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Rafael I. Perez-Cartagena

Andrzej P. Tarko tarko@purdue.edu

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Final Report

FHWA/IN/JTRP-2004/29

PREDICTING TRAFFIC CONDITIONS AT INDIANA SIGNALIZED INTERSECTIONS

By

Rafael I. Perez-Cartagena Graduate Research Assistant

and

Andrew Tarko Associate Professor of Civil Engineering

> School of Civil Engineering Purdue University

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Conducted in Cooperation with the Indiana Department of Transportation and the U.S. Department of Transportation Federal Highway Administration

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Indiana Department of Transportation or the Federal Highway Administration at the time of publication. The report does not constitute a standard, specification, or regulation.

Purdue University West Lafayette Indiana December 2004



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December 2004 Final Report

Predicting Traffic Conditions at Indiana Signalized Intersections

Introduction

The Highway Capacity Manual (HCM) recommends the use of locally measured capacity parameters for the design of signalized intersections. Currently, national default capacity parameters are widely used for their convenience and because it is difficult to measure the parameters. This research sought to determine

Findings

The capacity parameters investigated include the base saturation flow rate, start-up lost time, green time extension, and heavy vehicle equivalency factor. The state average capacity parameter values for Indiana are comparable to the HCM recommended default values. Also, peak hour factor (PHF) was calculated for a number of intersections.

From the estimated parameters, the base saturation flow rate and PHF had a high variability across locations. Population size and lane position in a lane group were found to have effect on the saturation flow rate while time of day, population size, and volume had a considerable effect on PHF.

Implementation

The research report includes a stand-alone document that provides the Indiana values of capacity parameters and an equation to calculate the Peak Hour Factor. The local values are tabulated in a convenient format and can be used in capacity and delay predictions with the Highway Capacity Software, Synchro, and CORSIM. Indiana-specific capacity parameters. Sitespecific characteristics were investigated to determine the factors that influence parameter variability. Improvement in the quality of delay predictions was demonstrated when using the developed Indiana parameter values in lieu of the default values.

Control delay was calculated to evaluate the benefit of using local capacity parameter values. For this task the Highway Capacity Software (HCS 2000) was used. The capacity parameters evaluated were base saturation flow rate, start-up lost time, and green time extension. The delay prediction using the local capacity parameters on average had a lower mean error when compared with delay predictions using the default parameter values. Also, the local parameters produced lower variability compared to the default parameters. PHF should be predicted whenever traffic counts are not available.

After approval, the document will be circulated among Indiana Department of Transportation operation, design, and planning units. The circular will also be provided to the Metropolitan Planning Organizations and other local agencies.

Contacts

For more information:

Prof. Andrew Tarko Principal Investigator School of Civil Engineering Purdue University West Lafayette IN 47907 Phone: (765) 494-5027 Fax: (765) 496-7996 E-mail: tarko@ecn.purdue.edu

Indiana Department of Transportation

Division of Research 1205 Montgomery Street P.O. Box 2279 West Lafayette, IN 47906 Phone: (765) 463-1521 Fax: (765) 497-1665

Purdue University

Joint Transportation Research Program School of Civil Engineering West Lafayette, IN 47907-1284 Phone: (765) 494-9310 Fax: (765) 496-7996 E-mail: jtrp@ecn.purdue.edu http://www.purdue.edu/jtrp

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| factors lost times and other parameters F | Recause these parameter | s provide default value | ure the use of the defau | it values is a common | | | | |
| practice and it can cause significant bias in | the results if the actua | l local values are differ | rent. The main objective | of this research was to | | | | |
| investigate the variability of capacity and | delay parameters used i | n engineering tools for | predicting delays and for | or determining level of | | | | |
| service (LOS) at signalized intersections in | n Indiana. The variabili | ty investigation produc | ed Indiana default value | s that better reflect the | | | | |
| behavior of Indiana drivers at signalized in | tersection than the defa | ult values recommende | d in the existing engineer | ring tools and based on | | | | |
| arbitrary assumptions or nationwide studies | | | | | | | | |
| Traffic was observed at 19 intersections. A | procedure combining | Highway Capacity Ma | nual (HCM) methodolog | v and linear regression | | | | |
| was used to estimate the capacity parameter | ers. Variability was inv | estigated using weighte | ed linear regression. Alth | ough the state-average | | | | |
| values for Indiana are similar to the HC | M-recommended value | es, the base saturation | flow rate and PHF ext | hibit strong variability | | | | |
| depending on population size and other loc | al factors. On average, | the saturation flow rate | in medium towns was 89 | % and in small towns it | | | | |
| was 21% lower than large towns. These | results confirm the fi | ndings for Florida and | l particularly for Kentu | cky. A table with the | | | | |
| were very close to the HCM values. The po | presented. The start-up pulation size morning/ | afternoon rush hour and | d volume considerably af | fected the PHF | | | | |
| were very crose to the field values. The po | pulation size, morning, | arternoon rush nour, un | a volume consideratily a | | | | | |
| The improvement of delay estimation wh | en using local capacit | y parameters instead o | of default values was as | sessed using Highway | | | | |
| Capacity Software 2000 (HCS2000) by con | mparing the results to t | he measured delays. The | ne average mean error fo | r the default parameter | | | | |
| values was slightly higher than for local val | ues. Also, the standard | deviation of the estimat | e for local parameter val | ues was also lower. | | | | |
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IMPLEMENTATION REPORT

The research report includes a stand-alone document that provides the Indiana values of capacity parameters and an equation to calculate the peak hour factor. The local values are tabulated in a convenient format and can be used in capacity and delay predictions with the Highway Capacity Software, Synchro, and CORSIM.

The implementation does not require additional funds. After approval, the document will be circulated among Indiana Department of Transportation operation, design, and planning units including the INDOT Environment, Planning and Engineering Division; Design Division; Operations Support Division; and the INDOT Districts. The circular will also be provided to the Federal Highway Administration (Indiana Division Office), Metropolitan Planning Organizations and other local agencies. It is recommended that the periodic training sessions conducted for INDOT staff on HCM include the results of this study.

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CHAPTER 1. INTRODUCTION

Traffic signals are commonly used at Indiana intersections to control movements. These signals generally reduce overall delays but increase traffic interruptions. A primary means to improve traffic operation, they are also a source of delays, so adequate use and design of signalized intersections is imperative for satisfactory performance.

The Indiana Department of Transportation (INDOT) uses the Highway Capacity Software 2000 (HCS 2000), SYNCHRO, and CORSIM, among other tools, to evaluate the performance of signalized intersections. The results obtained with these methods become the basis for decisions regarding the intersection geometry and the type of control. Inaccurate calculation results may lead to non-optimal design and planning decisions.

The Highway Capacity Manual (HCM) advises its users: "The methodologies in the Highway Capacity Manual are based on calibrated national average traffic characteristics observed over a range of facilities. Observations of these characteristics at specific locations will vary somewhat from national averages because of unique features." The HCM also recommends, "The variation in the data and the importance of prevailing conditions suggest that local data collection be performed to determine saturation flow rates and lost times, which can lead to more accurate computations."

All of the methods for predicting traffic conditions at signalized intersections provide default values for saturation flow rates and their adjustment factors, lost times, and other parameters. These parameters are difficult to measure. Therefore, use of the default values is a common practice and this is a source of possibly significant bias in the results if the local parameter values are different.

The high sensitivity of delays and level of service (LOS) results to inaccurate saturation flow and other inputs raises these questions: How much do the local capacity parameters vary from the default values? How much can the errors in the predictions be improved when the local parameters are used in calculations? Local parameters values for Indiana will be estimated to address these questions, and will be proposed for Indiana if justified with the research results.

An accuracy analysis of the default prediction methods for Indiana, to our knowledge, has not been conducted. Research is therefore needed to determine the Indiana values of basic saturation flow, lost times, and other selected capacity factors.

This research project will investigate Indiana-specific capacity and delay parameters used in engineering tools for predicting delays and for determining the LOS at signalized intersections. This investigation aims to produce local default values that reflect the behavior of Indiana drivers at signalized intersections in a variety of settings. If justified, these values should replace the default values recommended in the existing engineering tools that are represent national average rather than local conditions. These parameter values should also reflect the intrastate variability caused by varying Indiana conditions, such as the level of urbanization.

The report includes eight chapters. The literature review first discusses the work done in the past and highlights findings relevant to this study. Then, the third chapter presents the research methodology, which includes five components: field data collection, data extraction, capacity parameter estimation, variability analysis, and method evaluation. Chapter 4 discusses the data collection process, including the planning, field data collection, and data extraction. This process required a great amount of time in order to select the locations, videotape the traffic and extract the values to estimate the capacity parameters.

The methodology adopted to estimate capacity parameters and the final values for the state of Indiana are summarized in the fifth chapter. Then, the variability analysis of the capacity parameters is discussed in Chapter 6, and the recommended values are summarized and evaluated for Indiana, in Chapter 7 The results are evaluated and presented. Finally, the conclusions are stated in the last chapter.

CHAPTER 2. LITERATURE REVIEW

2.1. Tools for Signalized Intersections Analysis

There are a number of methods supporting the design and evaluation of signalized intersections. INDOT uses HCS2000, CORSIM, and SYNCHRO, among other tools. The HCS2000 incorporates the capacity and level of service calculations present in the 2000 Highway Capacity Manual (TRB 2000). SYNCHRO, developed by Trafficware Inc., is a software package that is able to model and optimize traffic signal timings (Trafficware, 1998). SYNCHRO also has the ability to compute optimum intersection timings for intersection offsets as well as cycle lengths and phase splits. CORSIM, developed by the Federal Highway Administration (FHWA), is a simulation program that can be used to input traffic data, provide a common user interface, perform microscopic simulation, and animate a given network.

The results produced with these methods are commonly the basis for decisions regarding an intersection's geometry and the type of control. Inaccurate results of calculations may lead to inadequate design and planning decisions (Tarko and Tracz, 2000). A comparison of control delays estimated by four different methods, HCS2000, SYNCHRO, PASSER, and CORSIM, highlighted the similarities and differences of computed delays (Benekohal, et al. 2001). They determined that the differences between programs were not significant when precautions were taken. However, no comparison to the field delay was provided.

Past field and analytical evaluations of the models indicated that the obtained results might not match the values observed in the field. In a study conducted at five signalized intersections, delays were measured and then compared with the results generated by seven different methods, including HCS and SYNCHRO. It was concluded that the evaluated methods did not replicate the field-measured average delay accurately (Petraglia, 1999). Another study evaluated delay predictions along signalized arterial streets and found strong discrepancies between the calculated and observed values (Courage et al., 1995). Improvements were proposed to the delay equation and discrepancies were thus reduced. However, the HCS method continued to overestimate delays by 37 percent.

On a HCS users' survey, many professionals indicated having doubts about the results produced by the Highway Capacity Manual (Tarko and Praprut 2001). The chapter on signalized intersections caused the most comments. The respondents indicated that the results were sometimes unrealistic. Some of the responders believed that the inputs were not accurate while equations to calculate delays were sensitive to these inputs and amplified the inaccuracies.

2.2. Variability of Method Parameters

The methods mentioned above combine a number of variables present at each intersection to estimate capacity and delay. These variables include the volume, number

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of lanes, percentage of heavy vehicles, saturation flow rate, arrival type, and other traffic data provided by the user. Some of these input values vary across locations.

In practice, input values such as the base saturation flow rate and lost time are considered constant across locations. Input values to the methods that are considered constant across locations are referred to as parameters. Input values such as the base saturation flow rate and lost time are related to the service flow rates and capacity of the intersection. Other factors such as peak hour factor are related to the demand side of the analysis.

Even though capacity parameters are commonly used as constant values, past research supports the idea of site-specific factors having an impact on their value. This variability, if significant, should be considered in the methods previously mentioned, in order to better emulate traffic conditions. This research will investigate those factors and their impact, if any, on capacity parameters.

It has been found that the saturation headways in business areas is significantly lower than those in other areas, including residential, shopping, and recreational areas. Also, the saturation flow in the recreational areas was, on average, eight percent lower than anywhere else (Le, et. al. 2000). The same authors concluded, based on another study, that non-local drivers had significant impacts on the saturation flow rate (Zhou, et. al., 2000). They further concluded that when the non-local driver population increased, the saturation flow rate decreased as much as 19%. Tourists may respond more slowly to phase interchanges and have a longer car-following distance at signalized intersections compared to commuters. Also, drivers may be affected by complex environmental conditions in business areas while those in recreational and shopping areas may drive more casually.

The effect of the days of the week and the time of the day on saturation flow rates was analyzed in another study that determined there were no differences found between the day of the week and the time of day (Torbic and Elefteriadou 2000).

Bonneson and Messer (1997) concluded that traffic pressure, as quantified by traffic volume per cycle per lane, had a statistically significant effect on saturation flow rate. In general, the saturation flow rates of low-volume movements were lower than those of high-volume movements, which might be partly attributed to the differences in average queues.

In studies conducted between 1947 and 1979, the through-lane saturation flow rates of between 1,500 and 1,800 vehicles per hour of green (vphg) were reported. The most extensive database recently reported in the United States was collected in Kentucky in medium and smaller-sized communities. An average saturation flow rate of 1,650 vehicles per hour (vphg) was found for lanes with widths between 10 and 15 ft. and with approach grades between -3 and +3 percent.

Surveys were conducted in Kentucky in two communities with populations of more than 100,000 persons, three communities with populations of between 20,000 and 50,000 persons, and three communities with populations of less than 20,000 persons (K.R. Agent., et. al. 1982). Saturation flow rates in the cities with 20,000 to 50,000 persons were eight percent lower than in the largest city surveyed. Saturation flow rates in the cities with populations flow rates in the cities with populations flow rates in the cities with population flow rates in the cities with populations flow rates in the largest city surveyed. Saturation flow rates in the cities with populations less than 20,000 persons were 17 percent lower than those in the

7

largest city surveyed.

Zegeer (1986) investigated various factors of capacity affecting signalized intersections and concluded, based on the Kentucky data (K.R. Agent., et. al. 1982), that the saturation flow rate for the same conditions in different-size communities varied considerably within the range of 83 - 100 percent with high values for large urban areas.

The mentioned research studies support a hypothesis that significant part of the saturation flow variability can be explained with the local behavior of drivers. The range of this variability is wide enough to cause a considerable bias in the calculations of delay if a single default value is used in HCS, SYNCHRO, CORSIM, or any other tool.

2.3. Sensitivity Analysis

The HCM 2000 remarks the importance of the sensitivity analysis:

"Once one or more performance measures have been selected for use in reporting analysis results, decision-making can be improved by showing how the numerical values (or the letter grade for LOS) change when one or more of the assumed input values changes. For the decision-maker, it may be quite important to know how an assumed increase of 15 percent in future traffic volume (compared with the standard forecast volume) will affect delay and level of service at a signalized intersection."

The importance of accuracy in capacity analysis is noted in the HCM 2000:

"The limitations on accuracy and the validity of predictions for performance measures should be recognized when applying the results of the analysis. For instance, small differences between the values of performance measures for alternative design should not be assumed to be real (statistically significant) differences. Furthermore, if the predicted value for an MOE is near, but below, a critical threshold, it should be recognized that there is some probability that it will in fact be higher than predicted, and may exceed the critical threshold. The HCM user should recognize, therefore, that judgment is required when applying the results of HCM analyses. One basis for judgment should be a good understanding of the structure and basis of the models that are used in this manual."

The effect of inaccurate input values in LOS prediction has been investigated. Tarko and Tracz (2000) investigated the effect of even limited error in saturation flow on LOS estimates. Errors in saturation flow rates were attributed to temporal variance of the saturation flow causing measurement errors, omission of some capacity factors in models, and prediction bias in model variables. The authors found that inaccurate saturation flow rates strongly affected on the ability to correctly determine the LOS for unopposed streams. Furthermore, it was recommended that

"...an obvious need for frequent updates of predictive formulas for saturation flows and for a careful consideration of local conditions."

A comprehensive study by Dowling (1994), studied the effect of using default rather than measured values. It was found that the use of local values for the peak hour factor (PHF), saturation flow rate, and signal progression factor considerably reduced the errors in the delay estimates when the traffic stream was stronger that 85 percent of capacity.

Khatib and Kyte (2000) support this opinion. They investigated the sources of error and their impacts on level of service (LOS). One of the important sources of uncertainty was the input data propagated through to the final results and calculations. They found that errors in the input parameters were responsible for significant bias in the results when the analyzed intersections operated at high delays. One example given was the traffic volume forecast which is subject to a high degree of uncertainty. Driver behavior, as measured by

the saturation headway, is site specific, similar to the critical gap at unsignalized intersections.

2.4. Methods of Measurement

Several methods have been proposed and used by authors to estimate capacity parameters, mainly the saturation flow rate and lost times. The methods are described in chronological order.

Headway Method (Greenshields et al. 1947; TRB 1997): estimates the average time headway between the vehicles discharging from a queue as they pass the stop-line. The first several vehicles are skipped to avoid the effect of vehicles' inertia in the initial seconds of the green time. The saturation flow rate is calculated as a reciprocal of the mean headway.

TRL Method (TRRL 1963): vehicles are count during three saturated intervals of green. The saturation flow rate is calculated dividing the count of the middle interval over the length of the interval itself.

Regression Technique (Branston and Gipps 1981; Kimber et al. 1985; Stoke et al.1987): used to develop an equation involving the saturated green time, number of vehicles in various categories, and lost time. A regression analysis yields the saturation flow, the lost times, and the passenger car equivalents for vehicles other than passenger cars.

Several studies have suggested the use of a full-motion video recording to collect data, which would provide an accurate record of the data. In addition, the impact of special conditions can be considered. For example, in a certain signal cycle, when the discharge of the vehicles is impacted by a vehicle not moving for a long period of time, that data can be isolated and analyzed separately. It also allows for real-time analysis of the data. Also, any unusual event that may affect the saturation flow rate, such as buses, stalled vehicles, and unloading trucks can be identified. For example, Li and Prevedouros (2001) also conducted their field measurement using a video recorder.

Le, et al. (2000) suggested factors to be considered in the selection of study locations. The factors were designed to locate ideal intersections as described by a lack of adjustment factors for prevailing conditions.

Some methods of calculating saturation flow and lost times utilize the HCM2000 methods. These methods involve the use of tables and predetermined parameters, from which the saturation flow and lost times are calculated. Statistical analysis and regression analysis have also been used

2.5. Summary

Capacity at a signalized intersection operating under conditions different from the base conditions is affected by many traffic and geometry characteristics. Some of the characteristics influence capacity parameters such as vehicle headway and lost time. Local factors, such as area type and percent of non-local drivers, have been demonstrated to affect capacity at signalized intersections.

The varying capacity factors and the sensitivity of the intersection performance sometimes can explain the strong variability of delays and LOS, which is not fully grasped by the HCS2000, CORSIM and SYNCHRO tools. Overestimation of delay at signalized intersections can be minimized using local capacity parameters to account for driver behavior. Local values for saturation flow rate, lost time, and PHF improve delay-based LOS prediction. Also, the size of the community and the area type are factors that affect capacity and delay predictions.

Some of the sources of errors for delay prediction include the temporal variance of the saturation flow rate and the omission of some factors. A survey showed that HCS users have doubts on the estimation of delay, due to the sensitivity of the equations to the input values. The sensitivity of the equations amplifies the inaccuracies when incorrect input values are used.

The scope of this study is to estimate Indiana's capacity parameters values and compare them to the national default values, investigate factors that affect the variability of capacity parameters across locations, and evaluate the benefit of using the local values instead of the default values of capacity parameters when predicting delay at these intersections.

Previous studies have used video recording techniques to estimate the saturation flow rate at selected locations. When selected locations lack conditions where adjustment factors are needed, then base saturation flow rate can be measured.

CHAPTER 3. RESEARCH METHOD

The research includes five phases. In the first phase, field data collection, traffic at a number of signalized intersections is recorded during AM or PM rush hours using video recording equipment. Intersections are selected where conditions are close to the base conditions as defined by the HCM. Locations are classified by the population size of the community with the intersection.

In the second phase, data required in order to estimate capacity parameters is extracted from the videotapes. This task is performed using Excel-based software. The discharge time of vehicles in a queue and the number of vehicles per type are recorded. Also, signal timings and traffic volumes are collected for the analysis. The average control delay is estimated for lane groups according to the HCM methodology.

Capacity parameters for each lane are estimated in the third phase using data extracted in Phase Two. The estimation of capacity parameters is consistent with the HCM. The only departure from the HCM method is the use of linear regression to account for the effect of heavy vehicles in the saturated flow. Using the saturation vehicle headway estimated with regression, the saturation flow rate is calculated. Also, the start-up lost time, green time extension, and heavy vehicle equivalency factor are estimated, for straight ahead and left-turn flows.

The fourth phase investigates the variability of the capacity parameters. Site-specific

factors, such as population, road class, and area type, are considered in this analysis. Recommended capacity parameter values for the State of Indiana are obtained.

The fifth phase includes using HCS2000 and the local capacity parameters to evaluate the improvement of delay prediction. The first scenario consists of delay prediction using the default capacity parameters and then repeated using the recommended Indiana parameter values. Using the measured values, the predictions are compared and analyzed.

CHAPTER 4. DATA COLLECTION

The capacity parameters were measured at various locations across the state of Indiana. The data collection process was divided into three steps. First, intersections were selected and the field data collection was planned. This step also involved gathering preliminary information about the geometric and traffic characteristics of each location.

In the second step, field data collection, each intersection was visited and videotaped. Traffic discharging the intersection, traffic signals, and other important data, such as geometry, area type and posted speed limit, were recorded.

The last step of data extraction was performed in the Transportation Laboratory in the School of Civil Engineering at Purdue University and consisted of observing the videotapes and measuring a number of traffic events needed in order to estimate the capacity parameters.

4.1. Planning

Intersections were selected based on the following criteria: population, long queues, and base conditions. Communities of various population sizes were desirable to diversify the sample and to enable quantification of the effect of the population size on capacity. The Census Bureau Report for the year 2000 was used to determine the population of the

communities used in the study. Table 4.1 shows the population of the selected communities where traffic was recorded.

| County | City/Town | Population | | | | |
|------------|----------------|------------|--|--|--|--|
| Fountain | Attica | 3,491 | | | | |
| Parke | Rockville | 2,765 | | | | |
| Owen | Spencer | 2,508 | | | | |
| Hendricks | Avon | 6,248 (1) | | | | |
| Hamilton | Carmel | 37,733 (1) | | | | |
| Marion | Indianapolis | 781,870 | | | | |
| Jasper | Demotte | 2,482 | | | | |
| Howard | Kokomo | 46,113 | | | | |
| Tippecanoe | Lafayette | 56,397 | | | | |
| Madison | Pendleton | 3,100 | | | | |
| Hamilton | Westfield | 9,293 (1) | | | | |
| Tippecanoe | West Lafayette | 28,778 | | | | |
| White | Monticello | 5,723 | | | | |

Table 4.1 Communities Selected for Study

⁽¹⁾ Locations in close vicinity to Indianapolis

Table 4.2 shows the classification of the communities used in the study. Only Indianapolis was classified as a large population community. Those communities with a population larger than 20,000 persons were classified as medium-size communities. Small population is defined as communities with less than 20,000 persons. Cities in close vicinity of Indianapolis were classified as large urban areas due to the fact that most of the residents in these cities commute to work in Indianapolis, and their driving style can be aggressive and comparable to those of large urban areas. These cases include Avon, Carmel and Westfield.

| Population (hab.) | Size |
|-------------------|--------|
| ≤ 20,000 | Small |
| 20,000 - 100,000 | Medium |
| 100,000 ≤ | Large |

Table 4.2: Population Size Classification

Base saturation flow rate can be measured at intersections with long queues and lanes operating under base conditions (12-ft. lanes, no pedestrian flow, no parking maneuvers, no bus stops, and zero grade approach). Geometric, traffic, operational, and environmental conditions at the studied intersections should be as consistent with the requirements specified in the HCM2000 as possible. A low percentage of heavy vehicles is acceptable as it can be accounted for when estimating the base saturation flow rate. Intersections near business districts should be avoided due to the large flow of pedestrians. Intersections with irregular lane configurations should also be avoided due to their unknown effect on driver behavior. Shared lanes with low number of right turning vehicles are acceptable because the impact of weak right turns is negligible.

Twenty-one signalized intersections were selected for our study. Some of the intersections initially selected were dropped due to power cables near intersections, undesirable parking conditions, or absence of queued vehicles. Usually, signalized intersections within small communities use one lane for through, left, and right movements making base saturation flow rate difficult to measure without the effect of the

turning maneuvers.

4.2. Videotaping Traffic

Traffic at the locations then was recorded during the morning or afternoon rush hours. Only a few intersections were videotaped during the noon rush hour. The following section will present the equipment and procedure used to videotape traffic.

4.2.1. Equipment

The mobile traffic laboratory (MTL) shown in Figure 1-2.a was used to record traffic queues and signal heads. The MTL consists of a van equipped with a 45-ft. mast, a computer, a digital video recorder, and two cameras. Two additional cameras on tripods recorded the displayed signals. A multiplexer and a computer, Figure 1-2.b, mixed the signal from all four cameras. The digital video recorder has the capability to record and playback simultaneously the signal from the four cameras. This allows for relating in real-time discharging traffic with the state of the traffic signals.





Figure 1.2.a Mobile Traffic Laboratory

Figure 1.2.b Computers and Traffic Monitoring System

Figure 4.1: Field Data Collection Equipment

4.2.2. <u>Equipment Setup</u>

The equipment setup depended on the particular characteristics of each intersection. Weather conditions were a major factor when videotaping traffic. Conditions such as heavy rain or dense fog were avoided due to the impact on drivers' behavior; also, conditions such as thunderstorms and strong winds were considered hazardous and data collection was postponed.

With the MTL parked at one of the corners of the intersection, the mast was raised to the highest possible elevation and the tripods were positioned next to the MTL as shown in Figure 4.2. On occasion, the tripods with cameras were placed on the roof of the MTL to

avoid sight obstruction by heavy vehicles.



Figure 4.2: Schematic Diagram of Field Data Collection Equipment Set-up



Figure 4.3: Field Data Collection Screen View (US36 & Girls School Rd, Avon, IN)

Important site characteristics were documented at the location, including the road class, area type, median type, speed limit and presence of a right shoulder or curb. Also, the control type at the intersection was documented.

Each of the mast-mounted cameras could only be used for one approach to guarantee that the last vehicle in a queue for each approach appeared in the field of view. Seeing the end of the queue was needed for accurate estimation of the saturation flow rate and particularly delay.

As shown in Figure 4.2, only the signal heads opposite to the approach with the recorded traffic could be viewed. As an example, at the intersection of US 36 and Girls School Road in Figure 4.3, vehicles discharging from the westbound and southbound lanes were recorded while the signal heads recorded at this intersection displayed signals for the eastbound and northbound approaches. The lengths of the recorded signals and the known ring structure were sufficient to derive the lengths of not-seen phases. Numerical examples will be presented.

4.3. Extracting Data from Tapes

In the third phase of data collection, videotapes were watched and needed information was extracted. It was extremely important to follow the procedure carefully to minimize measurement errors. First, the vehicles were counted and time-measured while playing back the tapes. The data extraction was conducted on a lane-by-lane basis. Then, these measurements were used to calculate intermediate results and estimate capacity parameters.

4.3.1. <u>Intermediate Measurements</u>

Table 4.3 shows the quantities directly measured with the video images. Vehicles departing from a queue included those stopped on the approach and those that joined the queue before the stop-bar when the end of the queue was already moving. Cycles with at least five vehicles were used for the estimation of the saturation flow rate. Those cycles with exactly four vehicles were included for the estimation of start-up lost time. The first four vehicles in a queue heretofore will be referred to as the "first part of the queue," while those vehicles between the fifth and the last in queue will be referred to as "second part of the queue." Other important measurements taken were the length of the yellow and the all-red phases (clearance interval). The process can be summarized in the following steps:

- Step 1: Count the number of vehicles per type at the beginning of the green phase. Record the time when the green phase signal is displayed.
- *Step 2*: Count the number of heavy vehicles that exited among the first part of the queue. Record the time when the fourth vehicle exited the intersection.
- Step 3: Count the number of vehicles per type in the queue after the fourth vehicle. Record the time when the last vehicle in queue exited the intersection.
- Step 4: Count the number of vehicles, per type, exiting the intersection after the

end of the queue.

Step 5: Record the time when the yellow phase ends.

Step 6: In the case of an oversaturated cycle, count the number of vehicles that were in queue, but could not exit the intersection. This will be the residual queue for the cycle and will be added to the arriving vehicles for the next initial queue.

| Symbol | Description | | | | | | | |
|-------------------------|----------------------------------------------------------|--|--|--|--|--|--|--|
| T_G | Time when green phase starts | | | | | | | |
| T_4 | Time when 4 th vehicle exits the intersection | | | | | | | |
| T_q | Time when queue ends | | | | | | | |
| T_Y | Time when yellow signal ends | | | | | | | |
| q_i | Initial queue | | | | | | | |
| n_{hvl} | Number of heavy vehicles in the first part of queue | | | | | | | |
| n_{pc2} | Number of passenger vehicles in second part of the queue | | | | | | | |
| n _{hv2} | Number of heavy vehicles in second part of the queue | | | | | | | |
| <i>n_{pcnq}</i> | Number of passenger vehicles not in queue | | | | | | | |
| n _{hvnq} | Number of heavy vehicles not in queue | | | | | | | |

Table 4.3: Measurements Directly Obtained from the Video Tapes

The quantities presented in Table 4.3 were measured for every cycle. One-tenth of a second was considered sufficient precision for signal phases and discharge time. The queue discharge and the number of passenger and heavy vehicles in the queue were used to estimate the saturation flow rate. The discharge time of the first four vehicles was used to estimate the start-up lost time. These calculations are explained in the next section. The

values in Table 4.3 were used to calculate the additional values shown in Table 4.4 and needed to estimate the capacity parameters.

| Symbol | Equation | Description |
|-----------------|-----------------------|---------------------------------------------------------|
| t_4 | T_4 - T_G | Discharge time of the first part of queue |
| t_s | T_q - T_4 | Discharge time of the second part of queue |
| е | $T_q - (T_y - A)$ | Effective green time extension ⁽³⁾ |
| n_{pcl} | $4-n_{hvl}$ | Number of passenger vehicles in first part of the queue |
| n_{pcq} | $n_{pc1} + n_{pc2}$ | Total number of passenger vehicles in queue |
| n_{hvq} | $n_{hvq1} + n_{hvq2}$ | Total number of heavy vehicles in queue |
| n_{pc} | $n_{pcq} + n_{pcnq}$ | Total number of passenger vehicles during cycle |
| n _{hv} | $n_{hvq} + n_{hvnq}$ | Total number of heavy vehicles during cycle |

Table 4.4: Values Calculated from Field Measurements

⁽³⁾ Estimated for oversaturated cycles

Cycles with rare events that might affect the discharge process were eliminated from the analysis, i.e., a pedestrian illegally crossing a road or a vehicle blocking other vehicles for an extensive time.

Figure 4.4 shows the Excel-based spreadsheet used to tabulate the values measured in the field and calculate the additional quantities.

Other components, including the average cycle length, average length of green, percent of vehicles arriving on red, and percent of vehicles arriving on green, were calculated. For those coordinated intersections, the progression factor (PF) was calculated for use during the analysis.

4.3.2. <u>Data Collection for Delay Estimation</u>

Delay was measured according to the HCM method of counting stopped vehicles. Average delay in 15-minute periods was estimated by counting stopped vehicles every 20 seconds.

Delay was estimated for lane groups with through-movements only. The estimated values were later compared with those calculated with the HCS2000. HCM uses an acceleration/deceleration correction factor, which takes into account the free flow speed. In order to simplify the analysis, the approach speed was assumed to be equal to the posted speed limit.

The steps in measuring delay and the methodology for the evaluation of delay estimation are discussed in Chapter 7.

| | ene | SS | 10 | 58 | 7 | 31 | 10 | 29 | 47 | 6 | 24 | 33 | 49 | 26 | 50 | 23 | 14 | 48 | 17 | 5 | |
|--------------------|-----------------------------------------------------------|---------------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|
| ۴ | ne qu ends | шш | 39 | 40 | 43 | 46 | 49 | 51 | 53 | 56 | 59 | 2 | 4 | 7 | 8 | 11 | 14 | 17 | 21 | 24 | ţ |
| | Tr | ЧЧ | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 4 |
| , N _{hv2} | m. of cles in nd part lueue | Heavy Veh. | 0 | 0 | 0 | - | ٢ | 0 | 0 | ١ | ١ | 0 | 0 | 0 | 0 | 0 | 0 | ٢ | 0 | 0 | • |
| n _{pc2} | Nu Vehi seco of q | Pass. Veh. | 29 | 7 | 7 | 28 | 26 | 23 | 11 | 8 | 13 | 29 | 19 | 11 | 8 | 7 | 26 | 33 | 42 | 23 | , c |
| n _{hv1} | Num. of Heavy vehicles in first part of queue | trucks | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | ٢ | 0 | 0 | 0 | 0 | 0 | c |
| | hicle | SS | 15 | 45 | 55 | 39 | 15 | 51 | 27 | 51 | 56 | 41 | 15 | 5 | 34 | 11 | 29 | 49 | 5 | 24 | 33 |
| T_4 | 4th ve exits | mm | 38 | 40 | 42 | 45 | 48 | 50 | 53 | 55 | 58 | 1 | 4 | 7 | 8 | 11 | 13 | 16 | 20 | 23 | 26 |
| | Time | ЧЧ | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| ά | Queue | Heavy Veh. | 0 | 0 | 0 | 3 | ١ | 0 | 0 | ١ | 0 | 0 | 0 | 0 | ١ | 0 | ١ | ١ | 0 | 0 | - |
| U | Initial | Pass. Veh | 13 | 10 | 10 | 22 | 17 | 13 | 10 | 12 | 11 | 22 | 2 | 15 | 11 | 11 | 38 | 37 | 29 | 14 | 37 |
| | arts | SS | 7 | 36 | 46 | 30 | 9 | 41 | 17 | 43 | 48 | 31 | 8 | 55 | 18 | ٢ | 20 | 40 | 57 | 16 | 23 |
| T_{G} | ne gre ase str | шш | 38 | 40 | 42 | 45 | 48 | 50 | 53 | 55 | 58 | ٢ | 4 | 9 | 8 | 11 | 13 | 16 | 19 | 23 | 26 |
| | pha | ЧЧ | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| | j Du | SS | 5 | 34 | 44 | 28 | 4 | 39 | 15 | 41 | 46 | 29 | 9 | 53 | 16 | 59 | 18 | 38 | 55 | 14 | 21 |
| T_{cy} | End of onflicti Phase | шш | 38 | 40 | 42 | 45 | 48 | 50 | 53 | 55 | 58 | ٢ | 4 | 9 | 8 | 10 | 13 | 16 | 19 | 23 | 26 |
| | ŏ | ЧЧ | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| | Queue | Heavy Veh | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | С |
| Ъ. | Residual | Pass. Veh | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | MO | ss | 47 | 11 | 40 | 50 | 34 | 10 | 53 | 24 | 27 | 13 | 53 | 31 | 37 | 2 | 54 | 4 | 24 | 41 | С |
| ŕ | of Yell | mm | 36 | 39 | 41 | 43 | 46 | 49 | 50 | 54 | 57 | 0 | 2 | 5 | 7 | 10 | 11 | 15 | 18 | 21 | 25 |
| | End c | ЧЧ | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| | Lane | | WBT2 |
| ပ | Cycle | | 1 | 2 | 3 | 4 | 5 | 9 | 7 | 8 | 6 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |

Figure 4.4: Excel-Based Worksheet for Data Extraction

| Θ | extension time | (sec) | 2.73 | | | 0.73 | 4.13 | | | | | | | | | | | | | | |
|----------------------|-----------------------------------------------------------|---------------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| Я | Length of Red | Time (sec) | 80.00 | 85.13 | 65.47 | 100.67 | 92.27 | 91.40 | 144.53 | 78.60 | 81.40 | 77.60 | 74.87 | 83.33 | 41.27 | 58.73 | 86.00 | 95.33 | 93.00 | 95.67 | 83.40 |
| G/CL | Green Portion | | 0.42 | 0.40 | 0.46 | 0:36 | 0.38 | 0.07 | 0:30 | 0.55 | 0.49 | 0.49 | 05.0 | 0:30 | 69.0 | 0.44 | 0.53 | 0.50 | 0.51 | 0:50 | 0.53 |
| G | Length of Green | Time (sec) | 60.13 | 60.13 | 59.87 | 59.73 | 59.73 | 7.20 | 63.13 | 100.13 | 81.13 | 77.60 | 79.93 | 37.73 | 100.00 | 49.67 | 100.00 | 100.13 | 99.87 | 99.47 | 100.13 |
| CL | Cycle Length | Time (sec) | 144.13 | 149.27 | 129.33 | 164.40 | 156.00 | 102.60 | 211.67 | 182.73 | 166.53 | 159.20 | 158.80 | 125.07 | 145.27 | 112.40 | 190.00 | 199.47 | 196.87 | 199.13 | 187.53 |
| n _{arrG} | Veh. Arr. On green | # veh | 20.00 | 21.00 | 5.00 | 8.00 | 13.00 | 19.00 | 24.00 | 21.00 | 24.00 | 14.00 | 31.00 | 0.00 | 29.00 | 0.00 | 16.00 | 11.00 | 25.00 | 29.00 | 6.00 |
| ۸h | mber of sles ed from ction | heavy veh. | 0 | 0 | 0 | ٢ | ٢ | 0 | 0 | ٢ | ٢ | 0 | 0 | 0 | ٢ | 0 | 0 | 2 | ٢ | 0 | - |
| n _{pc} r | Total nur vehic Discharg interse | pass. Veh | 33.00 | 31.00 | 15.00 | 32.00 | 30.00 | 32.00 | 34.00 | 33.00 | 34.00 | 36.00 | 33.00 | 15.00 | 40.00 | 11.00 | 55.00 | 47.00 | 53.00 | 43.00 | 43.00 |
| , n _{hvq}) | nber of es in Je | heavy veh. | 0 | 0 | 0 | 1 | 1 | 0 | 0 | 1 | 1 | 0 | 0 | 0 | 1 | 0 | 0 | 1 | 0 | 0 | 1 |
| N (n _{pcq} | Total nur vehick quei | pass. Veh | 33.00 | 11.00 | 11.00 | 32.00 | 30.00 | 27.00 | 15.00 | 12.00 | 17.00 | 33.00 | 23.00 | 15.00 | 11.00 | 11.00 | 30.00 | 37.00 | 46.00 | 27.00 | 25.00 |
| n _{pc1} | Num. of pass. Vehicles in first part of queue | Pass. Veh. | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 3.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 | 4.00 |
| tq | Total queue Discharge Time | Time (sec) | 62.87 | 21.47 | 21.67 | 60.47 | 63.87 | 47.20 | 29.87 | 26.13 | 35.73 | 61.47 | 41.27 | 31.00 | 32.40 | 22.53 | 53.87 | 68.27 | 80.13 | 48.73 | 50.13 |
| ţ | Discharge Time of second part of queue | Time (sec) | 54.73 | 12.60 | 12.80 | 52.00 | 54.87 | 38.00 | 20.40 | 18.00 | 28.27 | 51.87 | 33.73 | 20.60 | 16.40 | 12.13 | 45.00 | 59.00 | 72.27 | 40.73 | 40.67 |
| t4 | Discharge Time of first part of queue | Time (sec) | 8.13 | 8.87 | 8.87 | 8.47 | 9.00 | 9.20 | 9.47 | 8.13 | 7.47 | 9.60 | 7.53 | 10.40 | 16.00 | 10.40 | 8.87 | 9.27 | 7.87 | 8.00 | 9.47 |
| , n _{hvnq} | m. of les not jueue | Heavy Veh. | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 0 | 0 |
| n _{pang} | Nu vehic in q | Pass . Veh | 0 | 20 | 4 | 0 | 0 | 5 | 19 | 21 | 17 | 3 | 10 | 0 | 29 | 0 | 25 | 10 | 7 | 16 | 18 |

Figure 4.5: Excel Based Worksheet: Additional Data Extracted

CHAPTER 5. ESTIMATING PARAMETERS

5.1. Vehicle Counts and Discharge Time

Saturated vehicle headways were estimated using the data of the second part of queue. Instead of using the standard method of saturation flow rate for a lane group, it was estimated for each lane in order to analyze its variability across lanes of a particular location. The time to discharge the intersection and the number of passenger and heavy vehicles per cycle were used to estimate the passenger and heavy vehicle headways in the following linear model:

$$t_{s} = h_{pc} \cdot n_{pc2} + h_{hv} \cdot n_{hv2} + e \quad , \tag{5.1}$$

where:

 t_s = average time for vehicles in second part of the queue to exit the intersection, n_{pc2} = average number of passenger vehicles in second part of the queue, n_{hv2} = average number of heavy vehicles in second part of the queue, h_{pc} = estimated average saturated headway for passenger vehicles, h_{hv} = estimated average saturated headway for heavy vehicles,e= error term.

The saturated headways (regression parameters) were estimated with the least-squares method using the regression procedure of the Statistical Analysis System (SAS) Computer Program Package (SAS Institute 1990). Table 5.1 shows example results for the northbound through-lanes at US 36 and Girls School Rd in the vicinity of

Indianapolis. The presented values were obtained based on data collected in 33 cycles for 531 vehicles.

| Variable | DF | Parameter Estimate | Standard <i>Error</i> | t Value | $\Pr > t $ | 95% Confidence Limits | | | | |
|----------|----|-----------------------|--------------------------|---------|-------------|--------------------------|------|--|--|--|
| h_{pc} | 1 | 1.826 | 0.037 | 48.5 | <.0001 | 1.75 | 1.90 | | | |
| h_{hv} | 1 | 4.198 | 0.427 | 9.8 | <.0001 | 3.33 | 5.07 | | | |

Table 5.1: Saturated Headways at US 36 & Girls School Rd (WBTH1)

Equations 5.2 to 5.5 were used to calculate the saturation flow rates, the passenger car equivalency factor, the lost time, and the corresponding estimation errors presented in Table 5.2. Estimates of the saturation flow rates *s*, the passenger car equivalency factor *E*, and the corresponding variances (standard deviations) were calculated for each traffic lane using the regression results and the following equations (respectively (symbol "~" signifies an estimate):

$$\widetilde{s} = \frac{3600}{\widetilde{h}_{pc}},\tag{5.2}$$

$$\operatorname{var}\widetilde{s} = \left(\frac{3600}{h_{pc}^2}\right)^2 \sigma_{pc}^2, \qquad (5.3)$$

$$\widetilde{E} = \frac{\widetilde{h}_{hv}}{\widetilde{h}_{pc}}, \qquad (5.4)$$

$$\operatorname{var} \widetilde{E} = \operatorname{var} \left(\frac{\widetilde{h}_{hv}}{\widetilde{h}_{pc}} \right) = \left(\frac{1}{h_{pc}} \right)^2 \sigma_{hv}^2 + \left(\frac{h_{hv}}{h_{pc}^2} \right)^2 \sigma_{pc}^2, \qquad (5.5)$$

The start-up lost time estimate \tilde{l}_1 is calculated from the estimated saturated headway. It is defined as the time at the beginning of the green phase that is not effectively used. It is calculated as the difference between the average discharge time of the first part of the queue \tilde{t}_4 and the discharge time of the same vehicles if they follow each other at the saturation time headway \tilde{h}_{pc} :

$$\widetilde{l}_1 = \widetilde{t}_4 - 4\widetilde{h}_{pc}, \qquad (5.6)$$

The variance of the estimated start-up lost time includes the variance of $\tilde{l_1}$ estimate and the variance of the saturated headway estimate is:

$$\operatorname{var} \widetilde{l}_{1} = \operatorname{var} t_{4} + 16 \,\sigma_{pc}^{2} \,, \tag{5.7}$$

Example estimates of the saturation flow rate and other capacity parameters obtained based on the results presented in Table 5.1 are shown in Table 5.2.

Table 5.2 Capacity Parameter Estimate (US 36 & Girls School Rd WBTH1)

| Parameter | Value | Standard Error |
|----------------------------------|-------|-------------------|
| Heavy Vehicle Equivalency Factor | 2.30 | 0.24 |
| Start-up Lost Time (sec) | 2.34 | 0.26 |
| Sat Flow Rate (pvphgpl) | 1971 | 41 |

The HCM2000 defines the extension of the effective green time (e) as:

"The amount of the change and clearance interval, at the end of the phase for a lane group, that is usable for movement of its vehicles." Also, it defines the change and clearance interval as "the yellow plus all-red interval that occurs between phases of a traffic signal to provide for clearance of the intersection before conflicting movements are released."

In order to avoid misinterpretation, the letter Y will be used for the length of the change and clearance interval and the letter A for the length of the amber phase.

The effective green extension time was calculated using the time when the last vehicle on a saturated queue exit the intersection and the time when the displayed green phase ends as shown in Equation 5.8:

$$e = T_q - (T_y - A)$$
 (5.8)

Forty-one through-lanes and 12 exclusive left-turn movements were included in the study. Table 5.3 and Table 5.4 show a summary of the average values of capacity parameters. Estimation accuracy was considered when calculating the averages by weighting the individual results with the inverse of the estimate variance. The heavy vehicle equivalency factor average value for the through-movement was estimated based on 28 lanes with heavy vehicles present.

| Parameter | Weighted Average Value | Min – Max Values | Standard Deviation |
|-------------------------------------------|---------------------------|---------------------|--------------------|
| Saturation Flow Rate (pvphgpl) | 1842 | 1352 - 2178 | 199 |
| Start-up Lost Time (sec) | 1.87 | 0.53 - 3.91 | 0.74 |
| Heavy Vehicle Equivalency Factor | 2.13 | 1.41 - 4.15 | 0.73 |
| Green Time Extension (sec) ⁽²⁾ | 2.81 | 0.03 - 5.83 | 1.26 |

Table 5.3: Summary of Capacity Parameters for Through Movements

⁽²⁾Including through and left turn movements

| Table 5.4: Summar | y Capacity | Parameters for | Left Turn Moveme | ents (3) |
|-------------------|------------|----------------|------------------|----------|
|-------------------|------------|----------------|------------------|----------|

| Parameter | Weighted Average Value | Min – Max Values | Standard Deviation |
|----------------------------------|---------------------------|---------------------|-----------------------|
| Saturation Flow Rate (pvphgpl) | 1844 | 1764 - 2079 | 117 |
| Start-up Lost Time (sec) | 1.61 | 0.57 - 2.91 | 0.71 |
| Heavy Vehicle Equivalency Factor | 2.08 | 1.45 - 2.98 | 0.53 |

⁽³⁾ Small size towns not included

Although the average value for the saturation flow rate is slightly below that recommended by the HCM2000, the variability between locations is quite strong and cannot be explained with the measurement error. Small communities tend to have considerably lower values than large communities. The average value of start-up lost time is close to the default two seconds recommended by the HCM2000. Also, the heavy vehicle equivalency factor is not significantly different from the default value of two.

Even though the lost time and equivalency factor estimates vary quite considerably across traffic lanes and intersections, the preliminary inspection of the results did not reveal any obvious trends. Most of the variability of these estimates can be attributed to the measurement error.

The estimates of average saturation flow rates for through and left-turn movements are close. It has to be stressed, however, that exclusive left-turn lanes were not present at the studied intersections in small communities. A comparison of the average saturation flow rates measured in medium towns and in Indianapolis indicated that the base saturation flow rate for left-turn movements was, on average, 96% of the values for through-movements. The default value recommended in the HCM is 95%.

This section describes the estimation of the PHF at 45 intersections, including locations at large, medium, small, and rural communities. First, the morning and afternoon rush hours were determined based on 12-hour vehicle counts. Then, the highest 15-minute count within the rush hour and the rush hour count for the N-S direction were used to calculate the PHF for this direction. The calculations were repeated for the E-W direction.

In order to analyze the PHF variability across locations, several site-specific characteristics were recorded, including population, volume per direction, time of day, and road class. Table 5.5 presents the summary of the PHF obtained values.

| Parameter | Value |
|----------------|-------|
| Average | 0.86 |
| Min. value | 0.58 |
| Max. value | 0.99 |
| Std. deviation | 0.082 |

Table 5.5: Summary of PHF values at 45 intersections

CHAPTER 6. VARIABILITY ANALYSIS

6.1. Capacity Parameters

The considerable variability of capacity parameter estimates across sites prompted for studying the effects of several local characteristics considered to be good candidates for capacity factors: population, road class, lane position in the lane group, and lane volume.

Population – Previous studies indicated that community size affected capacity. It appears that drivers in large communities are more aggressive than drivers in small communities. To investigate this effect, the developed areas in Indiana were classified in three categories: small, medium, and large. The only exception was small towns outside of the Indianapolis city limits but close enough for commuters to exhibit the driving style of a large city. These communities were considered part of the metropolitan area because majority of their drivers commuted to Indianapolis and exhibit the large community driving style.

Road Class – The effect of road class on capacity parameters was investigated. The function of each class may exhibit correlation with the percent of non-commuters present in the traffic flow. Arterials and US/State Routes tend to have more non-commuters (driver unfamiliar with the road) than local roads.

Position of Lane on the Approach – The studied intersections have lane groups with one, two, or three lanes. Heavy vehicles use right lanes more frequently than other lanes. It is also possible that less aggressive drivers tend to use the right-most lane. If any effect is found, lane groups with more than one lane would be representative of medium and large size communities, due to its absence in small communities.

Curb, Shoulder or Exclusive Lane – The effect of geometric conditions to the right of the right-most lane was also evaluated. Three cases were considered: curb, shoulder, and exclusive right-turn lane. It is possible that the presence of curb and inlets may reduce the saturation flow rate.

Lane Volume – Lane volume has been considered in previous research as a factor that might increase saturation flow rate, especially among the first vehicles in a queue. The numbers of vehicles that exited the studied intersections in the cycles used to estimate saturation flow rates were converted to lane hourly volumes.

For the analysis, the base case was defined as a single lane approach on an arterial road located in a large community with an exclusive right turn lane.

A weighted regression analysis was used to account for the varying precision of the saturation flow estimates. The following model was applied to the saturation flow rate estimates for through-lanes:

$$s = s_0 + a_m \cdot m + a_{sm} \cdot sm + a_{LN} \cdot LN + a_c \cdot c + a_{sh} \cdot sh + a_{co} \cdot CO + a_L \cdot L + a_{LV} \cdot LV, \quad (6.1)$$

where:

s = base saturation flow rate,

 s_0 = base saturation flow rate for the base case scenario,

m = 1 if medium size town, 0 otherwise,

sm = 1 if small size town, 0 otherwise,

LN = 1 if multi-lane group, 0 otherwise,

- c = 1 if curb present at right side border of right-most lane, 0 otherwise,
- sh = 1 if shoulder present at right side border of right-most lane, 0 otherwise,
- CO = 1 if collector road, 0 otherwise,

L = 1 if local road, 0 otherwise,

LV = hourly volume for particular lane.

Two types of errors were present in the saturation flow estimate obtained from the model: the disturbance term and the measurement error. The disturbance term was caused by factors not included in the model. The measurement error was caused by imperfection of the measurement method and the limited size of the sample. To account for the latter error, weighted linear regression was used with weights equal to the inverse of the squared measurement error.

All model variables included in Equation 6.1 were tested for statistical significance. The following insignificant variables were removed from the model: road class, presence of curb, shoulder or exclusive right-turn lane, and hourly lane volume. The model specification is shown on Table 6.1.

| Variable | DF | Parameter Estimate | Standard Error | T value | $\Pr > t$ | 95% Con Lim | fidence its |
|-----------|----|-----------------------|-------------------|---------|-----------|----------------|----------------|
| Intercept | 1 | 1963.0 | 37.4 | 52.47 | <.0001 | 1887.3 | 2038.6 |
| m | 1 | -162.9 | 38.1 | -4.28 | 0.0001 | -240.1 | -85.9 |
| sm | 1 | -423.2 | 52.5 | -8.06 | <.0001 | -529.3 | -317.0 |
| LN | 1 | 88.2 | 38.7 | 2.28 | 0.028 | 10.0 | 166.4 |

Table 6.1 Parameter Estimate Using Weighted Regression

 $R^2 = 0.75$

The final model is:

$$s = 1963 - 163 \cdot m - 423 \cdot sm + 88 \cdot LN \tag{6.2}$$

The regression analysis indicated that the community size and the lane position had a significant impact on the saturation flow rate. The intercept of 1963 pvphgpl, represents the average saturation flow rate of a single-lane group in a large community. Figure 6.1 compares the predicted values for saturation flow rate with those measured.

The saturation flow rate model for a lane was converted to a base saturation flow rate model convenient for calculation of the group saturation flow rate:

$$s = 1963 - 163 \cdot m - 423 \cdot sm + 88 \cdot \frac{N-1}{N}, \tag{6.3}$$

where:

s = base saturation flow rate for lane group,

m = 1 if medium size town, else 0,

sm = 1 if small size town, else 0,

N = number of additional lanes from base scenario.

Table 6.2: Base Saturation Flow Rates Recommended for Indiana

| | | Population | |
|-----------------|----------------|------------|--------------|
| Number of Lanes | < 20 thousands | 20 to 100 | Indianapolis |
| 1 | 1540 | 1800 | 1960 |
| 2 | 1580 | 1840 | 2010 |
| 3 | 1600 | 1860 | 2020 |

The model is recommended for Indiana to replace the default value of 1900 veh/h/lane. Table 6.2 presents the values for various local conditions and rounded to the nearest tenths.



Figure 6.1: Comparison of Measured and Estimated Base Saturation Flow Rate

6.2. Peak Hour Factor (PHF)

The variability of the PHF was investigated. The investigated characteristics included the time of day, population size, directional volume, and road class.

Time of Day - It is believed that the morning peak is different from the afternoon peak. This difference may be expected because most morning trips are work-related while afternoon trips are more diversified. The afternoon peak tends to be longer and flatter than the morning peak. *Population* – The effect of the size of the town where the intersection is located was analyzed. Populations were classified as large, medium, small, and rural. It is postulated that large towns have a less pronounced peaking pattern as trips are longer and more staggered in time than in small towns.

Volume – The effect of the volume per direction was investigated. The selected morning and afternoon rush hour volume (in thousands) was used.

Road Class – The effect of road class might be related to the directional volume. Arterials and Collectors usually have larger volumes than local and rural roads. Roads were also classified as US, SR, local, and rural.

The proposed model is as follows:

$$PHF = PHF_{b} + a_{AM} \cdot AM + a_{POP} \cdot POP + a_{VOL} \cdot VOL + a_{SR} \cdot SR, \qquad (6.4)$$

where:

PHF = Peak Hour Factor for site specific conditions, PHF_b = Intercept PHF,AM = 1 if morning (AM); 0 otherwise,POP = 1 if large or medium town; 0 otherwise,VOL = Directional rush hour volume (in thousands),SR = 1 if US or State administered road; 0 otherwise.

All of the model variables included in Equation 6.4 were tested for statistical significance. The only statistically-insignificant variable was road class. The model specification is shown on Table 6.3.

| Variable | DF | Parameter Estimate | Standard Error | T value | $\Pr > t$ | 95% Con Lim | fidence its |
|--------------|----|-----------------------|-------------------|---------|-----------|----------------|----------------|
| Intercept | 1 | 0.832 | 0.013 | 62.92 | <.0001 | 0.806 | 0.858 |
| AM | 1 | -0.049 | 0.011 | -4.54 | <.0001 | -0.070 | -0.028 |
| РОР | 1 | 0.044 | 0.012 | 3.77 | 0.0002 | 0.021 | 0.066 |
| VOL | 1 | 0.032 | 0.006 | 5.49 | <.0001 | 0.020 | 0.043 |
| $P^2 = 0.26$ | | | | | | | |

Table 6.3 Parameter Estimate

 $R^2 = 0.26$

The regression analysis indicated that the time of day, community size, and lane volume have a significant impact on the peak hour factor. The intercept of 0.83 represents the average PHF for a local road in a small or rural community during the afternoon period. Figure 6.2 compares the predicted PHF with the measured values.

The PHF model for a direction is as follows:

$$PHF = 0.83 + -0.05 \cdot AM + 0.04 \cdot POP + 0.03 \cdot VOL, \qquad (6.5)$$

where:

PHF = Site and direction specific peak hour factor,
AM = (AM=1, PM=0),
POP = Population Size (Large or Medium =1, Small or Rural =0),
VOL = Total hourly volume on opposing approaches (in thousands).

The scatter plot in Figure 6.2 indicates large dispersion. It can be explained with the omission of some factors in the model and with the day-to-day variability of PHF values. The PHF is sensitive to fluctuations in demand, and the PHF measured day after day at a certain intersection during the same rush period will vary significantly. The data used from the 45 intersections reflect the PHF for a specific day when the data was collected,

while the value calculated with the model is the average value across various days. The model should be used to estimate demand for future signalized intersections or for those locations where data cannot be collected.



Figure 6.2: Comparison of Predicted and Measured Values of Peak Hour Factor

CHAPTER 7. EVALUATION OF THE IMPROVEMENT IN DELAY PREDICTIONS

Chapter 6 provided the base saturation flow rates and lost times recommended for Indiana. This chapter evaluates the improvement expected in delay predictions when using the recommended parameters.

7.1. Evaluation Method

For the evaluation task, 15 intersections with videotaped traffic were selected. Following the HCM2000, average control delay was measured at 18 lane groups within the selected intersections. Then, the collected input data, including traffic volume, percent of heavy vehicles, and signal timings, were used to estimate control delay with the HCS 2000. Two cases were calculated for each lane group:

- 1. Base saturation flow rates and lost times recommended by the HCM as default values
- 2. Base saturation flow rates and lost times recommended in Chapter 6 for Indiana

The obtained results were compared to the delay measurements to calculate the prediction errors. Table 7.1 shows the selected locations for evaluation of the method.

7.1.1. Delay Measurement

The errors in both cases were compared to evaluate the prediction improvement. The HCM uses control delay as a traffic quality criterion at signalized intersections. Average control delay per approach is the amount of time the average vehicle spends at the intersection. It includes stopped delay and acceleration/deceleration delay.

| Town | Intersection | Lane Group | Number of Lanes |
|----------------|---------------------------|------------|--------------------|
| Lafayette | SR 26 & Creasy Lane | WBTH | 2 |
| Lafayette | SR 26 & Creasy Lane | NBTH | 2 |
| Indianapolis | US 31 & Girls School Road | WBTH | 2 |
| Indianapolis | US 31 & Girls School Road | SBTH/R | 2 |
| Kokomo | US 31 & Alto Road | NBTH | 2 |
| Indianapolis | US 31 & 103rd St | SBTH | 3 |
| Indianapolis | US 36 & Raceway Rd | EBTH | 2 |
| Indianapolis | US 36 & High School Rd | EBTH | 3 |
| Pendleton | US 36 & SR 38 | NBTH | 1 |
| West Lafayette | US 231 & SR 26 | WBTH | 2 |
| West Lafayette | US 231 & SR 26 | EBTH | 2 |
| Demotte | US 231 & 9th St | NBTH | 1 |
| Kokomo | US 31 & Southway Blvd | SBTH | 2 |
| Indianapolis | US 31 & 116th St | SBTH | 2 |
| Lafayette | US 52 & SR 26 | SBTH | 2 |
| Indianapolis | US 31 & Greyhound Pass | SBTH | 3 |
| Indy/Carmel | US 31 & 126th St | SBTH | 2 |
| Indy/Carmel | US 31 & 126th St | EBTH | 2 |

Table 7.1 Evaluation Lane Groups

The HCM method of measuring control delay estimates the time-in-queue per vehicle for each lane group and then it adjusts the obtained estimates for deceleration and acceleration . The time in queue survey was conducted on a lane-by-lane basis, but the calculations were conducted by lane group. The input needed for delay measurement includes the number of lanes for the lane group (N), free flow speed (FFS), survey count interval (I_s), total number of vehicles arriving (V_{tot}), and the number of stopped vehicles (V_q) counted in this study every 20 seconds. A sample calculation from US36 & Girls School Road is included in the Appendix. Vehicles were considered stopped if they were within a car-length distance from the last stopped vehicle. Some vehicles could be counted multiple times if they are present on the approach for several counting intervals. During the time period T₁₅, there were 45 intervals of 20-seconds during which vehicles were counted.

After the lane group input field data was completed, time-in-queue per vehicle was calculated using Equation 7.1:

$$d_{vq} = \left(I_{s} \cdot \frac{\sum V_{iq}}{V_{tot}}\right) \cdot 0.9, \qquad (7.1)$$

where:

 $I_{s} = \text{interval between vehicle-in-queue counts (s),}$ $\sum V_{iq} = \text{sum of vehicle-in-queue counts (veh),}$ $V_{tot} = \text{Total vehicles arriving during the survey period (veh) and,}$ 0.9 = empirical adjustment factor.

Control delay includes additional acceleration-deceleration delay related to the time a vehicle needs to decelerate from and accelerate to free-flow speed. This component is not included in the value defined in Equation 7.1 above. The HCM gives an average acceleration-deceleration delay per one stopped vehicle based on the free-flow speed and the number of vehicles in queue and shown in Table 7.2.

Table 7.2 Exhibit A16-2 Acceleration-Deceleration Delay CF (s)

| Free-Flow Speed | \leq 7 vehicles | 8 – 9 vehicles | 20-30 vehicles ^a |
|--------------------|-------------------|----------------|-----------------------------|
| \leq 37 mi/h | +5 | +2 | -1 |
| > 37 – 45 mi/h | +7 | +4 | +2 |
| > 45 mi/h | +9 | +7 | +5 |

Note: ^a Vehicle-in-queue counts in excess of about 30 vehicles per lane are typically unreliable

The final delay correction for acceleration-deceleration is a product of the fraction of vehicles stopped, FVS, and the correction, CF, taken from Table 7.2. The fraction FVS is calculated as:

$$FVS = \frac{V_{stop}}{V_{tot}}.$$
(7.2)

And the acceleration-deceleration delay correction is:

$$\mathbf{d}_{\mathrm{ad}} = \mathrm{FVS} \cdot \mathrm{CF} \ . \tag{7.3}$$

Finally the control delay per vehicle is estimated adding the two components estimated previously:

$$\mathbf{d} = \mathbf{d}_{\mathrm{vq}} + \mathbf{d}_{\mathrm{ad}} \tag{7.4}$$

The delay measured at the selected lane groups is shown in Table 7.3

| Town | Intersection | Approach | Number of Lanes | Average Control Delay |
|----------------|-------------------------|----------|--------------------|--------------------------|
| Lafayette | SR 26 & Creasy Lane | WBTH | 2 | 14.2 |
| Lafayette | SR 26 & Creasy Lane | NBTH | 2 | 50.0 |
| Indianapolis | US 31 & Girls School Rd | WBTH | 2 | 53.5 |
| Indianapolis | US 31 & Girls School Rd | SBTH/R | 1 | 93.1 |
| Kokomo | US 31 & Alto Rd | NBTH | 2 | 32.1 |
| Indianapolis | US 31 & 103rd St | SBTH | 3 | 8.3 |
| Indy/Avon | US 36 & Raceway Rd | EBTH | 2 | 10.3 |
| Indianapolis | US 36 & High School Rd | EBTH | 3 | 32.5 |
| Pendleton | US 36 & SR 38 | NBTH | 1 | 29.1 |
| West Lafayette | US 231 & SR 26 | WBTH | 2 | 33.7 |
| West Lafayette | US 231 & SR 26 | EBTH | 2 | 33.5 |
| Demotte | US 231 & 9th St | NBTH | 1 | 9.0 |
| Kokomo | US 31 & Southway Blvd | SBTH | 2 | 13.4 |
| Indianapolis | US 31 & 116th St | SBTH | 2 | 17.1 |
| Lafayette | US 52 & SR 26 | SBTH | 2 | 30.5 |
| Indy/Westfield | US 31 & Greyhound Pass | SBTH | 3 | 46.0 |
| Indy/Carmel | US 31 & 126th St | SBTH | 2 | 32.0 |
| Indy/Carmel | US 31 & 126th St | EBTH | 2 | 45.1 |

Table 7.3 Measured Delays

7.2. Delay Prediction

The HCS2000 was used to calculate delays for the selected 18 lane groups. Delay was predicted for the two cases: Case A using default capacity parameters and Case B using local capacity parameters. Other input values used in both cases were included. The measured input values were vehicle volume, signal timings, percent of heavy vehicles, and arrival type. The geometry input data reflect the base conditions specified in the HCM 2000. The target output from HCS2000 was the control delay for the selected lane groups.

Because base conditions were present at each location, the adjustment factors were maintained as the default values recommended by HCM2000. Two-lane groups with one lane shared between through and right-turn movements were considered due to the weak turning movement effect being negligible.

Case A used the default capacity parameters values shown in Table 7.4. These values are provided by the HCS2000 as default values.

| Parameter | Default Value |
|----------------------------------|---------------|
| Start-up Lost Time (sec) | 2.0 |
| Green Time Extension (sec) | 2.0 |
| Base Saturation Flow Rate (vphg) | 1900 |

Table 7.4 Default Input Capacity Parameters

The base saturation flow rate used was selected from Table 6.2 based on the population size and number of lanes. If two different lane groups at the same intersection were evaluated, the base saturation flow rate would be a specific value based on the number of

lanes

The PHF model developed in Chapter 6 was not used for the evaluation; instead, the HCS2000 recommended PHF value was used. The model was developed using peak hour volume but the data collected for the evaluated intersections in some occasions was not necessarily during the peak hour. In order to determine the benefit of using the local capacity parameters, PHF was remained constant for both cases.

The measured and predicted delays were compared. The difference between the predicted and measured delay was calculated by subtracting the second from the first. The sign of the error showed if the prediction was an over or under-estimation. Table 7.5 shows the results of the evaluation. The trend of the prediction in Cases A and B shows that the method overestimates and underestimates evenly throughout the evaluated intersections.

The results show that the mean error of the delay predicted with local capacity parameters is close to zero. Also, the standard deviation of Case B is lower than for Case A. This trend shows that the error with local capacity parameter values remains constant compared to the error with default values. The prediction error at some locations is lower with default parameters, but, on average, across all locations it is larger by three seconds.

| | | | | LA | NE GROUP CON | TROL D | ELAY (SEC/VEH | |
|-------------|------------------------|---------------|--------------------|----------|--------------------------------------------|--------|------------------------------------------|-------|
| | Intersection | Lane Group | Number of Lanes | Measured | Default Capacity Parameters (Case A) | Error | Local Capacity Paramaters (Case B) | Error |
| L R R | 26 & Creasy Lane | WBTH | 2 | 14.2 | 18.9 | 4.7 | 18.5 | 4.3 |
| SR R | 26 & Creasy Lane | NBTH | 2 | 50.0 | 52.2 | 2.2 | 51.4 | 1.4 |
| 3 | 31 & Girls School Rd | WBTH | 2 | 53.5 | 80.4 | 26.9 | 61.8 | 8.3 |
| 3 | 3 31 & Girls School Rd | SBTH/R | 1 | 93.1 | 123.9 | 30.8 | 107.9 | 14.8 |
| Ľ, | S 31 & Alto Rd | NBTH | 2 | 32.1 | 25.3 | -6.8 | 25.0 | -7.1 |
| Ξ. | S 31 & 103rd St | SBTH | 3 | 8.3 | 15.2 | 6.9 | 12.8 | 4.5 |
| | S 36 & Raceway Rd | EBTH | 2 | 10.3 | 18.0 | L'.L | 16.8 | 6.5 |
|) | JS 36 & High School Rd | EBTH | 3 | 32.5 | 36.5 | 4.0 | 33.4 | 0.9 |
| _ / | JS 36 & SR 38 | NBTH | ٦ | 29.1 | 16.0 | -13.1 | 18.6 | -10.5 |
| | JS 231 & SR 26 | WBTH | 2 | 33.7 | 29.0 | -4.7 | 28.5 | -5.2 |
| | JS 231 & SR 26 | EBTH | 2 | 33.5 | 29.3 | -4.2 | 28.7 | -4.8 |
| | JS 231 & 9th St | NBTH | 1 | 9.0 | 2.0 | -4.0 | 5.3 | -3.7 |
|) | JS 31 & Southway Blvd | SBTH | 2 | 13.4 | 22.4 | 0.6 | 20.7 | 7.3 |
| | JS 31 & 116th St | SBTH | 2 | 17.1 | 24.4 | 2.3 | 22.0 | 4.9 |
| | IS 52 & SR 26 | SBTH | 2 | 30.5 | 27.3 | -3.2 | 26.8 | -3.7 |
|) | IS 31 & Greyhound Pass | SBTH | 3 | 46.0 | 39.9 | -6.1 | 34.3 | -11.7 |
| | JS 31 & 126th St | SBTH | 2 | 32.0 | 25.9 | -6.1 | 24.1 | -7.9 |
| | JS 31 & 126th St | EBTH | 2 | 45.1 | 43.6 | -1.5 | 42.3 | -2.8 |
| | | | Mean Err | or | | 2.8 | | -0.3 |
| | | | Std Devia | ation | | 11.3 | | 7.2 |

Table 7.5 Delay Evaluation Results

CHAPTER 8. CONCLUSIONS

This study has shown that, on average, the capacity parameters for Indiana are similar to the national default capacity parameter values. The average base saturation flow rate for through-lanes for Indiana is 1842 pcphgpl, considering different communities with various population sizes. The average value for left-turn movements is 1844 pcphgpl considering medium and large communities only. These results show that the state average is 3.0 percent lower than the national default value of 1900 pcphgpl for through movements.

A strong variability of base saturation flow rates across locations was concluded and its sources were investigated. Also, the variability across lanes from the same lane groups was analyzed. The different factors investigated included the population size, road class, position of lanes in lane group, and lane volume. The population size and the lane position in lane group were found to have significant impact on the base saturation flow rate. Appendix A presents the recommended values for saturation flow rate based on the number of lanes and the population size.

The difference in the base saturation flow rate was moderate. Medium-size communities, on average, had a base saturation flow rate eight percent lower than Indianapolis. Medium and small–size communities presented, on average, a base saturation flow rate of 8 and 21 percent lower than larger communities. These results are consistent with the

Kentucky study (K. R. Agent., et. al. 1982) that showed that medium and small-size communities have 8 and 19 percent lower base saturation flows, respectively, than larger communities. This trend shows that an adjustment factor based on population might be needed for capacity analysis if the base saturation flow rate is to be kept fixed.

The average value of the saturation flow rate for left-turns has been estimated based on medium and large communities as the left-turns found in small communities were permitted-only. The average base saturation flow for through-movements in medium and large communities is 1942 pcphgpl, which means that the left-turn base saturation flow is 95 percent of the through-movement value for comparable local conditions. This is consistent with the HCM-recommended adjustment value for protected left-turns which is 0.95.

The average start-up lost time value for through-movements was 1.87 seconds, which is close to the HCM-recommended value of 2.0 seconds. Although the range of obtained values varied widely between 0.53 to 3.91 seconds, no factors of the start-up lost time could be identified. A relatively large measurement error at individual locations might be the major source of this variability.

Other capacity factors, such as the heavy vehicle equivalency factor (E) and the green time extension (e) were estimated. The E for through-movement 2.13 and for left-turns, 2.08, are close to the HCM-recommended value of 2.0.

The green time extension value of 2.81 was estimated by combining through and left-turn movements due to the low number of lane groups with oversaturated cycles. Left-turn movements were observed only during the protected phase. The average length of the change and clearance interval at the studied locations was 5.6 seconds.

The peak hour factor was estimated for 45 intersections using data provided by the INDOT district offices. A morning and afternoon value was estimated in order to investigate the impact of the time of day on PHF. The results show that the time of day, population, and approach volume are significant factors of PHF prediction. As expected, morning rush hour tended to be sharper, which means that demand during the peak hour is higher than for the afternoon peak. Also, when investigating the effect of population size, it was found that small and rural communities tend to have a lower PHF, due to the trend of short and sharp peak hours. On the other hand, medium and large communities tend to have long and more dispersed peak hours, and therefore higher PHF. We recommend estimating PHF from the provided equation if field data are not available.

Delay was estimated to evaluate the prediction improvement using the local capacity parameters. Delay estimation using the local parameter values was closer to the measured delay when compared with the delay estimated using the default parameter values. The mean error using local values was closer to zero and with default parameters, on average, tended to overestimate delay by about three seconds per vehicle.

Based on the findings of this research it is recommended that site specific capacity parameters are used whenever possible. Furthermore, for delay prediction of future

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intersections or future conditions of existing intersections, the local parameters should be used.

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APPENDIX A.

Recommended Base Saturation Flow Rates for Indiana

| Number of Lanes in | Population Size Near Intersection | | | | |
|--------------------|-----------------------------------|------------------|--------------|--|--|
| lane group | < 20,000 | 20,000 - 100,000 | Indianapolis | | |
| 1 | 1540 | 1800 | 1960 | | |
| 2 | 1580 | 1840 | 2010 | | |
| 3 | 1600 | 1860 | 2020 | | |

Recommended Capacity Parameters for Indiana

| Parameter | Value |
|---------------------------------------|-------|
| Green Time Extension | 2.8 s |
| Heavy Vehicle Equivalency Factor | 2.1 |
| Start-up Lost Time | 1.9 s |
| Protected Left-turn Adjustment Factor | 0.95 |

APPENDIX B.

Recommended Saturated Time Headways for Indiana

| Number of Lanes in | Population Size Near Intersection | | | | |
|--------------------|-----------------------------------|------------------|--------------|--|--|
| lane group | < 20,000 | 20,000 - 100,000 | Indianapolis | | |
| 1 | 2.34 | 2.00 | 1.83 | | |
| 2 | 2.27 | 1.95 | 1.79 | | |
| 3 | 2.25 | 1.94 | 1.78 | | |

Recommended Capacity Parameters for Indiana

| Parameter | Value |
|---------------------------------------|-------|
| Green Time Extension | 2.8 s |
| Heavy Vehicle Equivalency Factor | 2.1 |
| Start-up Lost Time | 1.9 s |
| Protected Left-turn Adjustment Factor | 0.96 |

APPENDIX C.

ESTIMATED BASE SATURATION FLOW RATE FOR THROUGH MOVEMENTS

| T | T , , , , , | Intersection | | Base Saturation |
|------------------------|----------------------------------|--------------|------|-----------------|
| lown | Intersection | Approach | Lane | pcphgpl |
| Indianapolis | Michigan Rd. & DePauw Blvd. | NB | 1 | 1945 |
| Indianapolis | Michigan Rd. & DePauw Blvd. | NB | 2 | 1720 |
| Indianapolis | Michigan Rd. & DePauw Blvd. | NB | 3 | 1880 |
| West Lafayette | Northwestern Ave. & Stadium Ave. | EB | 1 | 1872 |
| West Lafayette | Northwestern Ave. & Stadium Ave. | NB | 2 | 2005 |
| West Lafayette | Northwestern Ave. & Stadium Ave. | NB | 1 | 1786 |
| West Lafayette | Northwestern Ave. & Stadium Ave. | SB | 2 | 1839 |
| Lafayette | SR26 & Creasy Lane | NB | 2 | 1730 |
| Lafayette | SR26 & Creasy Lane | WB | 2 | 1783 |
| Lafayette | SR26 & Creasy Lane | WB | 1 | 1779 |
| Carmel/Indianapolis | US 31 & 126th St | EB | 2 | 1860 |
| Carmel/Indianapolis | US 31 & 126th St | SB | 2 | 2079 |
| Carmel/Indianapolis | US 31 & 126th St | SB | 1 | 1860 |
| Kokomo | US 31 & Alto Rd. | NB | 2 | 1858 |
| Kokomo | US 31 & Alto Rd. | NB | 1 | 1770 |
| Westfield/Indianapolis | US 31 & Greyhound Pass | SB | 3 | 2178 |
| Westfield/Indianapolis | US 31 & Greyhound Pass | SB | 2 | 2117 |
| Westfield/Indianapolis | US 31 & Greyhound Pass | SB | 1 | 2069 |
| Indianapolis | US 36 & Girls School Rd | SB | 2 | 1719 |
| Indianapolis | US 36 & Girls School Rd | WB | 2 | 2107 |

| _ | | | - (1) | Base Saturation Flow |
|-------------------|-----------------------------|----------|---------------------|----------------------|
| Town | Intersection | Approach | Lane ⁽¹⁾ | pcphgpl |
| Indianapolis | US 36 & Girls School Rd | WB | 1 | 1971 |
| Indianapolis | US 36 & High School Rd. | EB | 3 | 1980 |
| Indianapolis | US 36 & High School Rd. | EB | 2 | 1878 |
| Indianapolis | US 36 & High School Rd. | EB | 1 | 2051 |
| Pendleton | US 36 & SR 38 | NB | 1 | 1352 |
| Monticello | US 24 & 6 th St. | WB | 1 | 1634 |
| Attica | SR 28 & US 42 | WB | 1 | 1395 |
| Demotte | US231 & 9th St | NB | 1 | 1478 |
| Rockville | US41 & SR 36 | NB | 1 | 1649 |
| Spencer | US231 & SR67 | EB | 1 | 1455 |
| Lafayette | US 52 & SR 26 | SB | 2 | 1767 |
| Lafayette | US 52 & SR 26 | SB | 1 | 1636 |
| Lafayette | US 52 & SR 26 | WB | 1 | 1747 |
| West Lafayette | US231 & SR26 | WB | 2 | 1825 |
| West Lafayette | US231 & SR26 | WB | 1 | 1856 |
| Indianapolis | US31 & 103rd St | SB | 1 | 1979 |
| Indianapolis | US31 & 116th St | SB | 1 | 2169 |
| Kokomo | US31 & Southway Blvd. | SB | 2 | 2008 |
| Kokomo | US31 & Southway Blvd. | SB | 1 | 1752 |
| Avon/Indianapolis | US36 & Raceway Rd | EB | 2 | 2131 |
| Avon/Indianapolis | US36 & Raceway Rd | EB | 1 | 1801 |
| West Lafayette | US52 & Salisbury St. | EB | 1 | 1683 |
| West Lafayette | US52 & Salisbury St. | EB | 2 | 1925 |

⁽¹⁾Lane number starting from right-most lane