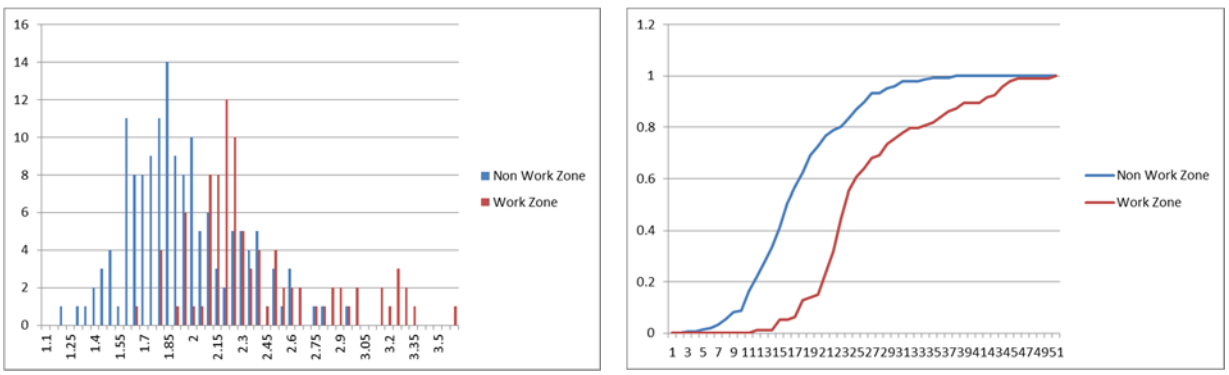
Exhibit 35 shows the data summary for the Wade Ave EB at Faircloth St site. 146 and 94 cycles of average vehicle headway data were collected for non-work zone and work zone conditions, respectively. The summary table shows that average headway under non-work zone condition is 1.90 seconds while 2.36 seconds for work zone condition.

Exhibit 35: Summary of Wade Ave EB at Faircloth St Vehicle Headway Data

Non Work Zone			Work Zone		
Sample Size	Average (pc/h/ln)	SD (pc/h/ln)	Sample Size	Average (pc/h/ln)	SD (pc/h/ln)
146	1.90	0.320	94	2.36	0.412

Exhibit 36 shows the comparison of the headway distribution with and without the work zone condition. Both the probability density function (PDF) and the cumulative distribution function (CDF) give strong evidence of increasing average headways under work zone condition compared to non-work zone.

Exhibit 36: Wade Ave EB at Faircloth St before and after Headway Comparison



(a) PDF of Cycle Average of Headway *(b) CDF of Cycle Average of Headway*

Wade Ave WB at Faircloth St

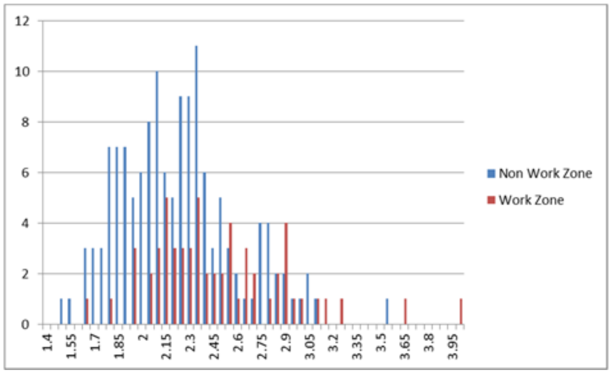
Exhibit 37 shows the data summary for the Wade Ave WB at Faircloth St Site. 140 and 59 cycles of average vehicle headway data were collected for during non-work zone and work zone condition, respectively. The summary table shows the average headway under non-work zone condition is 2.20 seconds and 2.46 seconds for work zone condition.

Exhibit 37: Summary of Wade Ave WB at Faircloth St Vehicle Headway Data

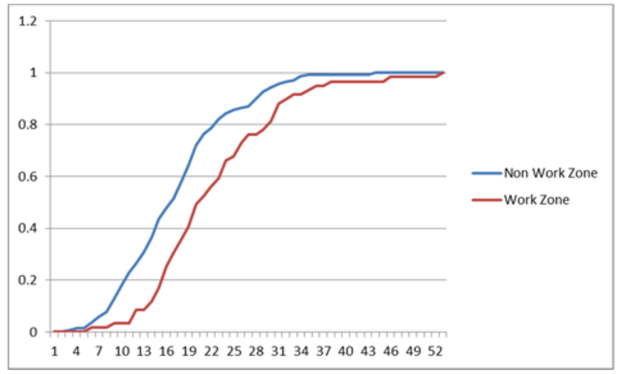
Non Work Zone			Work Zone		
Sample Size	Average (pc/h/ln)	SD (pc/h/ln)	Sample Size	Average (pc/h/ln)	SD (pc/h/ln)
140	2.20	0.365	59	2.46	0.427

Exhibit 38 shows the comparison of headway distributions with and without the work zone. Both PDF and CDF plots give evidence of increasing average headway under work zone condition compared to non-work zone.

Exhibit 38: Wade Ave WB at Faircloth St before and after Headway Comparison



(a) PDF of Cycle Average of Headway



(b) CDF of Cycle Average of Headway

Lake Wheeler Rd SB at I-40

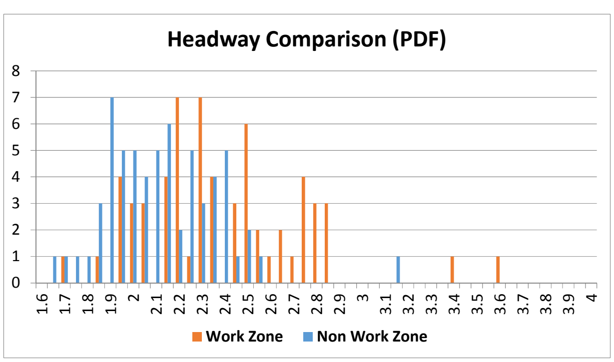
Exhibit 39 shows the data summary for the Lake Wheeler Rd SB at I-40 site. 63 and 62 cycles of average vehicle headway data were collected for during non-work zone condition and work zone condition, respectively. Exhibit 38 shows that the average headway under non-work zone condition is 2.10 seconds and 2.37 seconds for work zone condition.

Exhibit 39: Summary of Lake Wheeler Rd SB at I-40 Vehicle Headway Data

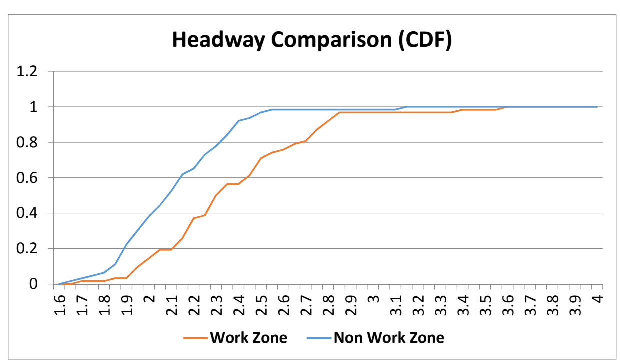
Non Work Zone			Work Zone		
Sample Size	Average (pc/h/ln)	SD (pc/h/ln)	Sample Size	Average (pc/h/ln)	SD (pc/h/ln)
63	2.10	0.248	62	2.37	0.3456

The PDF and CDP plots are presented in Exhibit 40 and again show strong evidence of increased headways during work zone conditions.

Exhibit 40: Lake Wheeler Rd SB at I-40 before and after Headway Comparison



(a) PDF of Cycle Average of Headway



(b) CDF of Cycle Average of Headway

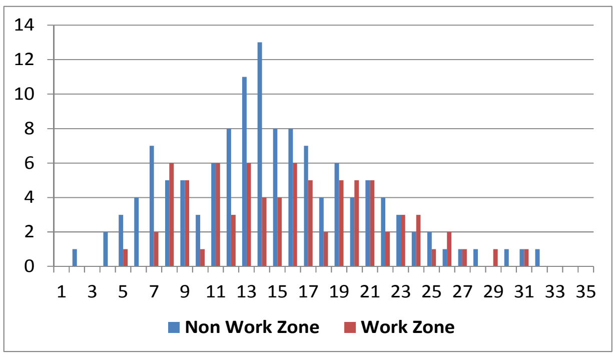
MLK Blvd EB at Hope Valley Rd

A total of 207 cycles of average headway data were collected at the MLK Blvd EB at Hope Valley Rd site. Exhibit 41 shows the summary results and Exhibit 42 shows the comparison of headway distribution of with and without the work zone. The PDF and CDF plots show an increase of headways under work zone condition but the average difference is only 0.07 seconds.

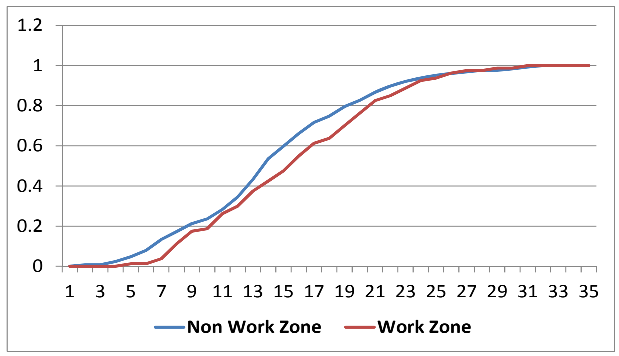
Exhibit 41: Summary of MLK Blvd EB at Hope Valley Rd Vehicle Headway Data

Non Work Zone			Work Zone		
Sample Size	Average (pc/h/ln)	SD (pc/h/ln)	Sample Size	Average (pc/h/ln)	SD (pc/h/ln)
127	2.16	0.303	80	2.23	0.290

Exhibit 42: MLK Blvd EB at Hope Valley Rd before and after Headway Comparison



(a) PDF of Cycle Average of Headway



(b) CDF of Cycle Average of Headway

MLK Blvd EB at Roxboro St

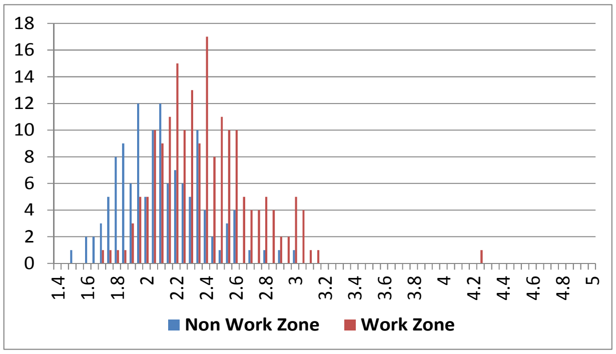
Exhibit 43 shows the data summary for the MLK Blvd EB at Roxboro St site. 127 and 188 cycles of average vehicle headway data were collected for during non-work zone and work zone conditions, respectively. The summary table shows that the average headway under non-work zone condition is 2.07 seconds, while the average is 2.38 seconds for the work zone condition.

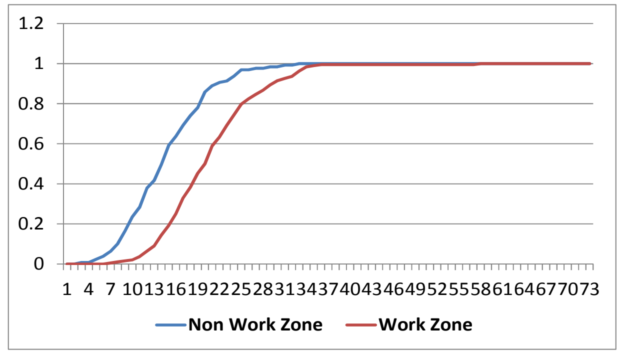
Exhibit 43: Summary of MLK Blvd EB at Roxboro St Vehicle Headway Data

Non Work Zone			Work Zone		
Sample Size	Average (pc/h/ln)	SD (pc/h/ln)	Sample Size	Average (pc/h/ln)	SD (pc/h/ln)
127	2.07	0.277	188	2.38	0.331

Exhibit 44 shows the comparison of headway distribution with and without the work zone. Both PDF and CDF plots show clear increases in average headway with the work zone in plce.

Exhibit 44: MLK Blvd EB at Roxboro St before and after Headway Comparison





(a) PDF of Cycle Average of Headway *(b) CDF of Cycle Average of Headway*

MLK Blvd WB at Roxboro St

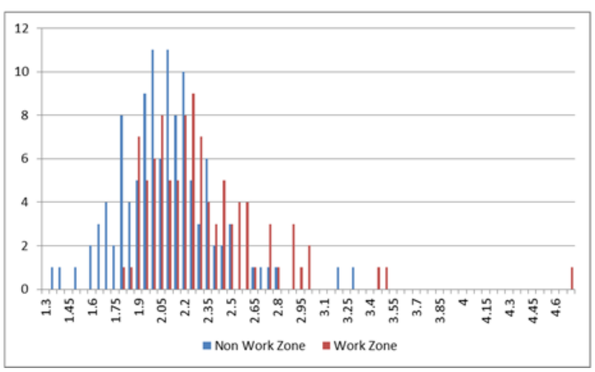
Exhibit 45 shows the data summary for the MLK Blvd WB at Roxboro St site. At the site, 113 and 99 cycles of average vehicle headway data were collected during non-work zone and work zone conditions, respectively. The summary table shows that the average headway under non-work zone condition of 2.06 seconds is increased to 2.30 seconds for the work zone condition.

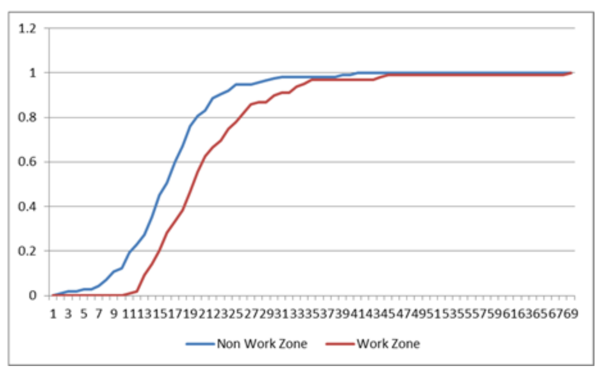
Exhibit 45: Summary of MLK Blvd WB at Roxboro St Vehicle Headway Data

Non Work Zone			Work Zone		
Sample Size	Average (pc/h/ln)	SD (pc/h/ln)	Sample Size	Average (pc/h/ln)	SD (pc/h/ln)
113	2.06	0.307	99	2.30	0.407

Exhibit 46 shows the comparison of headway distribution with and without work zone. Similar to other sites, both PDF and CDF show a clear increase in headways with the work zone in place.

Exhibit 46: MLK Blvd EWB at Roxboro St before and after Headway Comparison





(a) PDF of Cycle Average of Headway *(b) CDF of Cycle Average of Headway*

US 70 Bus EB at Shotwell St.

Exhibit 47 gives a data summary for the US 70 Bus EB at Shotwell St site. The team collected 99 and 29 cycles of average vehicle headway data for during non-work zone and work zone condition, respectively.

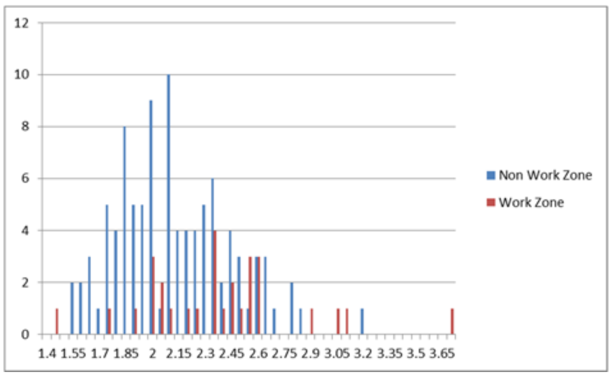
The summary table shows that the average headway under non-work zone condition is 2.11 seconds and 2.37 seconds for the work zone condition.

Exhibit 47: Summary of MLK Blvd WB at Roxboro St Vehicle Headway Data

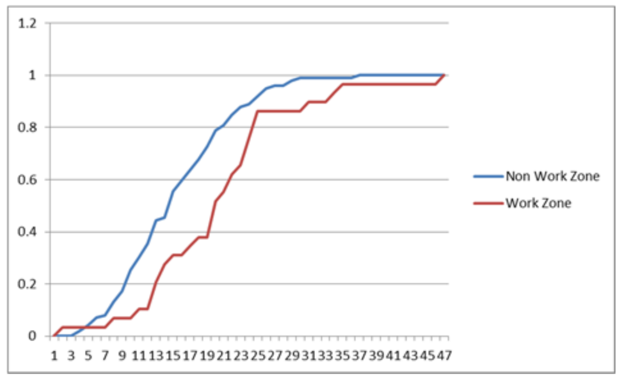
Non Work Zone			Work Zone		
Sample Size	Average (pc/h/ln)	SD (pc/h/ln)	Sample Size	Average (pc/h/ln)	SD (pc/h/ln)
99	2.11	0.325	29	2.37	0.437

Exhibit 48 shows the comparison of headway distribution with and without work zone condition, showing a clear increase in headways with the work zone in place.

Exhibit 48: US 70 Bus EB at Shotwell St before and after Headway Comparison



(a) PDF of Cycle Average of Headway



(b) CDF of Cycle Average of Headway

5. Modeling Results

This chapter presents the arterial work zone saturation headway estimation method and modeling results. The models are sensitive to work zone configuration attributes, including as lane width, number of lane close, pavement condition and etc. These work zone configurations are expected to affect the saturation headways of vehicles, and are treated as potential independent variables in a headway estimation model for work zones. In addition, this section introduces several default values and estimation procedures that will be used to simplify user input in the overall methodology, as well as the ARTVAL computation engine.

Arterial Work Zone Saturation Headway Model

Work zone and construction impacts on arterial streets can constitute a considerable source of congestion in urban environments. The saturation headway is a key attribute of intersection capacity, being inversely related to the saturation flow rate that is used to predict lane group capacity in an analytical analysis approach, like the Highway Capacity Manual (HCM).

The non-work-zone saturation flow rate in the HCM is a function of a variety of factors including approach grade, heavy vehicles, turning geometry, and pedestrian, bicycle, and transit activity. For work zones, the saturation headway is intuitively associated with lane closure conditions, pavement conditions, work activity, and other attributes of the work zone, which can all be associated with a capacity reduction. This capacity drop contributes to the severity of congestion and possibly to roadway safety, but guidance is presently limited on how to predict this capacity as a function work zone attributes.

The analysis of the operational impacts of work zones is important to estimate travel time, delay, and user cost borne by motorists and the community due to construction activities. Therefore, finding the best way to operate arterial work zones provides a safer condition for drivers and improves mobility. Due to the current increase in work zone activities in general and at intersection in particular, it is desirable to develop a model that can estimate the impact of work zones on urban arterial streets.

This section evaluates factors that are believed to influence the saturation flow rate at signalized intersections, and provides an empirical model to estimate the saturation headway for arterial work zones based on a set of independent variables. The proposed model helps analysts understand the factors contributing to capacity reductions on urban arterial streets due to work zones, and further can be used as guidance to calibrate simulation tools for proper representation of the resulting capacity impacts. In this project, the models form the basis of the work zone assessment modules in the ARTVAL computational engine.

Methodology

The objective of this section is to develop statistical regression models to estimate saturation headway based on different arterial work zone configurations. The project team first went through a rigorous process to identify different construction efforts in North Carolina, by contacting DOT district engineers, as well as municipal staff in major metropolitan areas. The project team was specifically focused on short-term duration work zones, to allow of data collection during construction, as well as after completion to allow a comparison to non-work zone conditions. The team also identified the main factors that were expected to influence saturation headway on arterial work zones, and targeted site identification and data collection to obtain a broad spectrum of observations. Data was collected and reduced as will be discussed later.

Then, independent variables are evaluated for linear correlations to ensure independence between the variables in the model. In the next step, conventional multi-linear and path analysis regression approaches are used to fit statistical models to the data. Both automatic variable selection approaches and manual variable selection informed by engineering judgment are utilized to finalize the regression models. In the remainder of this section, some detailed information on multiple linear regression, path analysis regression, and data collection and reduction approaches are presented.

Field Data Collection and Reduction

Data collection involved deployment of overhead video cameras that provided a view of queue discharge conditions at the stop bar, as well as a view of upstream conditions to confirm queued conditions. Video recordings were performed for multiple days at most locations, and the video data then separated into different phases of the construction (for example adjacent lane closure, followed by milling, followed by travel on milled surface without lane closure, etc.).

Data collection is performed when the work zone is present, as well as either before or after the work zone. This enables developing models that are anchored in the non-work zone base saturation headway and offer relative adjustments based on work zone configurations and other prevailing conditions. The data is collected using overhead video at six intersections (seven intersection approaches) in the state of North Carolina, with each offering multiple work zone configurations (total of 19 unique configurations) during the construction sequence. The video data allow measuring saturation headway, as well as identifying the type of work zone activity, and its configuration over time.

Bonneson and Nguyen (4) considered AADT, work zone timing, work zone duration, and number of closed lanes as their criteria to select suitable work zone sites. A volume criterion was established as a minimum AADT of 3,550 veh/day/lane, to assure that some congestion and queuing would be present during the work zone. In this research, the following criteria are used to select data collection sites:

- Lane closure configuration,
- Pavement condition, including travel on milled pavement, or travel across a ledge,
- Work zone intensity and location, including the presence of heavy equipment or lights that may impact driver behavior,
- Horizontal and vertical design details, such as work zones located in sharp curves or on hill crests that cause sight distance constraints, and
- Flat approach grade, as to not confound work zone effects with impacts of steep upgrades for example.

Data reduction was performed using a time-stamped computer macro to measure headways between successive vehicles. The data reduction method followed guidance in Chapter 6 of the ITE Manual of Transportation Studies (MTES) (22). The estimated saturation flow rate from field data is defined as the number of vehicles that pass the stop bar at the intersection with no interruptions in a defined period. The MTES describes a method for gathering saturation flow rate observations. In this method, a timer is started when the fourth vehicle in a queue passes the stop bar, as this is the point when the queue usually starts keeping consistent headways, and is after any incurred start-up lost time. The timer is stopped when either the seventh, eighth, ninth, or tenth vehicle passes the stop bar, whichever was the last in the stopped queue. With a typical standard deviation in saturation flow rate of 140 vehicles per hour, the MTES recommends observing a minimum of 30 valid queues to estimate the mean saturation flow rate within 50 vehicles per hour of the true rate with 95% confidence (22). It is noted that any queue with heavy vehicles or with less than seven passenger cars will be excluded from analysis in this research, to avoid confounding effects.

Saturation headway data are then aggregated at the cycle length level. It is important to note that at this aggregation level, the effects of individual driver behaviors can introduce large fluctuations in observed saturation headways. As a result, the team also attempted a 15-minute aggregation level for each unique set of independent variables (i.e., work zone configuration and prevailing conditions), which is consistent with the HCM 15-minute analysis period and is expected to reduce the variations that exists at a cycle length aggregation level. Furthermore, a third aggregation level generates a single saturation headway for each unique combination of independent variables or work zone configuration.

In addition to vehicle headway and time stamp, the research team collected right turn and left turn vehicle percentage, lane width, heavy vehicle percentage, number of lanes, lane configuration, right turn angle shape, equipment location, work intensity, work activity location, pavement type, and ledge presence.

Multiple Linear Regression Model

Intersection work zones can influence the saturation flow rate and consequently the capacity. It is assumed that variables such as the reduced number of lanes, non-ideal pavement conditions, presence of ledges, intensity of work activity, percentage of heavy vehicles, and percentage of turning movement are key contributing factors to changes in the saturation headway.

A regression approach can use either additive or multiplicative models to predict saturation headways. This study employs additive models due to their simplicity and consistency with the HCM. In additive models, the base saturation headway (for non-work zone conditions) is adjusted by subtracting or adding values corresponding to different work zone configurations and prevailing conditions. The following equation shows an example of additive model. The intercept (h_s) represents non-work zone condition with 100% through vehicles, and dependent variable (h_s^{adj}) represent adjusted saturation headway under a certain work zone configuration and prevailing conditions.

$$\boldsymbol{h_s^{adj} = h_s + \beta_1 RTP + \beta_2 LTP + \beta_3 CEL + \beta_4 CER + \beta_5 RTG + \beta_6 WI + \cdots + \beta_{n-1} PC + \beta_n PL}$$

Where:

h_s^{adj}: adjusted saturation headway (sec),

h_s : non-work zone average saturation headway (for through vehicles_,

RTP : right turn percentage,

LTP : left turn percentage,

CEL : closed exclusive left turn lane (0: open, 1: closed),

CER : closed exclusive right turn (0: open, 1: closed),

RTG : right turn geometry (0: sweeping, 1: tight),

WI : work intensity,

PC : pavement condition (0: normal, 1: milled condition), and

PL : pavement ledge (0: no, 1: yes).

Work intensity variable can take on three values: no activity, low intensity, and high intensity. No activity corresponds to the condition when a lane closure is in place with no construction equipment or crew present in the work zone. For low intensity conditions, the lanes are closed and construction equipment is present but with no construction activity underway. High intensity condition is defined when construction activity is underway.

The intercept in a regression analysis typically emerges from the observed data, as something like the average headway for all work zone data used in the analysis. However, the research team postulated that it would be desirable to explore the work zone effect relative to non-work zone conditions. The team therefore decided to fix the intercept (h_s) at the average non-work zone saturation headway from the field-observed data. This approach brings the advantage that all explanatory variables represent direct effects of the work zone relative to the base saturation headway. It further allows analysts to adapt the model to local conditions, by calibrating the intercept to a local estimate of non-work zone saturation headway.

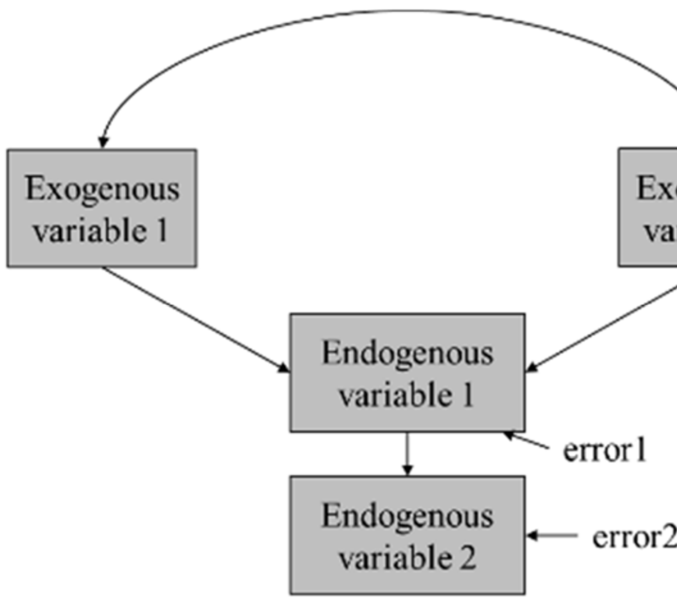
Path Analysis

Path analysis is used to identify the relationship between average headway per cycle and its causal factors in this study. Path analysis is a statistical analytic method which is generally used to investigate the comparative effectiveness of direct and indirect relationships among variables. The path analytic method is also an extension of the multiple regression model and a part of Structural Equation Model (SEM). Thus, it follows all the usual assumptions of both regression model and SEM such as linearity, additivity, uncorrelated residual variables, low multi-collinearity, and so forth.

An SEM or its parts such as path analysis and Multiple Indicators Multiple Causes (MIMIC) model have been applied in transportation, as well as economics, psychology, political science, and other fields. In transportation, path analysis and SEM have been used in various fields including driver behavior modeling, mode choice modeling, and public acceptability analysis of novel policies for traffic congestion mitigation management.

A path analysis consists of two types of variable: 1) Exogenous variable; 2) Endogenous variable. Exogenous variables are variables determined by causes outside the model, whereas endogenous variables are variables that are caused by one or more variables inside the model. The explanation of elements of a path diagram is depicted in Exhibit 49. There is a causal relationship leading the independent (causal) variables to the dependent variables indicated by the single straight arrows. On the other hand, the double-headed curved arrow connecting two exogenous variables indicates the correlation, but no prediction will remain due to the fact that no variable causes other variable in the model. In addition, extraneous variables are indicated by the arrow from Errors (Residuals or error terms) which reflect unspecified causes of variability in unexplained variance with any error due to measurement. There are some advantageous when it comes to using the path analytic method. First, the path analysis can be more flexible assumptions compared to multiple regression methods. Second, the powerful and unique advantage of the path analysis is to decompose association among variables into causal (i.e., direct and indirect) and non-causal components (e.g., spurious). Finally, researcher can control detailed theoretical models within the path diagram.

Exhibit 49: Path Diagram



Results

Data Summary

After completing data reduction, a total of 795 cycle-level observations for non-work zone condition and 677 observations for work zone conditions were recorded for 19 different configurations. These observations included queue observations of at least seven vehicles that did not contain any heavy vehicles. Exhibit 50 and Exhibit 51 show the summary of data reduction results.

Exhibit 50 shows the headway distributions in non-work zone and work zone conditions across all data collection sites, as an illustration of the type of data collected. From the figure, it is apparent that work zone presence at intersections results in a significant shift in saturation headway distribution towards longer headways.

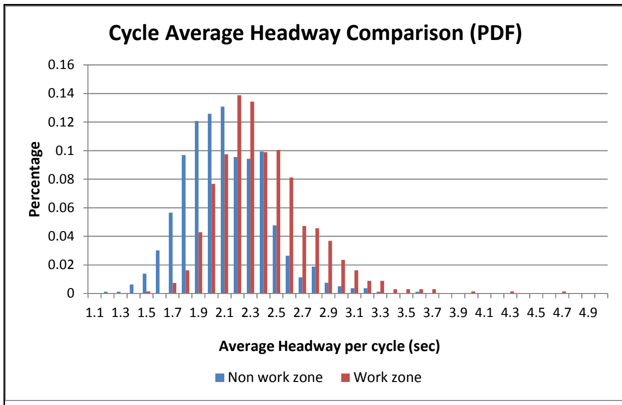
Exhibit 51 shows the probability density function (PDF) and cumulative distribution function (CDF) of all collected data. The plots show that at all data collection locations, the base saturation headway is less than the one during work zone, as expected. The same is true for the standard deviation of saturation headway, which indicates a more uniform headway distribution in non-work zone conditions.

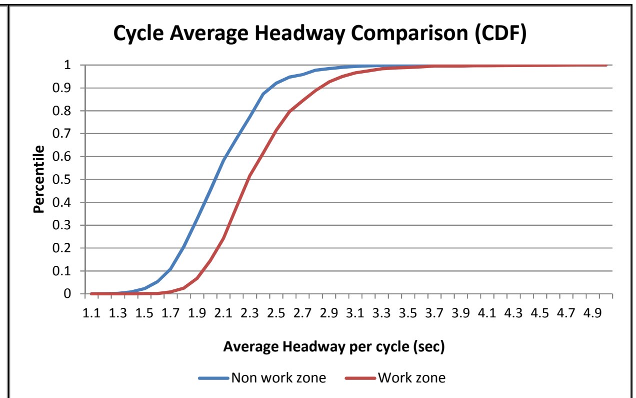
Exhibit 50: Data Collection Summary

Location	Non Work Zone Conditions				Work Zone Conditions			
	Sample Size		Average Headway (sec)	Standard Deviation of Headways	Sample Size		Average Headway	Standard Deviation of Headways
	Number of vehicles*	Number of Cycles			Number of vehicles*	Number of Cycles		
Wade Ave EB at Faircloth St	498 (249)	140	2.2	0.365	877 (433)	59	2.46	0.427
Wade Ave WB at Faircloth St	419 (209)	146	1.9	0.320	858 (438)	94	2.36	0.412
Lake Wheeler Rd SB at I-40	511 (322)	63	2.1	0.248	589 (314)	62	2.37	0.345
MLK Blvd EB at Hope Valley Rd	659 (338)	107	2.05	0.254	881 (443)	146	2.3	0.309
MLK Blvd EB at Roxboro St	1025 (644)	127	2.07	0.277	1516 (966)	188	2.38	0.331
MLK Blvd WB at Roxboro St	893 (557)	113	2.06	0.307	799 (518)	99	2.3	0.407
US 70 Bus EB at Shotwell Rd	598 (301)	99	2.11	0.325	180 (90)	29	2.37	0.437
Total	4603 (2620)	795	2.07	0.321	5700 (3202)	677	2.35	0.369

**The number represents the total number of observed headways; the number in parenthesis indicates the number of headway after the fourth vehicle.*

Exhibit 51: Cycle Based Average Headway Comparison





(a) PDF of Cycle Average Headway **(b) CDF of Cycle Average Headway**

Variable Selection

As the first steps in model development, a correlation test was used to identify any collinearity between candidate independent variables. Variables were considered to have strong correlation with correlation coefficients larger than |0.5|. The research team assured that no pair of independent variables with strong correlation is included in the list of candidate independent variables at the same time. After this step, the list of candidate variables emerged as follows:

- RTP (Percentage of right turn vehicles),
- LTP (Percentage of right turn vehicles),
- CEL (Exclusive left turn lane closure), (0: open, 1: closed),
- CER (Exclusive right turn lane closure), (0: open, 1: closed),
- RTG (Right turn geometry), (0: sweeping, 1: tight),
- LWI (Work intensity: low), (0: no, 1: yes),
- HWI (Work intensity: high), (0: no, 1: yes),
- PC (Pavement condition) (0: normal, 1: milled condition), and
- PL (Presence of a ledge) (0: no, 1: yes).

All intersections considered in this study had a single through lane closure during work zone conditions. Furthermore, the lane width at all data collection sites was approximately 12 ft for both normal and work zone conditions. Consequently, the lane width was not included in the modeling approach.

Multiple Linear Regression Model Results

As mentioned previously, the intercept of the model was set to the average saturation headway for non-work zone conditions at the data collection sites. This value, based on the data we collected in North Carolina, is 2.04 seconds, which corresponds to a saturation flow rate of 1,764 passenger cars per hour per lane (pcphpln). It is noted that right turn geometry, right turn percentage, and exclusive right turn closure variables showed very strong correlation. Among these three variables, right turn geometry was associated with the highest p-value in all three models and had the lowest contribution to the value of the adjusted R-squared. Thus, it was excluded from the models.

The right and left turn percentage values, when included separately in the model, had p-values larger than 0.1 (in all three aggregation levels); however, when combined, the p-values were reduced to acceptable thresholds (see p-values for variable TP in Exhibit 46). This means that these two variables were combined to have identical coefficients in each model. We defined a new variable, turn percentage from shared lanes (TP), to represent them. The same trend was observed for exclusive left and right turn closure variables. Consequently, we defined a new variable called the number of closed exclusive turn lanes (EX) to represent these two variables.

The final regression model form is:

$$\boldsymbol{h_s^{adj}} = \boldsymbol{h_s} + \boldsymbol{\beta_1 LWI} + \boldsymbol{\beta_2 HWI} + \boldsymbol{\beta_3 PC} + \boldsymbol{\beta_4 PL} + \boldsymbol{\beta_5 (TP)} + \boldsymbol{\beta_6 (EX)}$$

where : turning percentage from shared lanes,

EX: number of closed exclusive lanes, and all other variables are as defined previously.

Exhibit 52 summarizes the results of the multiple linear regression models at each aggregation level, starting from the cycle level at the top, 15-minute aggregation level at the middle, and fully aggregated model at the bottom. With a cycle-level aggregation, the model returned an adjusted R-squared value of 0.329. This value represents a good fit for aggregation at the cycle level since driver behavior introduces significant variation to the observed saturation headways. As shown in the Exhibit 46, all variables are significant with all p-values < 0.1. In addition, all variable coefficients have positive signs, as expected, indicating they increase saturation headway. Furthermore, high work intensity has a larger coefficient than low work intensity as expected.

Increasing the aggregation level to 15 minutes increased the value of the adjusted R-squared to 0.71. This is again expected as this aggregation level suppresses the impact of individual driver behavior on saturation headways. At this aggregation level, all independent variables are statistically significant with p-values < 0.1. Furthermore, all variable coefficients are positive and follow a trend similar to cycle-level aggregation.

Exhibit 52: Multiple Linear Regression Result

Cycle-Level Aggregation					
Model Statistics	**Value**				
R^2	0.3259				
Adjusted R^2	0.3229				
Number of Observation	1,472 cycles average headway				
Parameter Estimates					
Variable	**DF**	**Parameter Estimate**	**Standard Error**	**t Value**	**Pr > |t|**
Intercept	1	2.04	-	-	-
LWI	1	0.136	0.0265	5.15	<.0001
HWI	1	0.264	0.0275	9.61	<.0001
PC	1	0.217	0.0247	8.77	<.0001
PL	1	0.116	0.0286	4.07	<.0001
TP	1	0.003	0.0004	7.73	<.0001
EX	1	0.051	0.0268	1.90	0.0563

15-minute Aggregation					
Model Statistics	**Value**				
$\mathbf{R}^2$	0.7209				
Adjusted $\mathbf{R}^2$	0.7128				
Number of Observation	301				
Parameter Estimates					
Variable	**DF**	**Parameter Estimate**	**Standard Error**	**t Value**	**Pr > |t|**
Intercept	1	2.04	-	-	-
LWI	1	0.136	0.02519	5.414841	<.0001
HWI	1	0.271	0.026207	10.36285	<.0001
PC	1	0.211	0.023399	9.038081	<.0001
PL	1	0.112	0.027086	4.147682	<.0001
TP	1	0.003	0.000734	4.90602	<.0001
EX	1	0.055	0.025902	2.137819	0.0333

Fully Aggregated					
Model Statistics	**Value**				
$\mathbf{R}^2$	0.9752				
Adjusted $\mathbf{R}^2$	0.895				
Number of Observation	20				
Parameter Estimates					
Variable	**DF**	**Parameter Estimate**	**Standard Error**	**t Value**	**Pr > |t|**
Intercept	1	2.04	-	-	-
LWI	1	0.117	0.038489	3.047604	0.0086
HWI	1	0.273	0.030145	9.075614	<.0001
PC	1	0.224	0.031866	7.033681	<.0001
PL	1	0.115	0.029534	3.897105	0.0016
TP	1	0.003	0.001415	2.217567	0.0436
EX	1	0.055	0.031815	1.733478	0.1049

The highest aggregation level predictably retuned the highest adjusted R-squared values. Again all variable are statistically significant with all p-values < 0.1, except for the number of closed exclusive turn lanes that has a p-value of 0.105. All coefficients have positive signs with trends similar to the other two models.

Path Analysis Result

The proposed model used fixed coefficients for percentage of right turn vehicles, percentage of left turn vehicles and intercept since collected non-work zone 100% through vehicle's headway data indicate average of 2.04 second's saturation headway as well as HCM provides right and left turn equivalent values. The equivalent value provided by HCM (E_R: 0.15, E_L: 1.05) are directly converted to percentage of turning vehicle's coefficient. The corresponding coefficients are 0.00306 (f_{RT}: 0.869) and 0.00102 (f_{LT}: 0.952) for right turn and left turn coefficient, respectively. Therefore, the model is modified in order to use fixed effect of independent variables.

The proposed model's estimated coefficient, standard errors, critical ratio and p-values are shown in Exhibit 53. All independent variables show less than 0.024 p-values which are all acceptable range for the model. The proposed urban arterial work zone adjusted saturation headway estimation model using HCM turning movement effect is:

$$Y=2.04+0.00306(RTP)+0.00102(LTP)+0.094(CEL)+0.114(CER)+0.117(RTG)+0.138(LWI)+0.219(HWI)+0.245(PC)+0.110(PL)$$

The estimated model coefficients show that when the work zones have milled pavement condition, it increases 0.245 seconds of saturation headway. The worst case which is conditions of 100% right turn vehicles with all work zone condition except low work intensity, provides 3.245 seconds of adjusted saturation headway (saturation flow rate: 1,109 veh/h/ln).

Exhibit 53: Path Analysis Result

Absolute fit indices	Model value	Threshold
x^2/df	4.751	$2 < x^2/df < 5$
RMSEA	0.050	< 0.08
GFI	0.984	> 0.9
AGFI	0.965	> 0.85
CFI	0.929	> 0.9

Values		Coefficient	S.E.	C.R.	P
Fixed effect coefficient value	Intercept	2.040	-	-	-
	RTP	0.00306	-	-	-
	LTP	0.00102	-	-	-
Variable coefficient value	*CEL*	0.094	0.033	2.863	0.004
	CER	0.114	0.050	2.263	0.024
	RTG	0.117	0.032	3.604	<.0001
	LWI	0.138	0.043	3.212	0.001
	HWI	0.219	0.059	3.718	<.0001
	PC	0.245	0.024	10.429	<.0001
	PL	0.110	0.030	3.712	<.0001

It is essential to assess goodness-of-fit and the estimation of parameters of the hypothesized model in analyzing with the path method. Some studies have suggested using the conventional cutoff criteria for a variety of indices those can assess goodness-of-fit such as goodness-of-fit index (GFI), comparative fit index (CFI), root mean square error of approximation (RMSEA) as well as χ^2 test. Also the studies suggested using some rules of thumb criteria for goodness-of-fit indices. There is an agreement for an acceptable model which should be less than RMSEA of less than 0.08, GFI and CFI of more than 0.9, AGFI of above 0.85 and between 2 and 5 for x^2/df although the studies proposed using slightly different cut criteria. Exhibit 47 shows the statistics for goodness-of-fit tests of the estimated path analysis model. The calculated all fit indices indicate that the model have a satisfied fitness. The proposed model used disaggregated cycle-by-cycle field observed headway data. The observed headways are varied under variety work zone condition.

Summary of Model Result

Exhibit 54 shows result validation plots of model fitting for all four candidate regression models. The values on the y-axis represent observed saturation headway values and the x-axis shows headways predicted by our models. It is apparent from the figure that cycle level aggregation yields a considerable scatter (see Exhibit 54 a & d). On the other hand, the other two aggregation levels result in much less fluctuations.

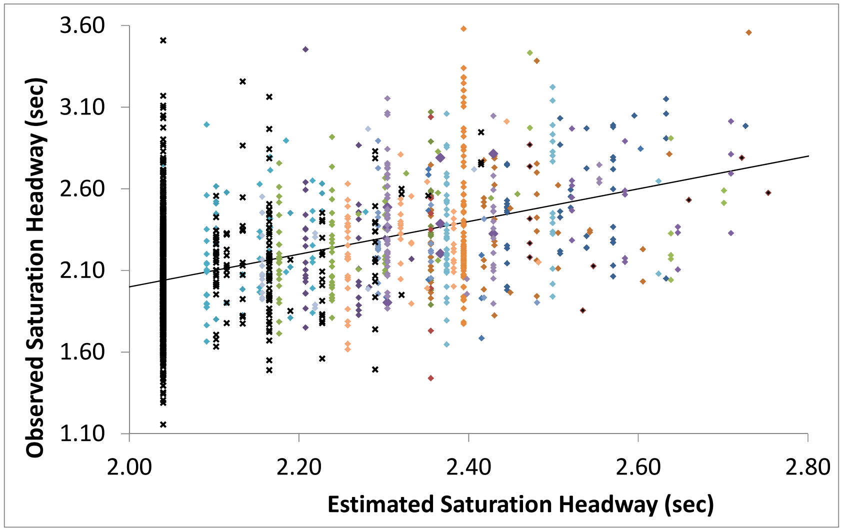
Exhibit 55 lists the saturation headways and corresponding saturation flow rate for the 19 unique work zone configurations that were observed in this study. Field headway observations are presented as well. As shown in the table, all modeling approaches predict saturation headways and flow rates that are at most 7.5% divergent from the observed values under identical work zone configurations.

The model that uses 15 minutes aggregated data shows a very good fit and is consistent with the 2010 HCM analysis period duration. Therefore, we recommend its adoption, especially when data on turning percentages are scarce. The model form is as follows:

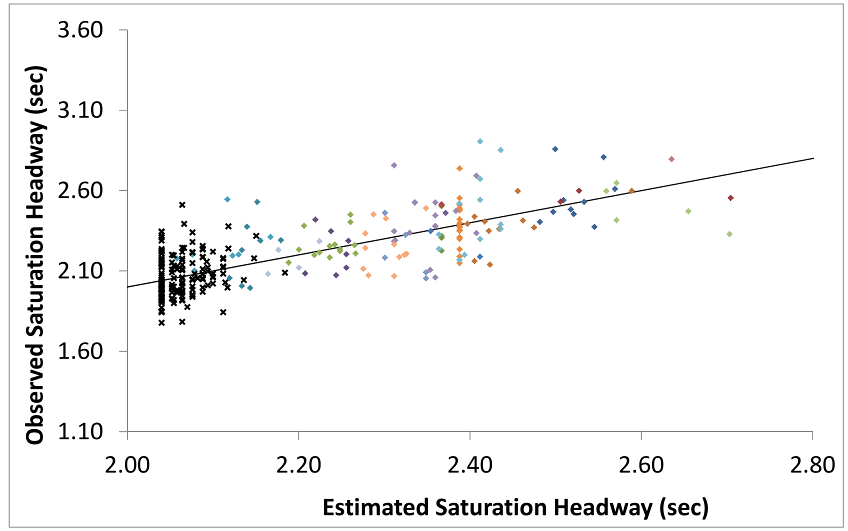
$$\boldsymbol{h_s^{adj} = 2.04 + 0.136(LWI) + 0.271(HWI) + 0.211(PC) + 0.112(PL) + 0.003(TP) + 0.055(EX)}$$

Where all variables are as previously defined.

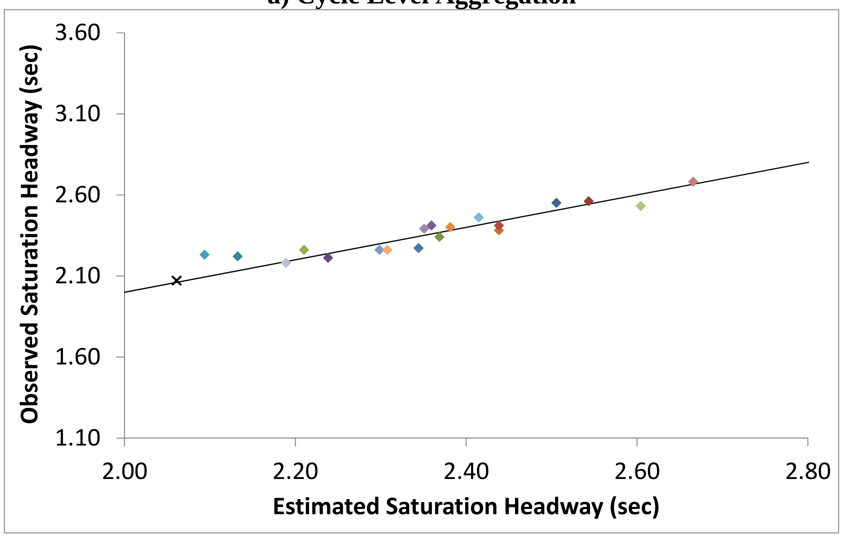
Exhibit 54: Field Observation vs Model Estimation Result



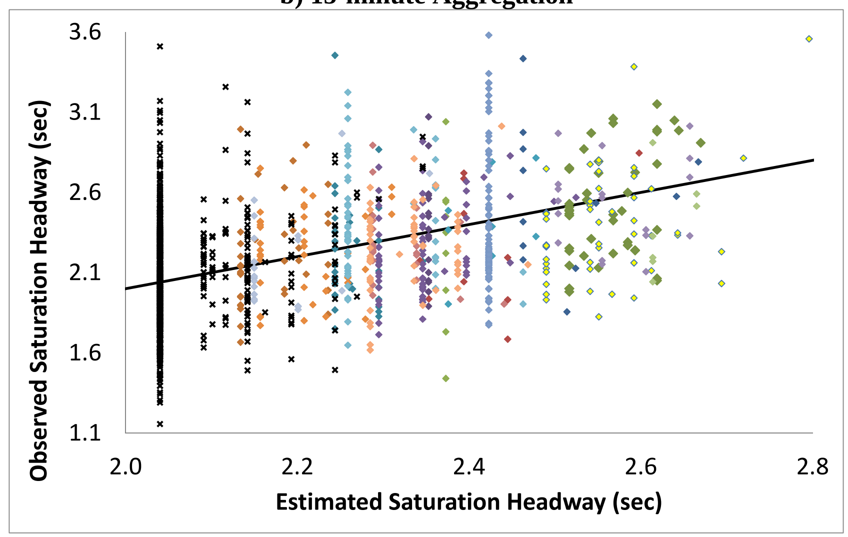
a) Cycle Level Aggregation



b) 15-minute Aggregation



c) Work Zone Configuration Level Aggregation



d) Path Analysis – Cycle Level Aggregation

Exhibit 55: Saturation Headway and Flow Rate Prediction of Model

Work Zone Configuration									Field		Cycle-Level Aggregation		15-Min Aggregation		Fully Aggregated		Path Analysis	
RTP	LTP	CEL	CER	TG	LWI	HWI	PC	L	$h_{s_{adj}}$ (s)	Flow Rate (v/h/l)	$h_{s_{adj}}$ (s)	Flow Rate (v/h/l)	$h_{s_{adj}}$ (s)	Flow Rate (v/h/l)	$h_{s_{adj}}$ (s)	Flow Rate (v/h/l)	$h_{s_{adj}}$ (s)	Flow Rate (v/h/l)
12.6	9.4	1	0	0	1	0	1	0	2.55	1412	2.53	1424	2.51	1432	2.51	1437	2.57	1403
14.2	4.9	1	0	0	0	1	0	1	2.56	1406	2.54	1415	2.54	1419	2.54	1415	2.51	1434
0.0	0.0	1	0	0	0	1	0	0	2.34	1538	2.36	1528	2.36	1526	2.37	1520	2.35	1530
5.6	3.4	1	0	0	0	0	0	1	2.21	1629	2.24	1606	2.24	1606	2.24	1608	2.26	1590
11.9	0.0	1	0	0	0	0	0	0	2.22	1622	2.14	1685	2.14	1686	2.13	1688	2.17	1659
22.2	0.0	0	1	1	0	1	0	0	2.38	1513	2.44	1476	2.43	1481	2.44	1476	2.56	1407
42.0	0.0	0	1	0	1	0	0	0	2.27	1586	2.39	1509	2.36	1527	2.34	1536	2.42	1487
0.0	0.0	0	1	0	0	1	0	0	2.41	1494	2.36	1528	2.36	1526	2.37	1520	2.37	1517
16.9	0.0	0	0	1	1	0	0	0	2.26	1593	2.24	1607	2.22	1623	2.21	1629	2.35	1534
14.6	0.0	0	0	1	0	1	0	0	2.41	1494	2.36	1526	2.35	1532	2.36	1526	2.42	1487
17.1	0.0	0	0	1	0	0	0	0	2.23	1614	2.10	1711	2.10	1717	2.09	1719	2.21	1629
0.0	0.0	0	0	0	1	0	1	0	2.40	1500	2.39	1504	2.38	1510	2.38	1512	2.42	1486
8.3	0.0	0	0	0	1	0	0	1	2.26	1593	2.32	1549	2.31	1561	2.30	1566	2.31	1556
4.2	0.0	0	0	0	0	1	1	1	2.68	1343	2.65	1356	2.65	1357	2.67	1350	2.63	1371
21.3	0.0	0	0	0	0	1	1	0	2.53	1423	2.60	1384	2.59	1388	2.60	1382	2.57	1401
11.9	0.0	0	0	0	0	1	0	0	2.39	1506	2.35	1533	2.34	1538	2.35	1531	2.30	1568
11.3	0.0	0	0	0	0	0	1	1	2.46	1463	2.42	1490	2.41	1491	2.41	1491	2.43	1482
14.0	0.0	0	0	0	0	0	1	0	2.26	1593	2.31	1559	2.31	1560	2.31	1560	2.33	1547
10.8	0.0	0	0	0	0	0	0	1	2.18	1651	2.20	1638	2.19	1643	2.19	1645	2.18	1649
6.7	0.0	0	0	0	0	0	0	0	2.07	1739	2.07	1743	2.06	1746	2.06	1747	2.06	1747

Should left turn and right turn percentages be significant and important to account for their impacts separately, the analyst may consider using the model based on the path analysis approach, as it allows using the 2010 HCM recommendations to adjust for right and left turn percentages.

Exhibit 56 presents a summary of the variable coefficients for all four models. All have an intercept value of 2.04 representing saturation headway values for non-work zone conditions. The major difference between the path analysis approach and the traditional regression approach is that the former allows the use of predetermined coefficient values for right and left turn movements. The coefficients of independent variables in all four models have approximately similar values indicating an overall similarity across models.

Exhibit 56: Data aggregation result by selected independent variable and location

Coefficient of Independent Variables				
Independent Variable	Conventional Regression			Path Analysis based Model
	Cycle Level Aggregated Model	15 Minutes Aggregation Model	Fully Aggregated Model	
Intercept	2.04	2.04	2.04	2.04
LWI	0.136	0.136	0.117	0.138
HWI	0.264	0.271	0.273	0.219
PC	0.217	0.211	0.224	0.245
PL	0.116	0.112	0.115	0.110
TP	0.003	0.003	0.003	-
EX	0.051	0.055	0.055	-
RTP	-	-	-	0.00306
LTP	-	-	-	0.00102
CEL	-	-	-	0.094
CER	-	-	-	0.114
RTG	-	-	-	0.117

Default Value Development

This section introduces a summary of default and estimation methods to be included in the ARTVAL computation engine. These defaults and quick estimation methods are intended to simplify user input, reduce data collection needs, and overall make the methodology and computational engine applicable in planning-level applications. All default values can of course be overridden by the user based on local data or expert judgment.

For signal timing, the 2010 Highway Capacity Manual (HCM2010) offers a Quick-Estimation Method (QEM) for the estimation of signal timing parameters in Chapter 31. The Quick Estimation Method for Urban Streets (QEM-US) is fully described in Chapter 30 in HCM2010. The QEM-US method is developed to evaluate the operation of an undersaturated coordinated urban street segment with signalized boundary intersections. The main focus of the approach is to analyze the performance of the through traffic movement at the boundary intersections.

The quick estimation methods require several input data that were listed previously in Exhibit 17. In a planning level analysis, it can be hard to collect all field data, including as signal operation parameters and turning movement data. The default values presented below help overcome these data needs.

Overall, the team developed default values or quick estimation methods for six key input parameters in the methodology analysis:

1. Hourly directional demand distribution
2. Total delay due to turn into access points (second/vehicle)
3. Lost time per cycle
4. Proportion of arrivals during green
5. Delay due to mid-segment sources (second/vehicle)
6. Turning vehicle percentage

These default values are either needed for direct input in the urban streets quick estimation methodology in the Highway Capacity Manual, or are needed to facilitate the conversion of planning-level volume inputs to 15-minute data. All proposed default values and estimation methods are used in the ARTVAL computation engine.

Hourly Directional Demand Distribution

The directional distribution of traffic (usually indicated by AADT) varies by time of day. Therefore, it is necessary to compute the hourly directional demand distribution in order to calculate performance measures for the arterial. The hourly directional demand distribution default values are developed by analyzing real-world arterial detector data. The project team proposes two type of hourly directional demand distribution default values, since the arterials could have different distribution pattern depending on their location and types. For example, some arterials might have only one peak pattern such as AM or PM peak, while other arterials might have both AM and PM peak pattern.

One peak flow pattern hourly directional demand distribution data were collected from the NCDOT signal system. The NCDOT signal system can provide one minute aggregated loop detector data. Exhibit 57 depicts the US 70 arterial in Clayton and Garner with respective brief description. The log files were downloaded from OASIS^{TM} for three intersections over an eight weekday time span. In addition, two peak flow pattern hourly directional demand distribution data were provided from the City of Cary Department of Transportation.

The proposed one peak pattern hourly directional demand distribution is created based on three intersections on the US 70 arterial in Clayton and four intersections on the US 70 arterial in Garner. Exhibit 58 and Exhibit 59 show the result of one peak flow pattern percentage of hourly flow to AADT.

Exhibit 57: One Peak Pattern Arterial Data Collecting Sites

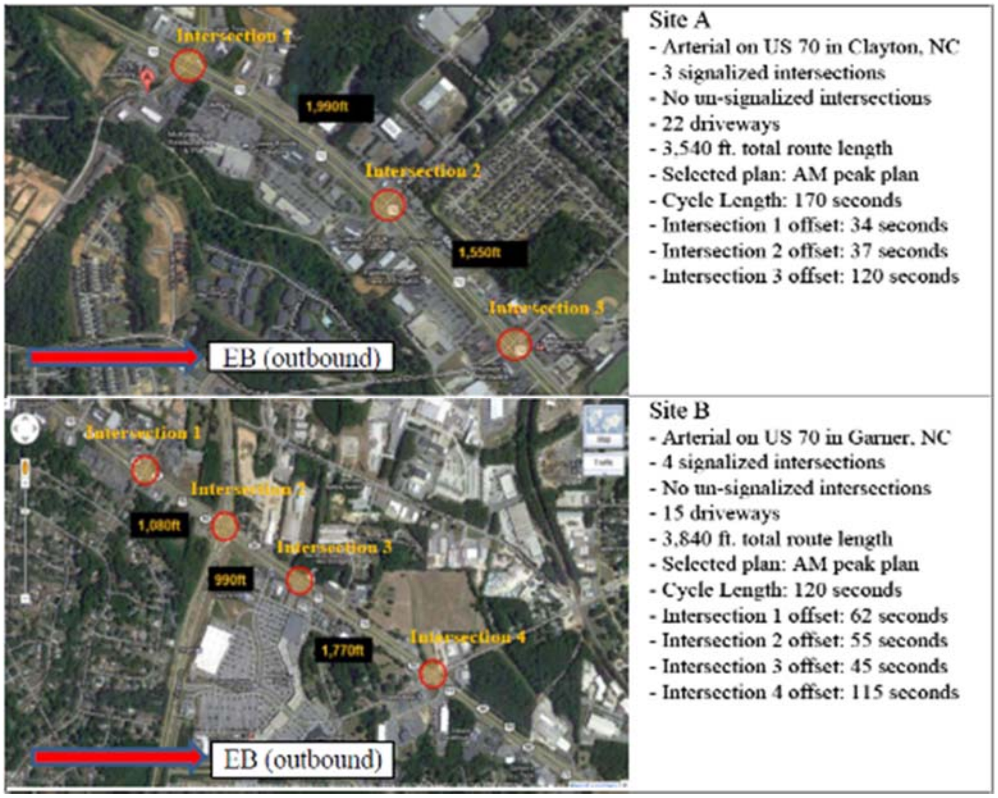


Exhibit 58: One Peak Flow Pattern Hourly Directional Demand Plot

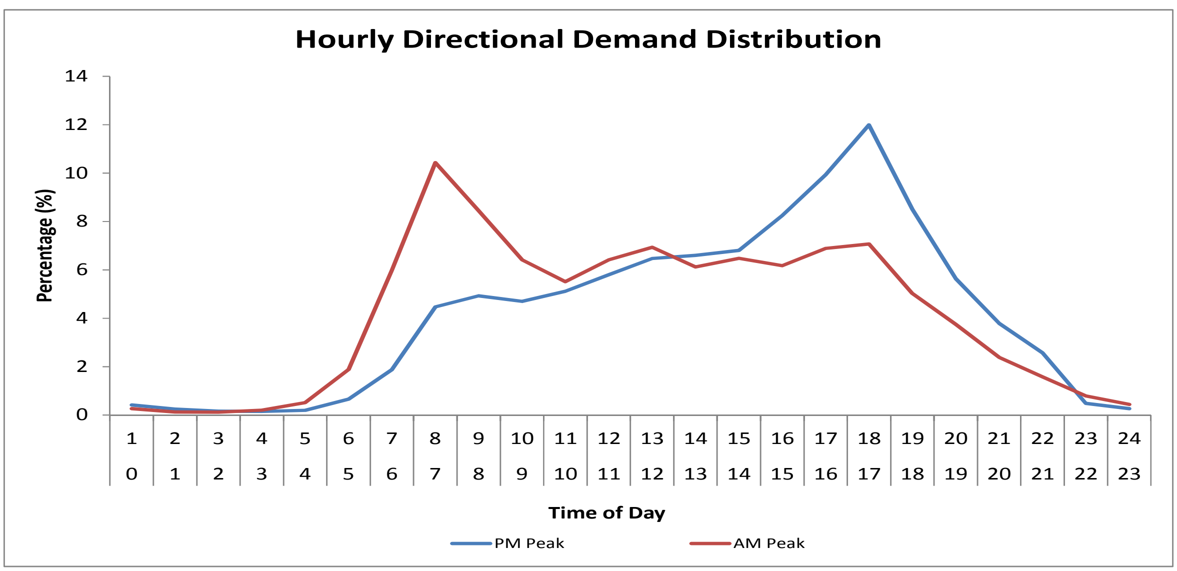


Exhibit 59: One Peak Flow Pattern Hourly Directional Demand Distribution

Hour		Demand Distribution in Direction 2 (PM Peak)	Demand Distribution in Direction 6 (AM Peak)
From	To		
0	1	0.416	0.268
1	2	0.253	0.132
2	3	0.158	0.121
3	4	0.149	0.209
4	5	0.195	0.516
5	6	0.665	1.886
6	7	1.880	5.990
7	8	4.467	10.431
8	9	4.930	8.451
9	10	4.697	6.414
10	11	5.115	5.513
11	12	5.799	6.420
12	13	6.468	6.934
13	14	6.597	6.124
14	15	6.803	6.475
15	16	8.245	6.177
16	17	9.928	6.887
17	18	11.988	7.071
18	19	8.502	5.026
19	20	5.634	3.750
20	21	3.792	2.389
21	22	2.572	1.581
22	23	0.485	0.792
23	24	0.265	0.442

Two peak pattern arterial field data are provided from City of Cary DOT. The provided data are 15 minute resolution signal detector log data for one month of two intersections on East Cary Parkway arterial. Exhibit 60 shows the provided data locations.

Exhibit 60: Two Peak Pattern Arterial Data Collecting Sites

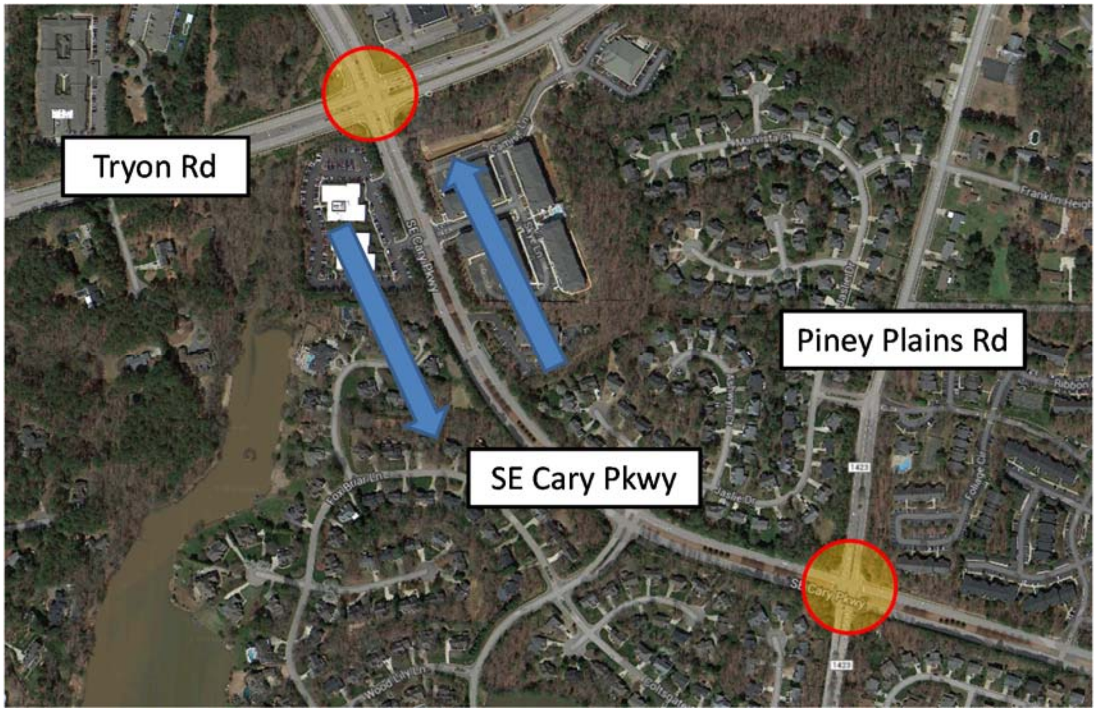


Exhibit 61 and Exhibit 62 show the results of the two peak flow pattern percentage of hourly flow to AADT.

Exhibit 61: Two Peak Flow Pattern Hourly Directional Demand Plot

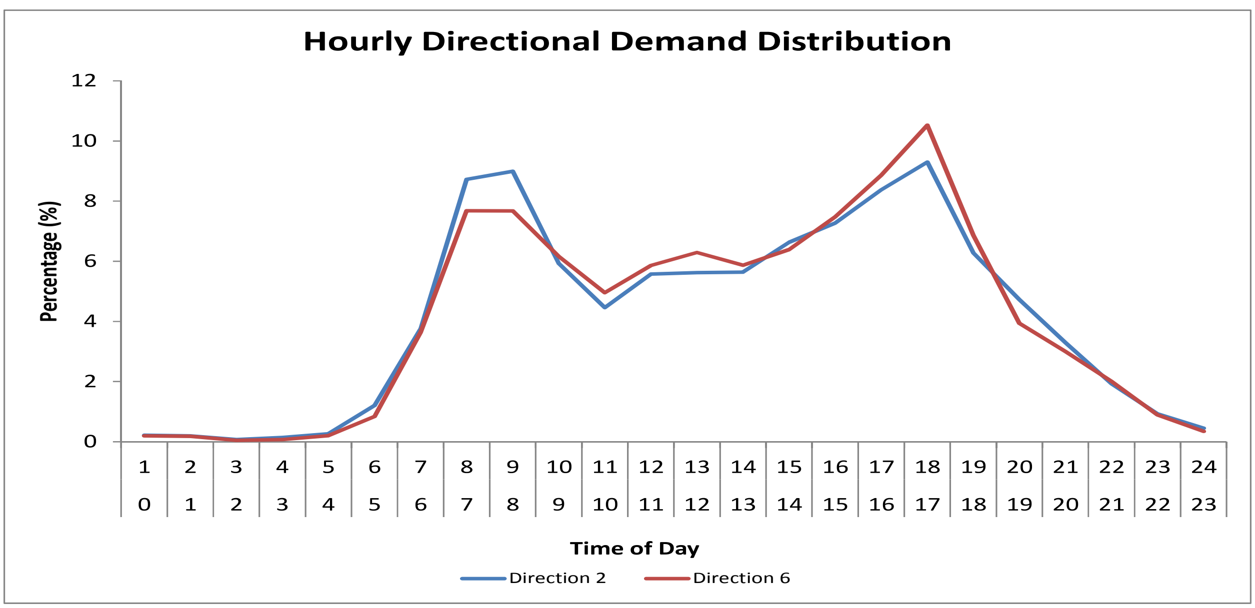


Exhibit 62: Two Peak Flow Pattern Hourly Directional Demand Distribution

Hour		Demand Distribution in Direction 2	Demand Distribution in Direction 6
From	To		
0	1	0.218	0.195
1	2	0.194	0.186
2	3	0.073	0.049
3	4	0.145	0.073
4	5	0.267	0.203
5	6	1.204	0.843
6	7	3.749	3.640
7	8	8.727	7.685
8	9	8.994	7.677
9	10	5.931	6.161
10	11	4.461	4.953
11	12	5.576	5.861
12	13	5.624	6.291
13	14	5.640	5.869
14	15	6.634	6.388
15	16	7.273	7.483
16	17	8.372	8.861
17	18	9.301	10.523
18	19	6.279	6.867
19	20	4.727	3.940
20	21	3.297	3.000
21	22	1.931	2.011
22	23	0.929	0.892
23	24	0.453	0.349

It is important to note that the proposed default value cannot perfectly represent the hourly directional demand profiles for all arterials. Therefore, the project team recommends using these default values only when there is no existing filed data.

Total Delay Due to Turn into Access Points

This parameter is used to adjust the segment running time due to delay incurred from mid-segment sources, and specifically from vehicles turning into and out of access points (i.e. driveways). It was beyond the scope of this project to develop customized defaults for this parameter, and the team thus relied on results from a national study to guide the default estimation (NCHRP Report 395).

The travel time on an arterial street is affected by the number of access points, or specifically the *access point density*. NCHRP Report 395 defined access point as "All unsignalized access Locations. An access point can be either a driveway or a public street approach". In addition, the report defines a driveway as any location on the arterial where the curb along the outside lane is removed or dropped for 10 feet or more to facilitate vehicular access to the adjacent property. The definition of access point density is the total number of access points on both sides of the major-street divided by the length of the segment. According to NCHRP Report 395, the effect of access point density is accounted by reducing the free-flow speed by 2.5 mph for each 10 access point per mile. Therefore, the total delay due to turn into access point should be the link distance divided by 2.5 mph for each 10 access point per mile.

Building on the NCHRP research result, the project team collected access point density data from urban and suburban areas in North Carolina. In total, 16 corridors on urban area arterials (Raleigh and Durham) showed an average access point density of 18.7 access points per mile. The team also evaluated 13 corridors on suburban area arterials (NC 54, NC 55, SR 1526, NC 96, NC 86, NC 97, US 15, US 96, US 56, US 50, US 42, BUS 70, US 70), and determined an average access point density of 8.96 per mile.

From these results, the team proposed default free-flow speed adjustment default values of -4.7 mph and -2.2 mph for urban and suburban arterials, respectively. The analyst may customize those defaults as needed based on local knowledge.

Lost Time per Cycle

Lost time per cycle is a function of the phase sequence, and occurs every time a signal transitions from one critical phase to the next. For instance, an intersection with two critical phases experiences lost time twice per cycle. The total lost time per cycle is thus formulated as the sum of lost times for each critical phase i (in seconds).

$$L = \sum_{i=1}^{n} t_{l_i}$$

Where,

$$t_{l_i} = \text{ } lost\ time\ for\ critical\ phase\ I\ (sec)$$

$$L = lost\ time\ for\ the\ entire\ cycle\ (sec/cycle)$$

Synchro 6 and the HCM both use a default value of 4 second per phase for total lost time. However, NCDOT signal design guideline uses 5 seconds per critical phase as a default lost time. The research team therefore proposes to use 5 seconds per critical phase as lost time.

Proportion of Arrivals during Green

A key characteristic of arterial streets is that many facilities have coordinated traffic signals. As such, the proportion of vehicles arriving in green is not random, but is optimized based on relative travel times between signals. In order to estimate the proportion of arrivals during the green, the use of three different arrival types is proposed for this research. Each arrival type will be associated with a platoon ratio, and with the aid of the HCM 2010 method, will estimate the proportion of arrivals during the green. Default values for the procedure are given in Exhibit 63.

$$P = R_p * (\frac{g}{C})$$

Where,

$P = proportion\ of\ vehicles\ arriving\ during\ the\ green\ indication$

$R_p = platoon\ ratio,$

$g = effective\ green\ time(s), and$

$C = cycle\ length\ (s).$

Exhibit 63: Default Values for Proportion Arrivals During Green

Configuration Type	Number of Critical Phases	Lost Time (sec) per Critical Phase	Platoon Ratio in Peak Direction	Platoon Ratio in Off-Peak Direction
Free Running Arterial	4	5	1	1
Medium Coordination of Arterial's Peak Direction	4	5	1.33	1
High Coordination of Arterial's Peak Direction	4	5	1.67	1.33
SuperStreet Coordination	2	5	2	2

Delay due to Mid-Segment Sources

Travel along a segment can be exposed to a variety of conditions that cause drivers to slow down and thus cause delay away from any intersections or access points. These mid-segment congestion points can increase the segment running time in ways not accounted for in the methodology. For example, pedestrians crossing at a mid-segment crosswalk, vehicles maneuvering into or out of an on-street parking space, double-parked vehicles blocking a lane, or vehicles in a dropped lane that are merging into the adjacent lane, all can insert delays into the mid-segment travel. However, there is not enough research on this effect, and even the HCM 2010 does not include any procedure for estimating this type of delay. The adjustment is merely a placeholder for analysts and the HCM recommends the use of zero as a default value. Accordingly, the project team proposes to use "*zero*" as a default value, unless the user does have any field information suggesting otherwise.

Turn Vehicle Percentage

The number of turning vehicles or the turning vehicle percentage is one of the key data items needed to estimate intersection and arterial level performance. In general, turning vehicle percentages for each approach are collected by field data collection. However, field data collection requires man power and time. As such, a default value or estimation method would greatly simplify data requirements and as such arterial performance estimation.

However, it is difficult to create a default turning movement proportion, since each intersection has different characteristics. Furthermore, even the same intersection has different turning movement percentage by the time of day. Therefore, it is better to develop a generalized method to estimate turning movement percentages, rather than a fixed default distribution. The resulting percentages can then be multiplied by an approach volume estimate to arrive at an estimate of turning volumes.

The suggested method is fundamentally based on the gravity model, which is often used in transportation planning applications to perform trip distribution between zones. It generally assumes that the trips produced by and attracted to a zone are proportional to the size of that zone (measured in the total number of attractions and productions). This approach is modified here to treat an intersection as a small system of four origin and destination zones, signified by the four approaches to the intersection. In a simplification of the gravity model, the method does not use any friction factors (all set equal to 1.0). The friction factor is often used in travel demand models to describe difference in travel times between zonal pairs (for example as an inverse of travel distance), which does not apply in this intersection application. Similarly, the method (initially) assumes that all socioeconomic factors (i.e. factors making certain zonal combinations more attractive than others) are balanced and equal to 1.0. The resulting simplified gravity model application then becomes:

$$\text{Gravity model } T_{ij} = \frac{A_j F_{ij} K_{ij}}{\sum A_j F_{ij} K_{ij}} * P_i$$

$$\text{Simplified Gravity model } T_{ij} = \frac{A_j}{\sum A_j} * P_i$$

Where:

T_{ij} = trip production at I and attracted at j

P_i = total trip production at i

A_j = total trip attraction at j

F_{ij} = friction factor between zones i and j.

K_{ij} = a socioeconomic adjustment factor for zone pair ij

The productions and attractions in above formulation are equal to the total traffic entering and exiting each approach to the intersection. They can take the form of hourly volume counts, or estimated from daily volumes, which are generally more easily obtained for arterial streets.

In the ARTVAL computational engine, the approach AADT is one of the few required user inputs, and will be used in the turning movement estimation. The research team further provided hourly demand default proportion as presented above, so that each intersection approach demand can be calculated. Therefore, intersection turning movement can be estimated from the above simplified gravity model.

Exhibit 64 shows the suggested intersection turning movement estimation method flow. The suggested method was tested for one 106 different field turning movement data. For the statistical model test, a Weighted Mean Absolute Percentage Error (WMAPE) statistic was used.

$$M_i = \frac{\{\sum_{t=1}^{n} \left|\frac{A_t - F_t}{A_t}\right| * 100 * A_t\}}{\sum_{t=1}^{n} A_t}$$

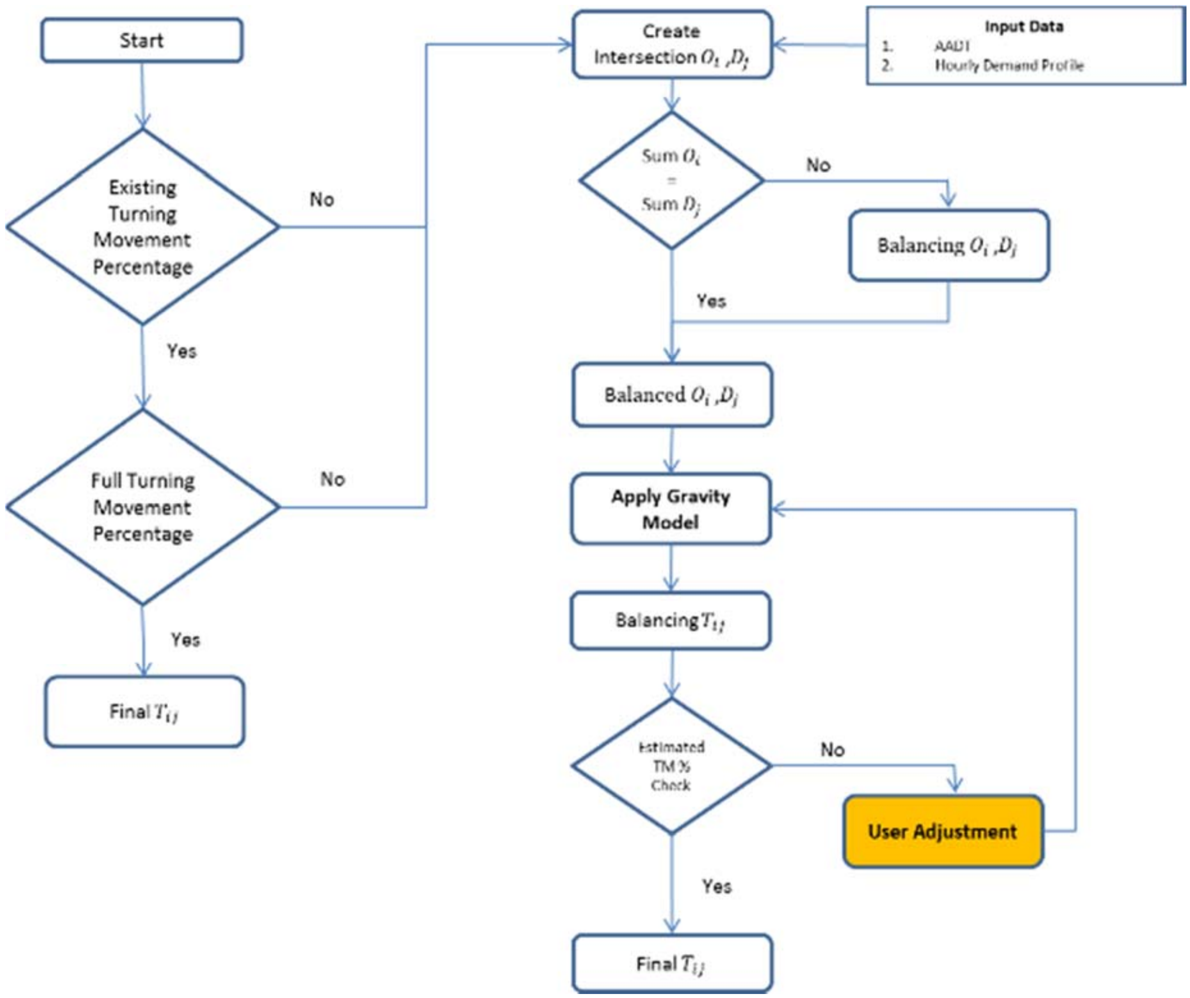
Where:

A_t = Actual count for approach movement t

F_t = Estimated count for approach movement t

M_i = Weighted Mean Absolute Percentage Error for intersection i

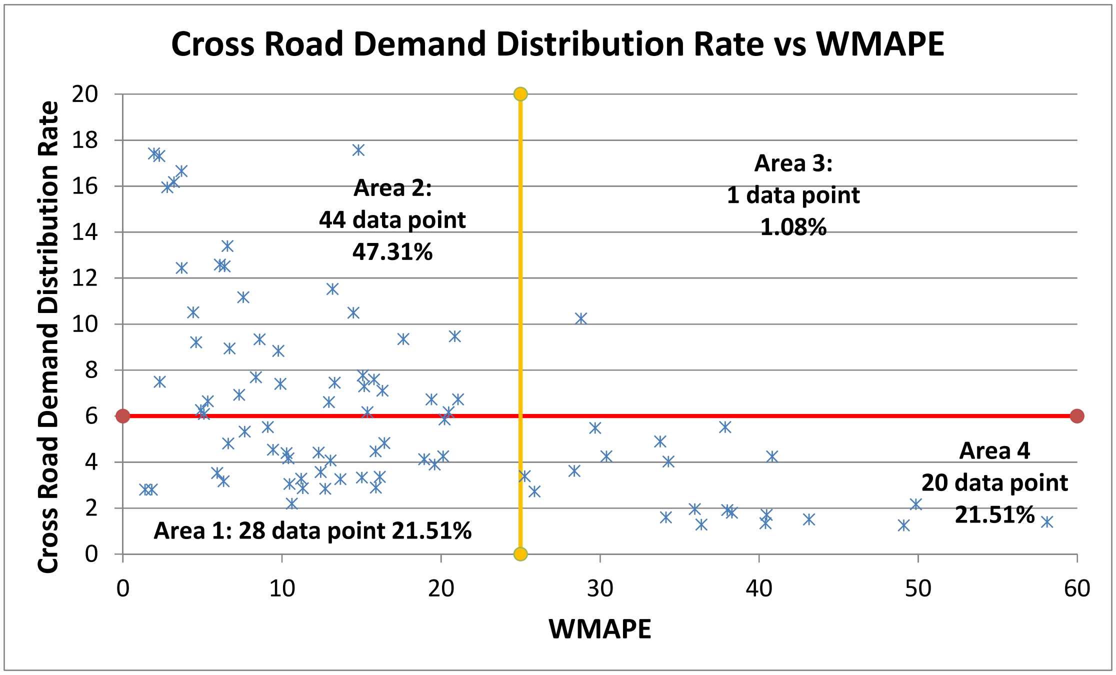
Exhibit 64: Gravity Model Estimation Process



After removing some observations, a remaining 93 field intersections turning movement data were divided by two different groups, which are balanced and unbalanced approach demand group. In this case, "balanced" refers to the approach AADT values of the two intersecting streets being approximately equal. In contrary, an "unbalanced" intersection is a more typical intersection of a major arterial and a minor street with significantly lower volume.

Among the 93 data sets, 48 data were categorized in the balanced intersection group and 45 data sets in the unbalanced intersection group. Exhibit 65 shows the model test result. The two groups' WMAPE are 9.10% and 22.61% for unbalanced and balanced intersection respectively.

Exhibit 65: Two Peak Flow Pattern Hourly Directional Demand Plot



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6. Application

This section describes the application of the methodology and the ARTVAL engine to a real-world case study. Results are also compared to other analysis tools available for urban street analysis. Specifically, two macroscopic modeling package chosen for evaluation and comparison to ARTVAL were Synchro and Vistro. The software packages were compared using a variety of criteria, including data requirement and relevance of performance measure reported in the output.

Synchro and Vistro were selected for evaluation as they represent the most commonly used traffic analysis packages in the U.S. These traffic simulation package can evaluate intersection level of service and optimize signal timing using industry standard methodologies. These traffic analysis packages use fundamental traffic flow, speed, and density relationships to estimate network system performance. VISTRO claims to be directly based on the HCM urban street methodology (the "full" method, as opposed to the quick estimation method used in ARTVAL), while Synchro uses its own adaptation of the HCM method.

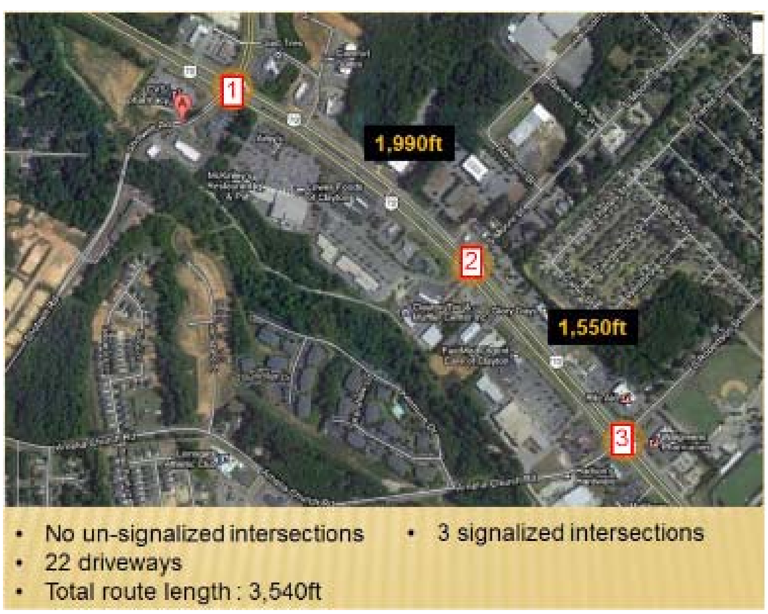
Test Site

To provide a reasonable range of traffic and geometric condition, it was decided to select an arterial segment for verification process in this study. The criteria used to select the study area included:

- Segment that is located between three signalized intersections;
- Segment that experiences moderate to heavy road volumes; and
- Segment with relatively short spacing between the two signalized intersections.

Based on these criteria, a suitable case study was identified as US-70 Business route in Clayton, NC. The selected arterial segment has a total route length of 3,540 feet with three signalized intersections along the arterial. Aerial photo are shown in Exhibit 66.

Exhibit 66: Aerial photo for the selected site



Comparison

Every model, no matter how carefully coded, will require validation and calibration to ensure its outputs are meaningful. The traffic volume requirement for comparison were obtained from local data, and lane geometry was gathered from aerial photos. Once the data were collected and the network were calibrated, each was run with multiple volume sensitivity inputs, and the results were analyzed with respect to key performance measures. The primary performance measures selected for comparison in this study included control delay and effective green time over cycle time ratio. The focus of this study was to compare these outputs for each of models and evaluate how well ARTVAL compares with these other tools.

The traffic volume collected from the study area were input into Synchro and the network were created for base condition. Twenty scenarios were defined for verification, which are the combinations of five critical demand over capacity ratio (d/c) and four cycle lengths (90, 120, 150, and 180). In this study, several OD demand multipliers were generated and obtained from ARTVAL (0.753, 1.004, 1.255, 1.506, and 1.575) based on the demand over capacity ratio of the network and applied to an existing OD demand matrix. The efficiency of the network is defined if the maximum origin-destination (OD) demand multiplier is larger than 1.

The adjusted volumes and the cycle lengths were then used for each movement at all intersections to compute performance measures for each model. For estimating the proportion of arrivals during the green and progression quality of corridor, the use of three different arrival types were proposed for this research (1, 3, and 5). The signal timing plans were exported from Synchro and imported to Vistro where the arrival types were added. The new data were generated for each scenario and control delay, effective green to cycle length ratio, and platoon ratio were recorded. The output results were recorded for eastbound through (phase 2) and westbound through movements (phase 6). The results of the simulation comparison are summarized in Exhibit 67.

Exhibit 67: Comparison Results for three models

ARTVAL ESTIMATES						
CONTROL DELAY	Cycle 90					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	15.91	13.48	7.75	8.84	8.71	15.73
DM_1.004	13.84	10.83	6.78	8.22	7.71	16.08
DM_1.255	13.45	11.54	4.91	6.38	3.62	11.72
DM_1.506	13.06	11.67	4.43	6.26	3.93	14.28
DM_1.757	17.01	21.82	4.85	7.63	4.29	20.26

ARTVAL ESTIMATES						
CONTROL DELAY	Cycle 120					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	18.78	15.06	5.47	6.83	7.60	15.93
DM_1.004	18.23	14.11	5.35	6.46	6.36	15.94
DM_1.255	15.55	12.27	4.20	5.44	3.21	12.88
DM_1.506	15.68	12.13	1.83	3.15	1.42	11.34
DM_1.757	16.57	14.18	1.62	2.54	1.21	13.34

ARTVAL ESTIMATES						
CONTROL DELAY	Cycle 150					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	21.93	17.01	4.62	5.27	6.95	16.59
DM_1.004	20.35	14.74	3.86	4.67	6.11	17.25
DM_1.255	17.94	13.41	3.10	4.02	2.56	13.51
DM_1.506	18.03	14.16	0.61	0.90	0.59	11.53
DM_1.757	19.03	16.47	-0.36	-0.51	-0.04	12.38

ARTVAL ESTIMATES						
CONTROL DELAY	Cycle 180					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	25.80	19.79	4.59	5.23	6.51	17.47
DM_1.004	23.89	18.23	3.12	3.77	5.54	18.01
DM_1.255	21.01	15.41	2.80	3.63	2.76	15.63
DM_1.506	20.11	14.94	0.46	0.69	0.44	13.23
DM_1.757	21.20	17.31	-1.02	-1.51	-0.31	14.14

VISTRO ESTIMATES						
CONTROL DELAY	Cycle 90					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	13.87	10.61	5.62	6.67	6.37	12.63
DM_1.004	11.71	8.51	4.57	6.11	5.82	13.22
DM_1.255	11.89	9.82	3.16	5.14	2.20	9.17
DM_1.506	11.51	11.85	2.68	6.70	2.50	11.36
DM_1.757	15.25	67.07	3.09	27.21	2.91	15.59

VISTRO ESTIMATES						
CONTROL DELAY	Cycle 120					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	16.99	12.34	3.95	4.56	5.76	13.46
DM_1.004	16.23	12.07	3.47	4.44	4.43	13.03
DM_1.255	14.33	10.57	2.68	4.11	1.88	10.33
DM_1.506	14.14	12.54	0.91	2.29	0.61	8.83
DM_1.757	15.20	28.97	1.49	5.71	1.51	10.59

VISTRO ESTIMATES						
CONTROL DELAY	Cycle 150					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	20.16	14.16	3.17	3.61	5.05	14.04
DM_1.004	18.48	12.74	2.16	2.71	4.25	14.40
DM_1.255	16.84	11.55	1.80	2.70	1.41	11.11
DM_1.506	16.80	13.61	1.40	2.63	1.44	9.42
DM_1.757	18.02	27.94	1.44	3.87	1.52	9.45

VISTRO ESTIMATES						
CONTROL DELAY	Cycle 180					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	25.28	19.19	3.06	3.77	5.22	17.47
DM_1.004	23.80	18.90	1.86	2.74	4.49	18.53
DM_1.255	21.16	17.85	1.55	3.29	1.69	15.57
DM_1.506	20.86	28.17	1.76	5.14	1.81	13.31
DM_1.757	20.28	25.62	1.58	3.99	1.69	10.93

SYNCHRO ESTIMATES						
CONTROL DELAY	Cycle 90					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	13.30	3.00	13.30	5.10	8.10	11.30
DM_1.004	11.00	2.40	13.30	4.70	8.00	11.20
DM_1.255	10.90	6.90	6.70	5.50	4.90	7.20
DM_1.506	10.30	6.60	6.00	5.10	4.90	8.10
DM_1.757	13.20	10.10	6.80	5.70	5.30	9.10

SYNCHRO ESTIMATES						
CONTROL DELAY	Cycle 120					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	16.30	6.50	6.90	4.90	6.10	12.10
DM_1.004	15.20	8.40	6.60	3.50	6.60	11.10
DM_1.255	13.10	7.60	5.90	6.10	5.20	8.20
DM_1.506	12.70	9.10	4.20	6.80	1.40	6.40
DM_1.757	13.20	10.80	3.70	5.70	3.50	6.80

SYNCHRO ESTIMATES						
CONTROL DELAY	Cycle 150					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	19.30	8.50	7.10	9.20	4.10	12.70
DM_1.004	17.30	7.80	6.20	8.30	4.50	12.30
DM_1.255	15.50	9.40	5.90	5.80	5.40	8.90
DM_1.506	15.10	12.10	3.90	4.60	4.10	6.90
DM_1.757	15.70	14.50	3.20	4.20	3.50	6.10

SYNCHRO ESTIMATES						
CONTROL DELAY	Cycle 180					
	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	23.70	10.20	7.90	9.10	5.90	15.00
DM_1.004	21.90	11.50	6.90	8.70	5.70	14.90
DM_1.255	19.00	12.30	6.50	8.50	5.00	11.50
DM_1.506	18.30	15.40	4.60	6.90	3.70	8.70
DM_1.757	17.70	16.40	3.80	4.60	3.80	7.20

To better understand the results obtained by all three tools, all values were normalized so we could see how well they correspond to each other. The ideal normalized difference value should be close to zero as possible (0.2 is 20 present difference). The comparison was performed three team, between two sets of tools each time.

As seen in table 2, some of the value has lager difference compare to ideal result. In general, a positive difference is shown as a blue bar, while a negative difference is shown using a red bar. By looking at the results, it seems that ARTVAL output estimation result is between Vistro and Synchro results. Any significant different were related primarily to different capability and features among the models.

Exhibit 68: Normalized Differences between tools

Normalized Difference (Artval-Vistro)/Vistro						
	Cycle 90					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.15	0.27	0.38	0.33	0.37	0.25
DM_1.004	0.18	0.27	0.48	0.34	0.32	0.22
DM_1.255	0.13	0.18	0.56	0.24	0.64	0.28
DM_1.506	0.13	-0.02	0.65	-0.07	0.57	0.26
DM_1.757	0.12	-0.67	0.57	-0.72	0.47	0.30

Normalized Difference (Artval-Vistro)/Vistro						
	Cycle 120					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.11	0.22	0.39	0.50	0.32	0.18
DM_1.004	0.12	0.17	0.54	0.46	0.44	0.22
DM_1.255	0.09	0.16	0.57	0.32	0.71	0.25
DM_1.506	0.11	-0.03	1.01	0.37	1.32	0.28
DM_1.757	0.09	-0.51	0.09	-0.55	-0.20	0.26

Normalized Difference (Artval-Vistro)/Vistro						
	Cycle 150					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.09	0.20	0.46	0.46	0.38	0.18
DM_1.004	0.10	0.16	0.79	0.72	0.44	0.20
DM_1.255	0.07	0.16	0.72	0.49	0.82	0.22
DM_1.506	0.07	0.04	-0.56	-0.66	-0.59	0.22
DM_1.757	0.06	-0.41	-1.25	-1.13	-1.03	0.31

Normalized Difference (Artval-Vistro)/Vistro						
	Cycle 180					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.02	0.03	0.50	0.39	0.25	0.00
DM_1.004	0.00	-0.04	0.68	0.38	0.23	-0.03
DM_1.255	-0.01	-0.14	0.81	0.10	0.63	0.00
DM_1.506	-0.04	-0.47	-0.74	-0.87	-0.76	-0.01
DM_1.757	0.05	-0.32	-1.64	-1.38	-1.18	0.29

Normalized Difference (Artval-Synchro)/Synchro						
	Cycle 90					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.20	3.49	-0.42	0.73	0.08	0.39
DM_1.004	0.26	3.51	-0.49	0.75	-0.04	0.44
DM_1.255	0.23	0.67	-0.27	0.16	-0.26	0.63
DM_1.506	0.27	0.77	-0.26	0.23	-0.20	0.76
DM_1.757	0.29	1.16	-0.29	0.34	-0.19	1.23

Normalized Difference (Artval-Synchro)/Synchro						
	Cycle 120					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.15	1.32	-0.21	0.39	0.25	0.32
DM_1.004	0.20	0.68	-0.19	0.85	-0.04	0.44
DM_1.255	0.19	0.61	-0.29	-0.11	-0.38	0.57
DM_1.506	0.23	0.33	-0.57	-0.54	0.01	0.77
DM_1.757	0.26	0.31	-0.56	-0.55	-0.66	0.96

Normalized Difference (Artval-Synchro)/Synchro						
	Cycle 150					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.14	1.00	-0.35	-0.43	0.69	0.31
DM_1.004	0.18	0.89	-0.38	-0.44	0.36	0.40
DM_1.255	0.16	0.43	-0.48	-0.31	-0.53	0.52
DM_1.506	0.19	0.17	-0.84	-0.80	-0.86	0.67
DM_1.757	0.21	0.14	-1.11	-1.12	-1.01	1.03

Normalized Difference (Artval-Synchro)/Synchro						
	Cycle 180					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.09	0.94	-0.42	-0.42	0.10	0.16
DM_1.004	0.09	0.58	-0.55	-0.57	-0.03	0.21
DM_1.255	0.11	0.25	-0.57	-0.57	-0.45	0.36
DM_1.506	0.10	-0.03	-0.90	-0.90	-0.88	0.52
DM_1.757	0.20	0.06	-1.27	-1.33	-1.08	0.96

Normalized Difference (Synchro-Vistro)/Vistro						
	Cycle 150					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.04	0.67	-0.55	-0.61	0.23	0.11
DM_1.004	0.07	0.63	-0.65	-0.67	-0.06	0.17
DM_1.255	0.09	0.23	-0.69	-0.53	-0.74	0.25
DM_1.506	0.11	0.12	-0.64	-0.43	-0.65	0.37
DM_1.757	0.15	0.93	-0.55	-0.08	-0.57	0.55

Normalized Difference (Synchro-Vistro)/Vistro						
	Cycle 180					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.07	0.88	-0.61	-0.59	-0.12	0.16
DM_1.004	0.09	0.64	-0.73	-0.69	-0.21	0.24
DM_1.255	0.11	0.45	-0.76	-0.61	-0.66	0.35
DM_1.506	0.14	0.83	-0.62	-0.26	-0.51	0.53
DM_1.757	0.15	0.56	-0.58	-0.13	-0.56	0.52

Normalized Difference (Synchro-Vistro)/Vistro						
	Cycle 90					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.04	2.54	-0.58	0.31	-0.21	0.12
DM_1.004	0.06	2.55	-0.66	0.30	-0.27	0.18
DM_1.255	0.09	0.42	-0.53	-0.07	-0.55	0.27
DM_1.506	0.12	0.80	-0.55	0.31	-0.49	0.40
DM_1.757	0.16	5.64	-0.55	3.77	-0.45	0.71

Normalized Difference (Synchro-Vistro)/Vistro						
	Cycle 120					
CONTROL DELAY	Intersection # 1		Intersection # 2		Intersection #3	
	Ø2	Ø6	Ø2	Ø6	Ø2	Ø6
DM_0.753	0.04	0.90	-0.43	-0.07	-0.06	0.11
DM_1.004	0.07	0.44	-0.47	0.27	-0.33	0.17
DM_1.255	0.09	0.39	-0.55	-0.33	-0.64	0.26
DM_1.506	0.11	0.38	-0.78	-0.66	-0.56	0.38
DM_1.757	0.15	1.68	-0.60	0.00	-0.57	0.56

At the end, three was no ranking of best or worst software. All packages were found to perform reasonably well, but with limitations that should be understood prior to selecting one for network evaluation. ARTVAL was found to be easiest of the model to use and its coding time is significantly shorter than the other two models. Overall, ARTVAL provides an easy to use solutions using HCM quick estimation methods for arterial segment, and requires less input that can make it an increasingly popular choice among traffic professionals and can be compatible with other traffic simulation software.

7. Conclusions and Recommendations

Summary

The research developed a methodology for quantifying delay and user cost impacts of arterial work zones in North Carolina in an analytical framework, supported by NC-specific empirical performance data of arterial work zones. The methodology developed in this project was implemented in a software tool, ARTVAL, which can be used directly for in-house analyses of these types of work zones to assure seamless technology transfer of these research products.

This research presented a statistical approach to model saturation headway at signalized intersection work zones based on work intensity, pavement condition, ledge presence, turn percentages from shared lanes, and the number of closed exclusive turn lanes. The saturation headway, work zone configuration, and prevailing conditions data were collected in North Carolina at six different intersections (representing seven intersection approaches), yielding a total of 19 unique work zone configurations and more than 4,600 headway observations.

The project team fitted four models, with three of the models applying traditional multiple linear regression at three different aggregation levels: a) cycle level aggregation, b) 15 minute aggregation, and c) full aggregation by work zone configuration. For the fourth model, the team used a path analysis based regression approach with a cycle level aggregation that allowed the inclusion of the 2010 HCM default values to adjust saturation headway based on right turn and left turn percentages.

All four models gave good fit and had statistically significant independent variables with logical signs. Overall, the project team recommends adopting the traditional regression model with 15-minute aggregation level, since it has a good statistical fit and is consistent with the HCM analysis period. However, if left turn and right turn percentages are significant and it is important to account for their impacts separately, the analyst may consider using the model based on path-analysis as it allows using the 2010 HCM recommended values to adjust for right and left turn percentages.

In addition to the headway model, the team developed default values needed to apply the analysis framework for urban streets analysis. The framework uses two quick estimation methods available in the Highway Capacity Manual: Quick Estimation Method for Signal Timing (QEM-ST) and Quick Estimation Method for Urban Streets (QEM-US). To facilitate the application of these methods, guidance was given on six specific input variables, including: (1) hourly directional demand distribution, (2) total delay due to turns into mid-segment access points, (3) lost time per cycle, (4) proportion of arrivals during green, (5) delay due to mid-segment sources, and (6) turning vehicle percentage estimation.

Of these defaults, the turning movement estimation method represents the most significant contribution. The method allows the estimation of turning percentages using a gravity-model approach, which was tested using over 100 intersection turning movement counts and corresponding AADTs from cities in North Carolina.

All models and methods have been implemented into the ARTVAL computational engine. The engine is based on the Microsoft Excel/Visual Basic platform, and allows estimation of urban street performance in a “data poor” and planning-level context. If available, more detailed operational data can be substituted in the engine, making the tool very versatile and user friendly. The tool is customized to allow for evaluation of arterial work zone impacts, but also functions well as a non-work zone tool due to its ability to evaluate an extended 24-hour period. This sets the tool apart from more traditional tools that focus on the analysis of the peak 15 minutes. The ARTVAL tool has been compared to two commercially available software tools and performed reasonably well, but with a much reduced data entry burden compared to these other tools.

Limitations

The data collected in this project were obtained from six intersections in North Carolina with and without intersection work zones. The reader should be cautious when generalizing the results of this work to locations outside of North Carolina, or locations significantly different than the triangle region. Furthermore, the research team was unable to identify any congested mid-segment work zones not significantly impacted by the operations of adjacent intersections. As such, no models for mid-segment work zones are presented.

The data collection purposely excluded cycles with heavy vehicles to arrive at a clean model to describe the passenger-car only capacity of work zones. It is assumed that the effect of heavy vehicles in work zones is proportional to the effect at non-work zone intersections, but this has not been tested or studied in this research.

Several other defaults were developed in this research, and these approaches are subject to some limitations in sample size and assumptions as documented in this report.

The developed software tool, ARTVAL, was tested against one case study application, but broader testing is needed to assure its validity for application to a broader set of work zone and non-work zone intersection.

Future Research

Future research should focus on additional testing and verification of the ARTVAL tool, to assure it can broadly be applied to intersections across North Carolina and beyond. That testing should specifically focus on the default values and assumptions in the model described in this report. While it is not expected that any model performs perfectly in all applications, it is important that the user community develops an understanding for how and when to best apply a particular tool.

In additional future work, added data collection at mid-segment work zones is recommended, as well as specific consideration of heavy vehicle effects at work zones. Furthermore, work zones are likely to interact with other capacity-reducing effects due to incidents and weather, which have not been evaluated in this research. It is recommended that future research more closely investigate these effects.

Finally, with completion of this project, NCDOT now has the ability to evaluate arterial streets, in addition to freeway facilities (method and engine developed in previous work). A logical extension of this work is the development of a network-level methodology and associated tool that allows the evaluation of the interaction of these two facility types.

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