COMPARISON OF STOPPED DELAY BETWEEN FIELD MEASUREMENTS AND HCM 2010 ESTIMATIONS AT ACTUATED SIGNALIZED INTERSECTIONS

BY

MOHAMMED ABDUL RAWOOF SHAIK

THESIS

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Adviser:

Professor Rahim F. Benekohal

ABSTRACT

The Highway Capacity Manual 2010 methodology of capacity analysis is widely used for performance evaluation of signalized intersections. However, very few studies have been performed to validate the HCM delay estimation model. This thesis aims to study the accuracy of the HCM 2010 delay estimation methodology for actuated signalized intersections using field data. The HCM stopped delay estimates were statistically compared with the field measurements for six signalized intersections along an actuated-coordinated corridor during four time periods of day, namely AM peak, off peak, noon peak, and PM peak. Overall, the HCM estimates were not significantly different from the field data only for 38.1% of the comparisons (32 cases out of 84). In the significant discrepancies, 74.2% of the HCM overestimations were on the minor streets (23 cases out of 31) and 95.2% of the underestimations were on the major street (20 cases out of 21). Hence, HCM significantly overestimated the delay on minor streets most of the time and significantly underestimated on major streets almost all the time. The over- and underestimation trend was verified for each intersection as well as each time period. In light of this trend, some insight is provided into the causes of observed discrepancies and recommendations are made for future studies.

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CHAPTER 1

INTRODUCTION

Capacity analysis for actuated or actuated-coordinated signalized intersections can be performed by following the methodology described in the Highway Capacity Manual (HCM) 2010 (TRB, 2010). This methodology has been widely recognized and implemented in the Highway Capacity Software (HCS) 2010 (McTrans, 2010). The 'Streets' module in HCS 2010 (version 6.70) implements the procedure for calculating control delay at signalized intersections as described in Chapter 18 of HCM 2010.

As part of the capacity estimation for an actuated signalized intersection, Chapter 31 of HCM 2010 recommends an iterative procedure to estimate the average duration of an actuated phase based on a queue accumulation polygon, probability of green extension and phase call, among other parameters. The delay calculations for an actuated signalized intersection are carried out essentially to a fixed cycle signal having an equal cycle length and the average phase durations.

1.1 Need for Research

Traffic engineers use the HCM 2010 methodology to determine the performance of a signalized intersection and classify its level of service (LOS) based on average control delay per vehicle. Thus, accurate estimates of the delay are important for better performance evaluation of signalized intersections.

In the literature as discussed in Chapter 2, studies of delay estimation accuracy of the HCM methodology for signalized intersections are very few and were performed for previous HCM editions of 1985, 1994, 1997, and 2000 only. Moreover, for the HCM 1997 and 2000 methodologies, there were no studies in the literature that correspond to actuated-signal control, but were performed only for pre-timed signal control, to the best of author's knowledge. Some other studies compared the delays computed by different software packages such as HCS, SIDRA, Synchro, PASSER, CORSIM, etc. Thus, the absence of a field validation study of the HCM 2010 delay estimation model for actuated signalized intersections highlights the need and motivation of this investigation.

1.2 Research Objective

The primary objective of this study is to compare the accuracy of HCM 2010 delay estimations with respect to field measurements. This is achieved by measuring field delay and making statistical comparison with computed HCM estimates.

Field data was collected along an actuated-coordinated signalized urban arterial corridor of six intersections in Champaign, Illinois. Data reduction was performed to determine desired traffic characteristics such as volume counts, saturation flow rate and field delay, among others. This data was analyzed to compare the stopped delay between field measurements and HCM 2010 estimations. The HCM estimates are computed through HCS 2010 (version 6.70) runs using existing field conditions in inputs. The detailed information of data collection, data reduction and data analysis follows in the succeeding chapters of the thesis. Results of the analysis are discussed to provide insight into potential causes of observations and trends.

1.3 Thesis Organization

The thesis consists of six chapters in all that contribute towards different aspects of the study. Chapter 2 reviews previous research on accuracy of delay estimation models of various HCM editions and other software packages. Chapter 3 describes the study area and data collection methodology used in the study. Chapter 4 presents the methodology and outcomes of data reduction performed following the data collection. Chapter 5 discusses the capacity analyses carried out to obtain HCM 2010 delay estimates using HCS 2010 and the statistical comparison between HCM estimates with field measurements. It then goes on to discuss the results and findings of the analysis and provide some insight into causes of the observed discrepancies. Chapter 6 concludes the thesis and provides recommendations for further study.

CHAPTER 2

REVIEW OF LITERATURE

This chapter reviews the relevant studies that were conducted in the past to investigate accuracy of HCM delay estimates in comparison to field measured delay, and comparison of estimates from different software packages.

Braun and Ivan (1996) published one of the first papers in this area which studied the average stopped delay estimation accuracy at approach level for HCM 1885 and 1994 editions. The authors measured average stopped delay at eight intersections with semi-actuated signalized operation during an afternoon peak hour using traffic videos recorded in Connecticut, USA. They also determined peak-hour flow rate, intersection geometry, and signal phasing to calculate delay estimates using the stopped delay equations of the two HCM models. They concluded that HCM 1994 estimated approach stopped delay better than HCM 1885. However, the error in the estimation by HCM 1994 model with respect to field was as high as 6.9 seconds for coordinated approaches and 8.6 seconds for non-coordinated approaches. The error caused by the HCM 1885 model was even higher.

Prevedouros and Koga (1996) also studied the delay estimation accuracy of HCM 1885 and 1994 editions. The data in this study was collected from two fully-actuated signalized intersections, one in Chicago, Illinois and the other in Honolulu, Hawaii. With the help of a crew, the authors collected the required traffic data from field and compared both the HCM models with field. It was concluded that HCM 1994 improved the accuracy of delay and level of service estimation as compared to HCM 1885. However, the range of errors was -19% to 42% at the

Chicago site and -28% to 19% at the Honolulu site for the 1994 model. The error range for the 1885 model were even higher. The difference of the trend in estimation errors at the Chicago and Honolulu intersections was attributed to the variation in saturation flow rate from the HCM's ideal value at the time and the role of progression factor.

Petraglia (1999) studied the HCM 1994's capacity analysis methodology with two objectives: first, compare various analysis tools to the HCM methodology using the example problems of the HCM, and second, compare the same software packages and the HCM to actual field data. The comparisons in this study were based on the average intersection stopped delay, and also volume-to-capacity ratio, and queues. The various software packages considered for comparison are CINCH88, CINCH94, SIDRA 5.0, SIGCINEMA, SIGNAL94, and Synchro. The field data in this study was collected at five isolated signalized intersection, one actuated-signalized and four pre-timed signalized. The comparison results showed that the average error of HCM 1994 estimates from the field measured delay was 53%. It was finally concluded in the study that the examined software packages do not accurately replicate the field measurements of average stopped delay.

Dion et al. (2004) compared the delay estimates from the 1994 and 1997 versions of HCM, among a number of other delay models, to the delays produced by the INTEGRATION microscopic traffic simulation model. Using a simulation model for comparison rather than field data provided the authors with flexibility to evaluate the consistency of estimation for a range of traffic conditions, from highly under-saturated to highly over-saturated conditions. The study network in the simulation model was a single-lane intersection approach operating with a fixed-time traffic signal. For a volume-to-capacity ratio ranging over 0.1 to 1.4, HCM 1997 model estimated delay that, in general, agreed with the simulation model. HCM 1994 model was found to estimate delay consistently lower that the 1997 model. The study also concluded that both the delay models produce similar results for low traffic demand, but increasing differences are noticed when demand approaches saturation.

Wang et al. (2015) studied the comparison of HCM 2000 delay model to field control delay. The field data required for estimating delay was obtained from three fixed-time signalized intersections in Shanghai. The field control delay in the study was measured by recording the actual travel times of all vehicles and subtracting from it the travel time estimate of a vehicle unaffected by the signal. The comparison of delays was carried out at lane-group level. It was concluded in the paper that the HCM model performed satisfactorily under various volume-to-capacity ratios as the absolute percentage errors were generally smaller than 30% over all lane-groups. Also, it was concluded in the study that signal progression, initial queue, and proportion of vehicles arriving on green were major influential factors to the accuracy of delay models.

In a study performed by Benekohal et al. (2001), the authors studied the similarities and differences between various traffic software packages in computing delay. The software were also compared to determine their delay estimation accuracy. The comparison was performed at approach level as well as intersection level. In the base conditions (or the existing field conditions), control delay from Synchro 4.0 and HCM 97 were not found to be significantly different for fixed-time uncoordinated signals. The readers are referred to this report for more details on the delay comparison and necessary precautions to be taken while comparing different software packages.

It is very important to have accurate field data in the studies of determining delay estimation accuracy. Saito et al. (2001) used image analysis to automate the estimation of average stopped delay per vehicle at signalized intersections. The two proposed methods produced better delay estimates for shorter count intervals, about 10 seconds, and had higher deviation for typical

15 second interval. If the stopped delay data is obtained in such automated fashion, there is a higher possibility of having error as compared to manual method. Anusha et al. (2016) developed an estimation scheme using the Kalman filter to determine the queue and delay at intersections that had erroneous automated data due to noisy detector conditions.

All studies in the past before HCM 1997 used stopped delay as the basis for comparison. However, HCM 1997 used control delay as the criteria for determining level of service. Thus all comparisons there onwards were based on control delay. It is also noted that most studies used aggregated delays for comparison, i.e. either average intersection delay or average approach delay, except Wang et al. (2015) wherein delays were compared on a lane-group basis.

HCM 2010 recommends to compute the average control delay for each lane-group using the following equation,

$$d = d_1 + d_2 + d_3 \tag{2.1}$$

where d_1, d_2, and d_3 are defined as shown in Figures 2.1-2.3 below. The readers are referred to Chapter 18 of HCM 2010 for the methodology of capacity estimation at signalized intersections and for further explanation of the below figures. Chapter 31 of the HCM can be referred for information on the iterative procedure for computing the average duration of an actuated phase.

Figure 2.1 Uniform delay term d1 adopted from Equation 18-20 of HCM 2010

$$\boxed{d_1 = \frac{0.5\,C\,(1 - g/C)^2}{1 - [\min(1, X) g/C]}}$$

Figure 2.2 Incremental delay term d2 adopted from Equations 18-45 and 18-46 of HCM 2010

$$d_2 = 900\,T\left[(X_A - 1) + \sqrt{(X_A - 1)^2 + \frac{8\,k\,I\,X_A}{c_A T}}\right]$$

with

$$X_A = v / c_A$$

Figure 2.3 Initial queue delay term d3 adopted from Equations 18-34 through 18-39 of HCM 2010

$$d_3 = \frac{3{,}600}{vT}\left(t_A \frac{Q_b + Q_e - Q_{eo}}{2} + \frac{Q_e^2 - Q_{eo}^2}{2\,c_A} - \frac{Q_b^2}{2\,c_A}\right)$$

with

$$Q_e = Q_b + t_A(v - c_A)$$

If $v \geq c_A$, then

$$Q_{eo} = T(v - c_A)$$

$$t_A = T$$

If $v < c_A$, then

$$Q_{eo} = 0.0 \text{ veh}$$

$$t_A = Q_b / (c_A - v) \quad \leq T$$

The following chapters discuss the data collection and data reduction performed in this study. It then goes on to present the data analysis and statistical comparison between HCM 2010 and field.

CHAPTER 3

DATA COLLECTION

This chapter describes the study area and presents the methodology used for data collection in the study. Data collection was performed in the fall of 2013 during the months of October, November, and December in Champaign, Illinois.

3.1 Description of Study Area

The study area consists of an urban arterial corridor of six intersections along Neil Street in Champaign as shown in Figure 3.1 below. Starting from north, these are the six intersections of Neil Street with Stadium Drive, Kirby Avenue, St. Mary's Road, Devonshire Drive, Knollwood Drive, and Windsor Road. The signal control at all intersections was operated as an actuated-coordinated system with coordination existing along Neil Street, i.e. northbound and southbound directions. The traffic pattern on Neil Street is such that it has higher volume going northbound towards downtown Champaign during morning and southbound during afternoon. Neil Street also forms the western boundary of the University of Illinois at Urbana-Champaign.

The intersection geometries of each of the 6 intersections are as shown in the schematic diagrams in Figures 3.2-3.7. Please note that these drawings are not scaled.

The traffic pattern on Stadium Drive, Kirby Avenue, St. Mary's Road, and Windsor Road is such that the traffic volume coming out of the westbound approach is higher in the afternoon as compared to the volume going into it, and vice versa in the morning. This is believed to be so because the university is located east of Neil Street and thus people attending work in the morning go into the westbound approach and people leaving work in the afternoon come out of the westbound approach.

The intersection at Neil Street and Devonshire Drive is a T-intersection with the westbound approach not present on Devonshire Drive as shown in Figure 3.5. The traffic demand on its eastbound approach is very low as compared to Neil Street and even other cross streets. The westbound approach of the intersection at Neil Street and Knollwood Drive is only a driveway for a commercial plaza but not a complete approach leading traffic elsewhere. The traffic demand on both eastbound and westbound approaches of Knollwood Drive is also much lower than that of Neil Street and even other cross streets. The remaining four are regular four-legged intersection.

Figure 3.1 The six study intersections on Neil Street in Champaign, IL

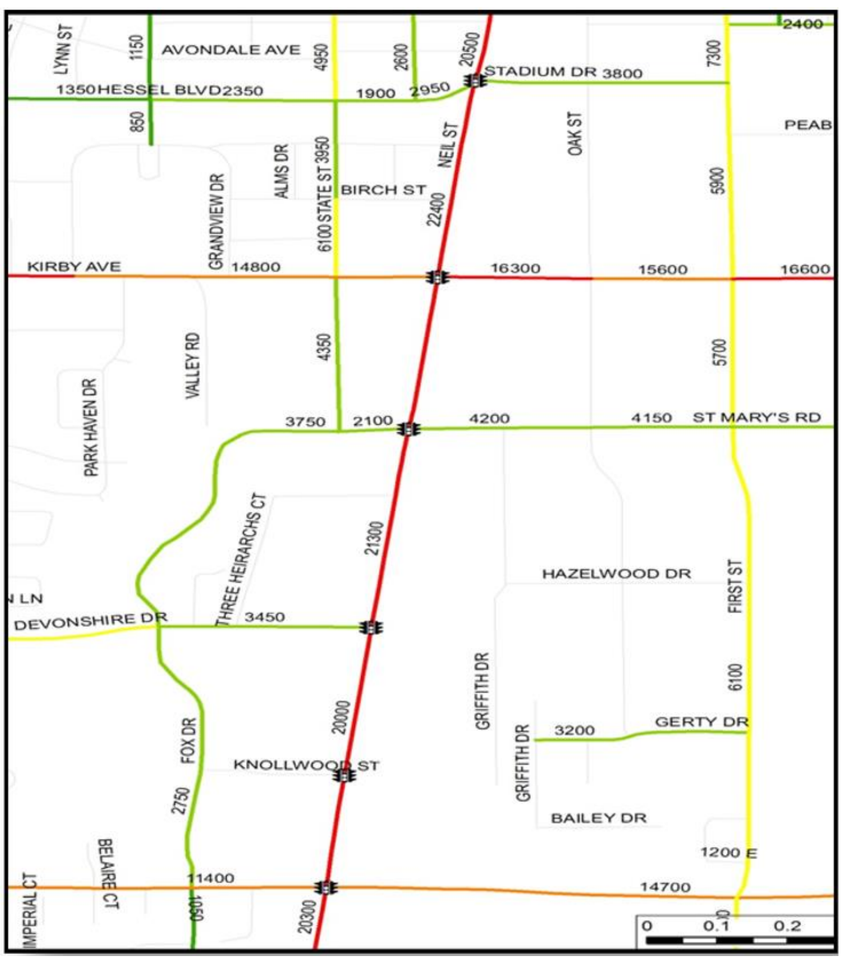


Figure 3.2 Geometry of the intersection of Neil Street and Stadium Drive

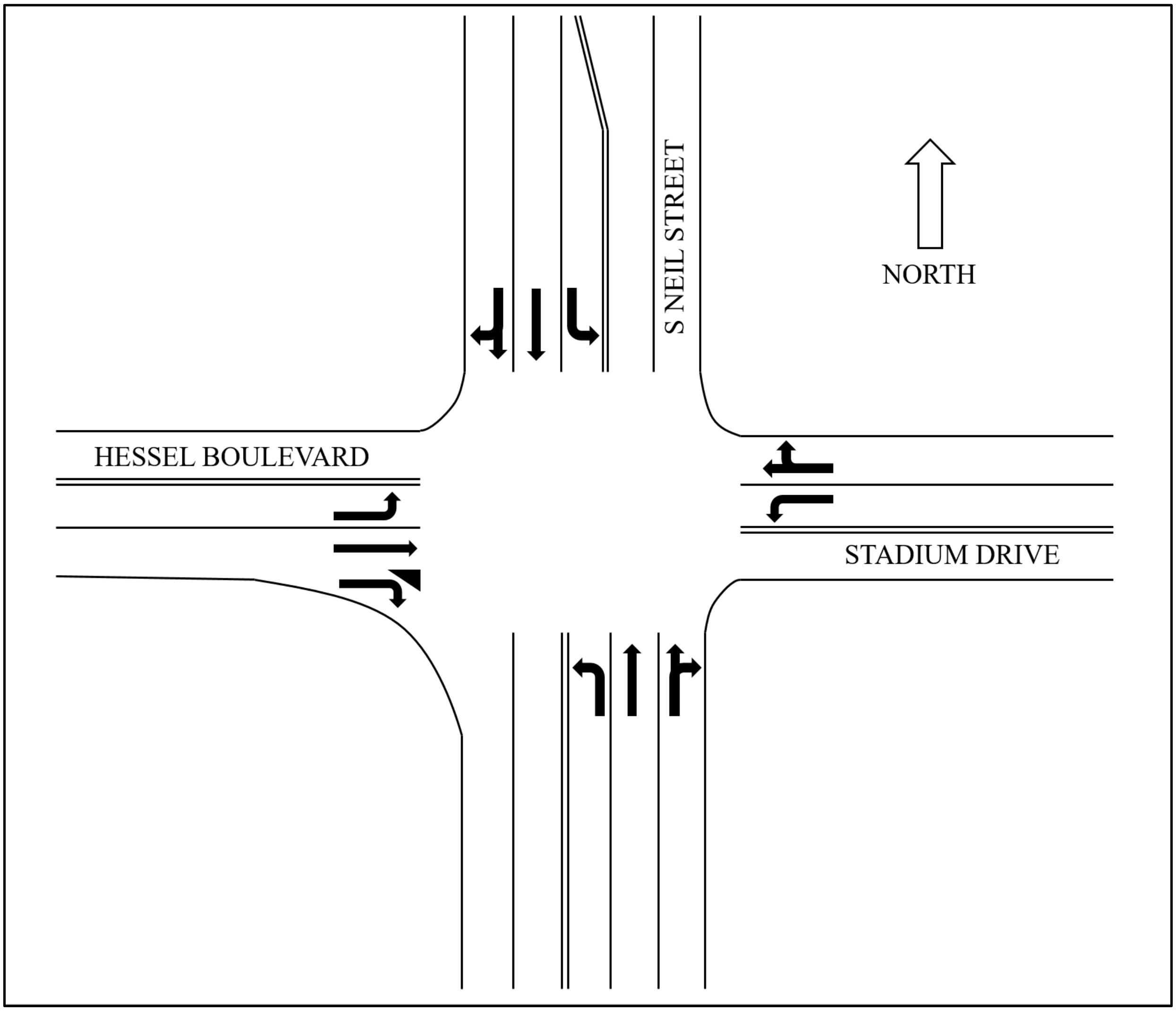


Figure 3.3 Geometry of the intersection of Neil Street and Kirby Avenue

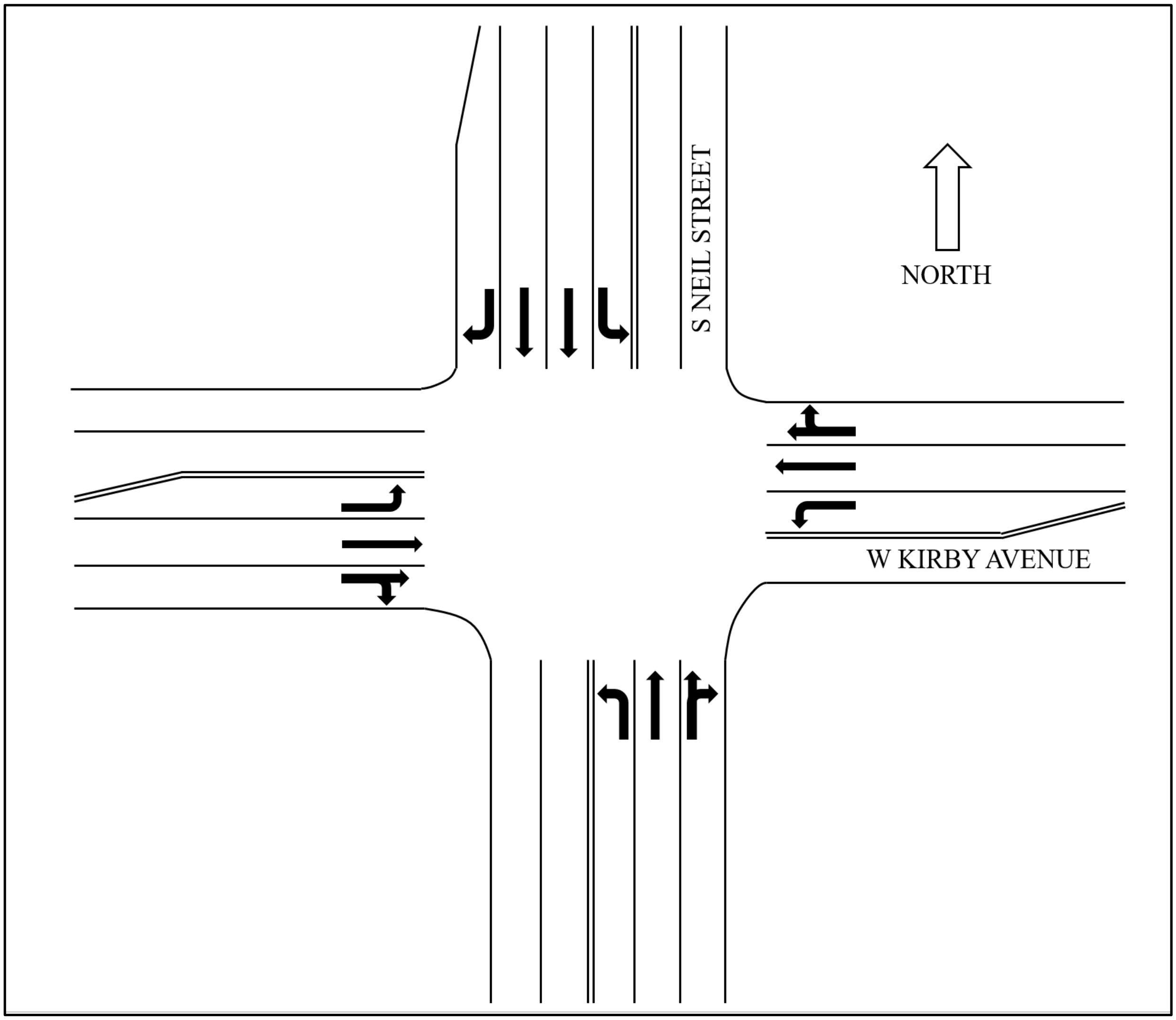


Figure 3.4 Geometry of the intersection of Neil Street and St. Mary's Road

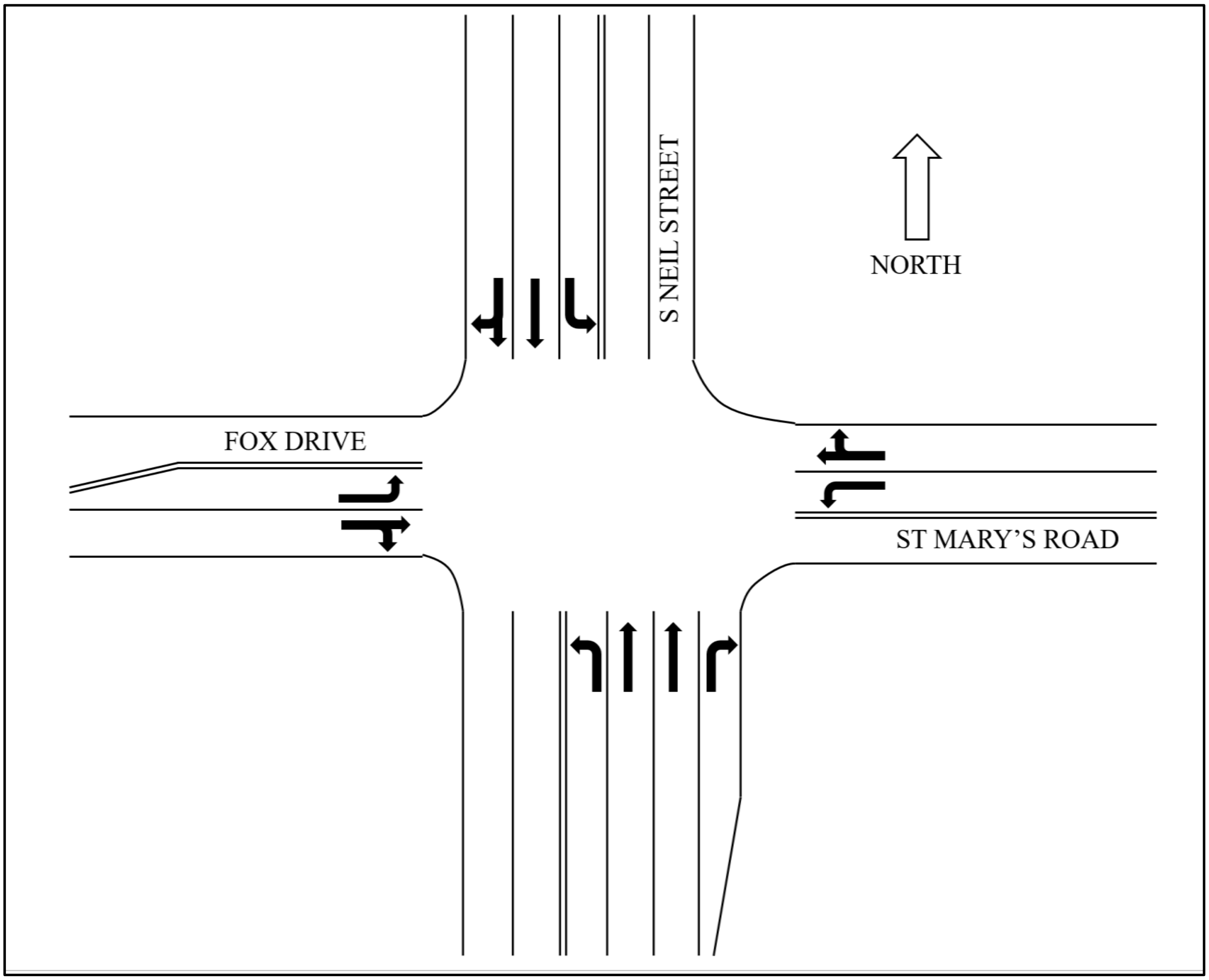


Figure 3.5 Geometry of the intersection of Neil Street and Devonshire Drive

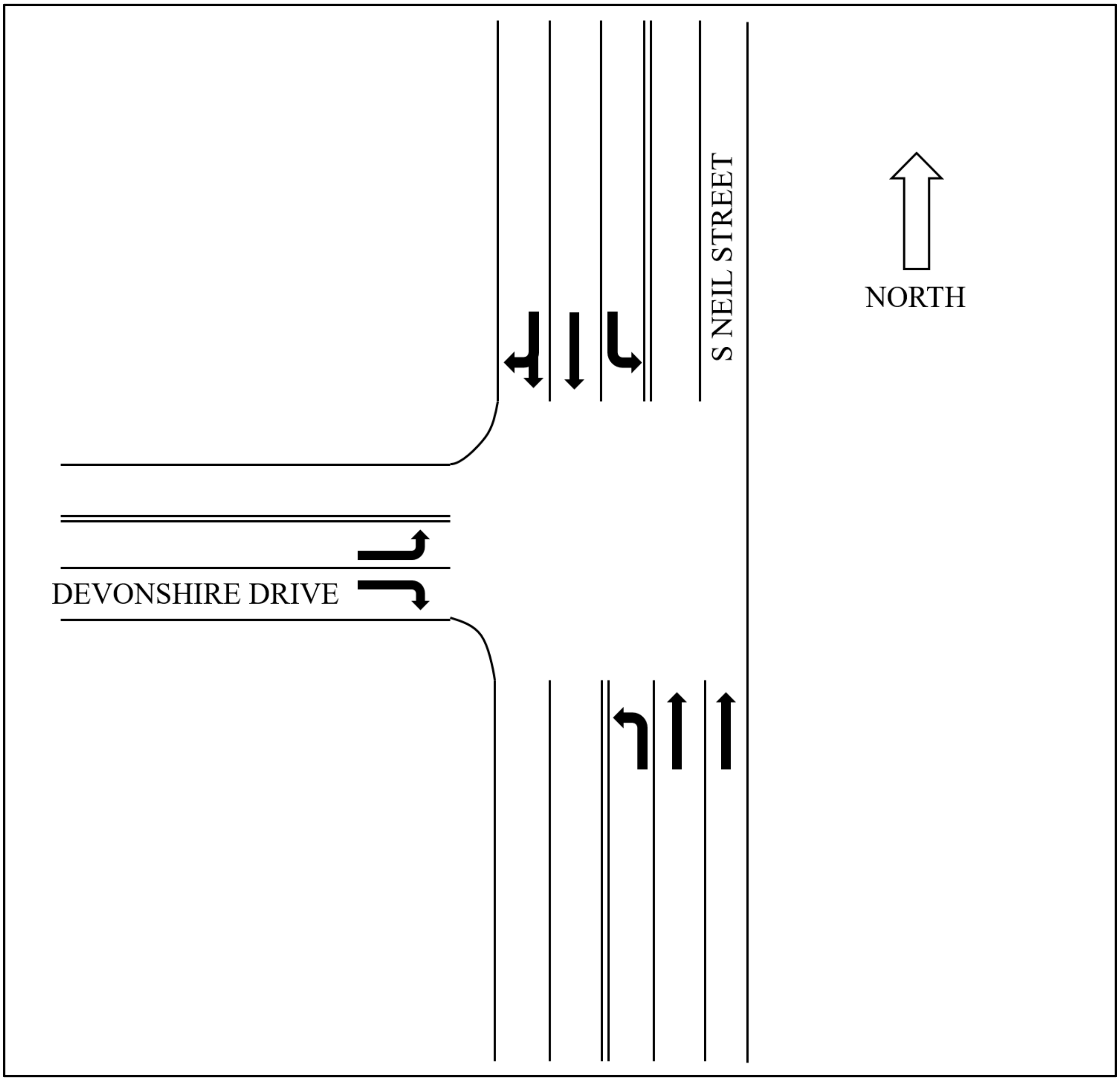


Figure 3.6 Geometry of the intersection of Neil Street and Knollwood Drive

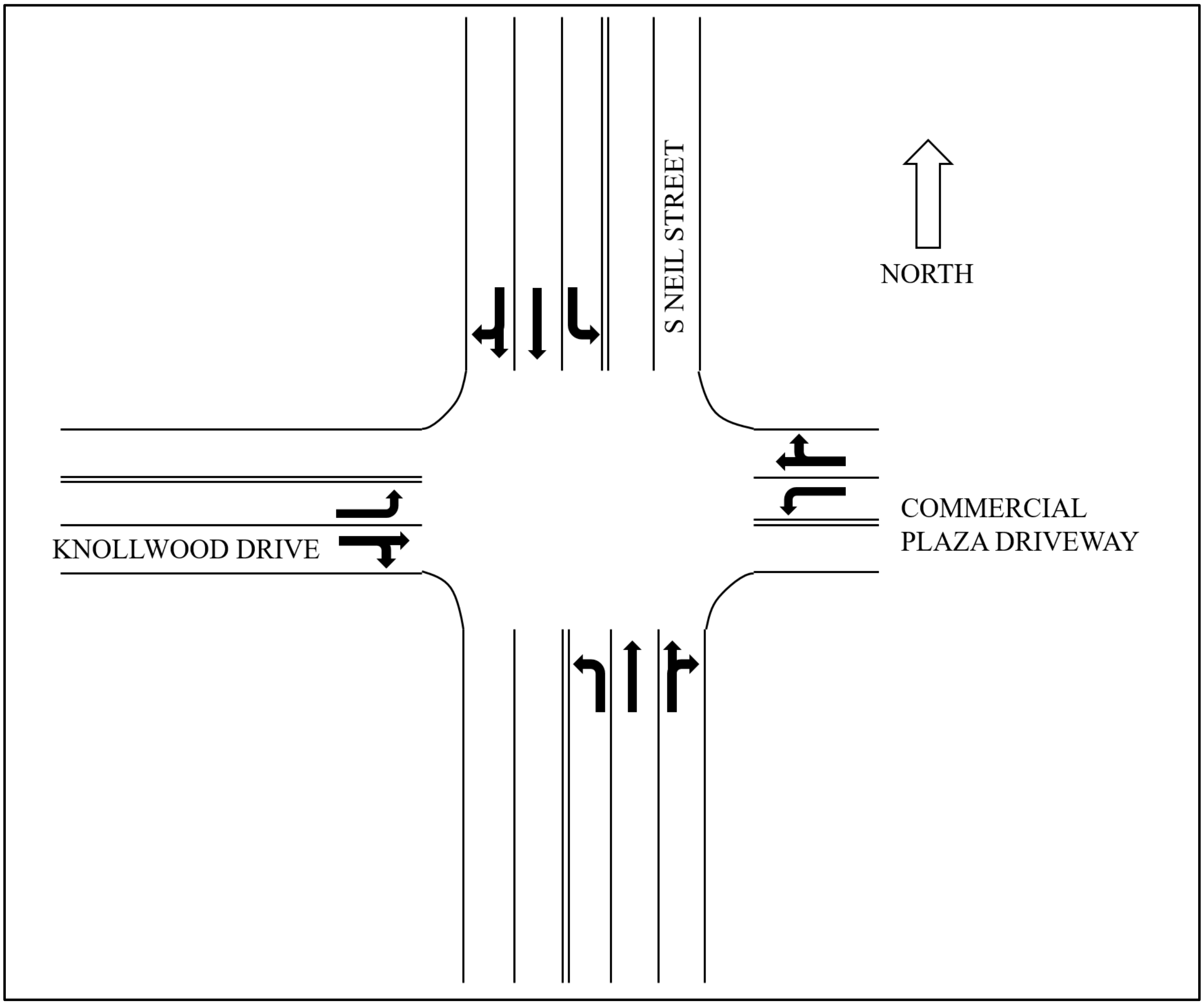
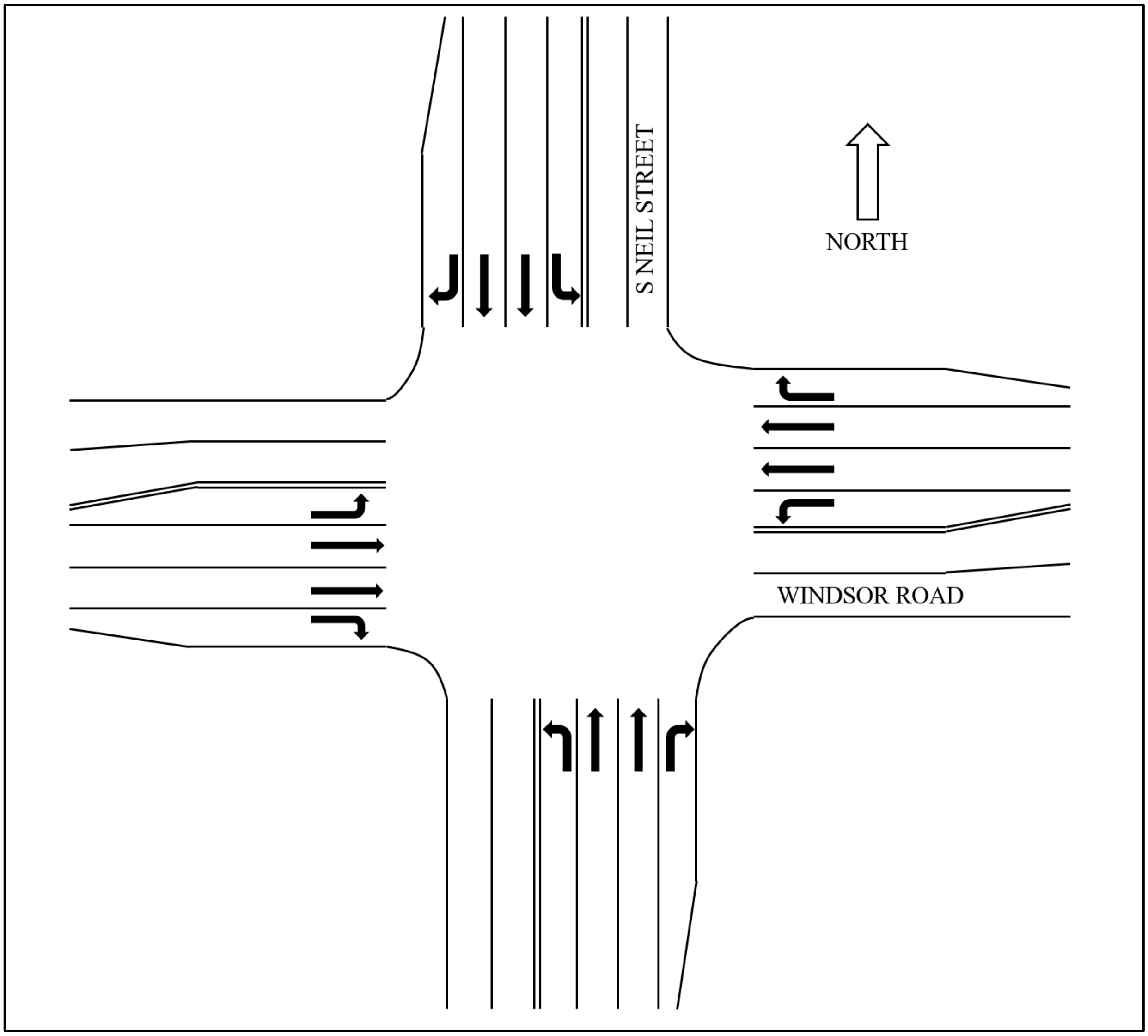


Figure 3.7 Geometry of the intersection of Neil Street and Windsor Road



3.2 Data Collection Methodology

Data collection was performed in the field by recording traffic data using video cameras. A separate camera was setup for each approach of an intersection. With the idea of performing data analysis for a morning peak, a noon peak, an afternoon peak, and an off peak hour, data was recorded during multiple time periods in a day as shown in Table 3.1. The data was collected on a different day for each intersection as shown in Table 3.2.

Table 3.1 Data collection time periods of a day

TIME PERIOD	DATA COLLECTED FOR
Morning	7:00 AM – 9:00 AM
Noon	10:30 AM – 1:30 PM
Afternoon	4:00 PM – 6:00 PM

Table 3.2 Data collection dates of the study

INTERSECTION	DATE	DAY
Neil Street & Stadium Drive	November 7, 2013	Thursday
Neil Street & Kirby Avenue	November 13, 2013	Wednesday
Neil Street & St. Mary's Road*	November 20, 2013	Wednesday
	December 11, 2013	Wednesday
Neil Street & Devonshire Drive	October 29, 2013	Tuesday
Neil Street & Knollwood Drive	November 12, 2013	Tuesday
Neil Street & Windsor Road	November 5, 2013	Tuesday

* PM data at this intersection was obtained on Dec 11, 2013 due to unavailability on November 20, 2013

The traffic videos recorded during the aforementioned dates and time periods were then reduced for further analysis as described in the next chapter.

CHAPTER 4

DATA REDUCTION

This chapter describes the methodology used for reducing the traffic videos and presents the data obtained for each traffic characteristic of interest. The items of data reduction were as following:

a) Peak hours
b) Hourly volume
c) Field delay
d) Saturation flow rate
e) Signal timing
f) Arrival type

The data reduction was performed for specific time periods within the videos for all intersections. These time periods are as shown in Table 4.1.

Table 4.1 Data reduction time periods in the videos

TIME PERIOD	DATA REDUCED FOR
Morning	7:10 AM – 8:40 AM
Noon	10:40 AM – 1:15 PM
Afternoon	4:40 PM – 6:00 PM

A description of each item in the data reduction list, the methodology used for reducing it, and outcome of the data obtained are provided in the following sections.

4.1 Peak Hour

The peak hours during morning, noon and afternoon time periods were determined in order to reduce other traffic data and perform the desired analysis for these hours. An off peak hour was also selected for the same.

4.1.1 Methodology

The thru movement volumes on Neil Street were manually counted from the traffic videos recorded at the intersections of Neil Street with Stadium Drive, Kirby Avenue, St. Mary's Road, and Windsor Road. This was done for the three data reduction time periods as mentioned in Table 3.3 above and 2-minute volumes were obtained for each.

The hour in the morning time period having the highest northbound total thru volume at the abovementioned four intersections was decided to be the AM peak hour. Similarly, the hour in the afternoon time period having the highest southbound total thru volume at these four intersections was decided to be the PM peak hour. The noon peak hour was the hour corresponding to the highest total thru volume in both north and southbound directions at these four intersections in the noon time period. The off peak selected was an hour from the beginning of the noon time period.

4.1.2 Data

The peak hours computed using the above methodology are as shown in in Table 4.2 below.

Table 4.2 Peak hours determined in the study

TIME PERIOD	PEAK HOUR
AM Peak	7:30 AM – 8:30 AM
Noon Peak	12:10 AM – 1:10 PM
PM Peak	4:40 PM – 5:40 PM
Off Peak	10:40 AM – 11:40 AM

4.2 Hourly Volume

The left, thru, and right turning movement volumes during the above peak hours were determined for all approaches of the six intersections. These hourly volumes can be used to perform capacity analysis.

4.2.1 Methodology

The turning movement volumes were manually counted from the traffic videos recorded at all intersections for the duration of the determined peak hours. These volume counts were obtained in the interval of 15 seconds for the entire hour.

4.2.2 Data

The hourly volume counts during the three peak hours and the off peak hour are presented in Table 4.3 below. It is evident from the data that northbound traffic volume is higher than southbound in the AM peak hour and vice-versa in the PM peak hour at all intersections. It is also apparent from Table 4.3 that the demand on cross streets at the intersections of Neil Street with Devonshire Drive and Knollwood Drive are much lower. The cells having entries of N/A in the

Table 4.3 Hourly volume counts reduced

Intersection	Time Period	NB			SB			EB			WB		
		LT	THRU	RT	LT	THRU	RT	LT	THRU	RT	LT	THRU	RT
Neil St & Stadium Dr	AM Peak	53	903	29	60	570	13	38	173	83	10	36	24
	Off Peak	23	553	30	41	559	20	26	28	29	22	28	32
	Noon Peak	37	739	49	57	838	19	34	52	53	51	60	60
	PM Peak	31	842	23	50	1025	26	28	38	62	69	181	93
Neil St & Kirby Ave	AM Peak	88	824	118	153	516	47	150	678	131	91	215	51
	Off Peak	101	532	64	71	514	99	121	248	93	116	183	88
	Noon Peak	145	642	104	135	751	115	108	353	141	153	252	88
	PM Peak	170	719	112	159	960	139	124	466	142	155	589	108
Neil St & St Mary’s Rd	AM Peak	33	888	153	171	531	68	17	81	35	22	36	42
	Off Peak	18	577	42	63	631	34	33	46	42	53	24	83
	Noon Peak	25	808	70	110	822	50	26	49	61	55	42	127
	PM Peak	23	652	53	24	1043	29	47	46	87	131	76	194
Neil St & Devonshire Dr	AM Peak	76	1095	N/A	N/A	452	38	70	N/A	24	N/A	N/A	N/A
	Off Peak	28	531	N/A	N/A	655	5	54	N/A	31	N/A	N/A	N/A
	Noon Peak	47	741	N/A	N/A	729	41	51	N/A	51	N/A	N/A	N/A
	PM Peak	35	582	N/A	N/A	1182	79	53	N/A	74	N/A	N/A	N/A
Neil St & Knollwood Dr	AM Peak	96	1152	8	19	442	29	6	1	14	4	1	10
	Off Peak	43	534	11	35	538	29	25	2	44	18	1	20
	Noon Peak	72	662	16	46	703	38	39	9	93	28	7	61
	PM Peak	23	555	9	27	1268	21	11	1	93	21	2	24
Neil St & Windsor Rd	AM Peak	79	899	264	70	320	77	241	625	68	130	331	186
	Off Peak	60	430	127	82	445	96	108	243	57	136	240	101
	Noon Peak	89	520	175	134	575	146	144	268	83	167	260	131
	PM Peak	67	387	140	237	877	235	120	370	85	298	659	119

table at the intersection of Neil Street and Devonshire Drive (T-intersection) signify that the respective lane group was not present at the subject approach.

4.3 Field Delay

The control delay and stopped delay in field were calculated from the video data. The field measurements presented in this section will later be compared to its estimates obtained through capacity analysis in Chapter 5.

4.3.1 Methodology

The field measurement technique of intersection control delay as described in Chapter 31 of HCM 2010 was adopted to calculate time-in-queue, i.e. stopped delay, and control delay using the field videos. The measurements were carried out on a lane group basis for each approach of the six intersections. The procedure was performed for all four time periods.

The procedure requires to identify the approach speed during each study period. The speed limit of each approach in the field was assumed to be its approach speed for each intersection. The duration of survey period was essentially equal to one hour for each peak hour and the off peak hour. The count interval of 15 seconds was selected for this study which is indeed an integral divisor of the duration of survey period (one hour) as required by the HCM.

4.3.2 Data

The control delay and stopped delay obtained for each lane group in the study are using the HCM field measurement methodology are presented in Table 4.4 and Table 4.5 respectively. The cells having entries of N/A in the tables signify that the respective lane group was not present at the subject approach.

Table 4.4 Control delay at lane group level calculated using the HCM 2010 field measurement technique

Intersection	Time Period	NB			SB			EB			WB		
		LT	THRU	RT	LT	THRU	RT	LT	THRU	RT	LT	THRU	RT
Neil St & Stadium Dr	AM Peak	16.2	5.0	N/A	33.6	7.3	N/A	8.5	14.3	N/A	18.0	13.7	N/A
	Off Peak	10.8	3.3	N/A	15.5	6.8	N/A	29.5	26.9	N/A	20.4	19.0	N/A
	Noon Peak	16.2	3.0	N/A	20.9	4.2	N/A	19.5	18.5	N/A	19.3	16.1	N/A
	PM Peak	30.4	5.8	N/A	22.0	8.5	N/A	32.5	13.2	N/A	19.2	13.5	N/A
Neil St & Kirby Ave	AM Peak	20.9	19.0	N/A	39.5	18.9	2.2	19.1	19.7	N/A	35.2	39.6	N/A
	Off Peak	18.9	21.2	N/A	23.6	19.4	3.9	27.9	25.1	N/A	21.7	19.4	N/A
	Noon Peak	31.0	24.8	N/A	27.2	24.0	3.6	25.7	22.0	N/A	21.5	19.9	N/A
	PM Peak	51.4	28.6	N/A	28.5	21.6	3.1	28.6	24.2	N/A	37.4	34.8	N/A
Neil St & St Mary’s Rd	AM Peak	17.5	6.9	2.9	25.7	11.4	N/A	43.6	38.7	N/A	29.9	26.2	N/A
	Off Peak	2.3	3.7	1.0	5.8	2.7	N/A	33.0	33.7	N/A	29.8	33.4	N/A
	Noon Peak	22.0	7.1	2.1	14.7	3.6	N/A	35.1	27.6	N/A	33.5	20.9	N/A
	PM Peak	5.0	9.5	4.3	26.5	6.1	N/A	46.2	37.1	N/A	38.2	30.1	N/A
Neil St & Devonshire Dr	AM Peak	5.3	1.3	N/A	N/A	1.7	N/A	48.5	10.2	N/A	N/A	N/A	N/A
	Off Peak	17.4	3.9	N/A	N/A	1.3	N/A	41.6	10.2	N/A	N/A	N/A	N/A
	Noon Peak	6.3	1.6	N/A	N/A	1.1	N/A	48.9	17.1	N/A	N/A	N/A	N/A
	PM Peak	18.9	1.6	N/A	N/A	1.4	N/A	48.6	19.8	N/A	N/A	N/A	N/A
Neil St & Knollwood Dr	AM Peak	5.2	0.6	N/A	10.3	1.6	N/A	35.4	18.0	N/A	36.5	11.8	N/A
	Off Peak	6.3	0.9	N/A	4.1	0.3	N/A	49.1	11.9	N/A	61.0	10.3	N/A
	Noon Peak	5.7	1.3	N/A	6.6	1.3	N/A	41.7	19.2	N/A	36.9	15.4	N/A
	PM Peak	17.5	0.6	N/A	4.6	1.5	N/A	43.4	19.7	N/A	40.1	8.2	N/A
Neil St & Windsor Rd	AM Peak	5.2	12.1	5.0	23.8	8.0	1.7	25.0	17.8	2.4	29.6	26.2	7.7
	Off Peak	15.0	11.4	3.8	16.5	7.7	2.1	25.6	27.1	9.9	24.2	23.6	7.1
	Noon Peak	12.5	17.5	2.8	6.5	9.9	15.7	26.8	27.3	10.6	28.6	24.3	4.5
	PM Peak	30.7	18.6	4.4	31.3	14.2	5.5	29.5	32.2	19.7	36.1	30.1	5.9

Table 4.5 Stopped delay at lane group level calculated using the HCM 2010 field measurement technique

Intersection	Time Period	NB			SB			EB			WB		
		LT	THRU	RT	LT	THRU	RT	LT	THRU	RT	LT	THRU	RT
Neil St & Stadium Dr	AM Peak	13.2	3.7	N/A	29.9	5.4	N/A	6.0	10.9	N/A	13.5	10.5	N/A
	Off Peak	8.8	2.3	N/A	12.8	5.0	N/A	21.3	23.1	N/A	17.2	14.8	N/A
	Noon Peak	13.1	2.0	N/A	17.8	2.8	N/A	15.9	14.8	N/A	15.4	13.1	N/A
	PM Peak	26.0	4.1	N/A	18.1	6.3	N/A	27.8	10.2	N/A	15.5	10.5	N/A
Neil St & Kirby Ave	AM Peak	19.5	17.7	N/A	34.8	16.0	0.9	16.1	18.7	N/A	31.0	35.2	N/A
	Off Peak	15.6	18.0	N/A	20.0	16.7	2.7	24.3	22.0	N/A	18.2	16.8	N/A
	Noon Peak	26.5	23.2	N/A	23.2	22.7	2.2	22.0	20.5	N/A	22.0	17.0	N/A
	PM Peak	46.7	27.2	N/A	24.1	20.7	2.4	25.1	21.8	N/A	33.6	33.0	N/A
Neil St & St Mary's Rd	AM Peak	14.7	5.8	2.0	21.5	8.7	N/A	38.9	34.2	N/A	25.8	22.2	N/A
	Off Peak	1.5	2.7	0.6	4.3	2.0	N/A	29.0	29.6	N/A	26.0	29.8	N/A
	Noon Peak	18.4	5.7	0.4	12.0	2.8	N/A	30.6	23.7	N/A	29.2	17.2	N/A
	PM Peak	3.5	7.7	2.5	22.5	5.0	N/A	41.4	32.5	N/A	33.9	26.2	N/A
Neil St & Devonshire Dr	AM Peak	4.1	0.9	N/A	N/A	1.2	N/A	44.0	5.6	N/A	N/A	N/A	N/A
	Off Peak	14.0	2.9	N/A	N/A	1.0	N/A	37.0	5.7	N/A	N/A	N/A	N/A
	Noon Peak	5.2	1.1	N/A	N/A	0.8	N/A	44.5	12.2	N/A	N/A	N/A	N/A
	PM Peak	16.2	1.1	N/A	N/A	0.9	N/A	44.3	15.4	N/A	N/A	N/A	N/A
Neil St & Knollwood Dr	AM Peak	2.9	0.3	N/A	6.8	1.0	N/A	30.4	13.5	N/A	31.5	9.6	N/A
	Off Peak	4.4	0.5	N/A	1.9	0.1	N/A	44.3	9.1	N/A	56.3	7.7	N/A
	Noon Peak	3.2	0.7	N/A	3.8	0.7	N/A	37.0	15.8	N/A	32.8	12.3	N/A
	PM Peak	12.3	0.3	N/A	3.0	0.7	N/A	39.3	15.8	N/A	35.4	6.2	N/A
Neil St & Windsor Rd	AM Peak	3.6	10.0	3.4	19.8	6.5	1.2	21.5	15.2	1.6	25.9	23.2	6.3
	Off Peak	11.7	9.2	2.8	13.7	6.3	1.4	21.9	23.8	7.8	20.8	20.4	5.1
	Noon Peak	9.1	14.5	1.9	4.8	8.2	12.6	23.1	24.1	8.5	24.9	21.3	3.1
	PM Peak	26.2	16.1	3.1	27.0	12.3	3.6	25.7	27.1	16.2	32.3	26.5	4.6

4.4 Saturation Flow Rate

Field saturation flow rate of thru lanes was measured to be used in the HCM delay estimation procedure. It is one of the input variables of the computation.

4.4.1 Methodology

The field measurement technique for measuring saturation flow rate as described in Chapter 31 of HCM 2010 was adopted to calculate the base saturation flow rate of thru lanes for local conditions of the study area. The base saturation flow rate of thru lanes did not require any adjustment for the thru direction. However, it was later adjusted in the computation process for left turns and right turns.

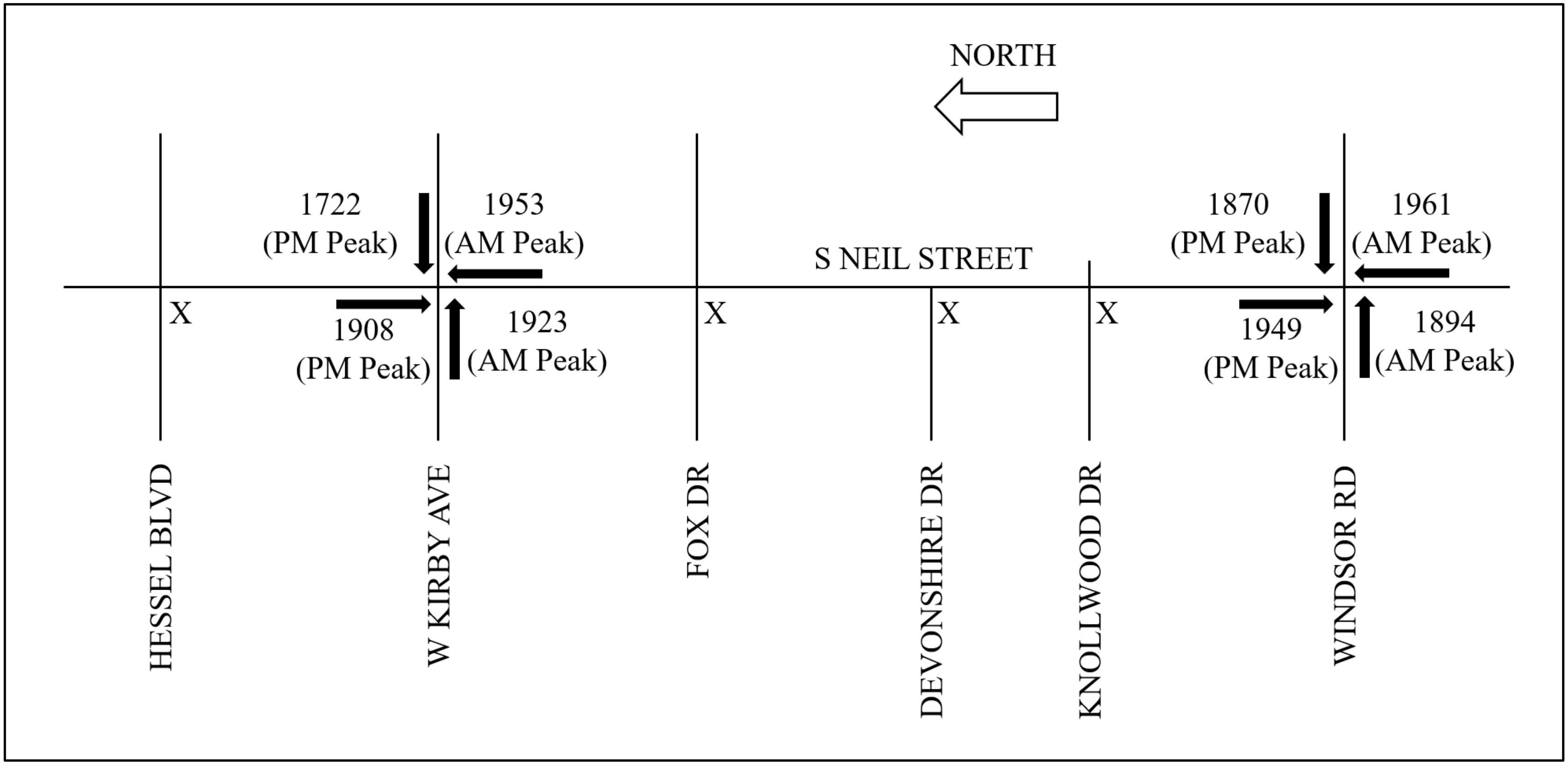
A minimum of 50 "valid" headways were sought for an approach in order to obtain a stable result. A "valid" headway is a headway of any vehicle from the fifth to the last vehicle in queue as required in the HCM field measurement technique. The technique was applied to all approaches of the six intersections.

4.4.2 Data

The saturation flow rate calculated for the thru lanes using the above procedure is presented in Figure 4.1 below. As shown in the figure, all intersections did not have adequate data to determine the saturation flow rate. Only the intersections of Neil Street with Kirby Avenue and Windsor Road were saturated enough.

Based on the available data and knowledge of the intersection geometries, it was decided to use a saturation flow rate of 1900 pcphgpl for northbound and southbound thru lanes on Neil Street at all intersections. For westbound thru lanes at all intersections, it was decided to use a value of 1750 pcphgpl except Windsor Road where 1900 pcphgpl was used. A value of 1900

Figure 4.1 Saturation Flow rate of thru lanes calculated using the HCM 2010 field measurement technique



X = Intersection did not have adequate data for measuring saturation flow rate
NOTE: All saturation flow rates in the figure are reported in passenger cars per hour of green per lane (pcphgpl)

pcphgpl was used for eastbound thru lanes of Kirby Avenue, Devonshire Drive and Windsor Road and 1750 pcphgpl was used for eastbound thru lanes of the remaining three cross streets.

4.5 Signal Timing

This section discusses the methodology used for obtaining the signal timing data, which include cycle lengths, yellow change, red clearance, and passage time. The data was obtained using the field videos as well as the signal controller settings to be used for carrying out the HCS runs.

4.5.1 Methodology

Cycle lengths were measured using a stopwatch from the videos of each peak hour for all six intersections. The yellow change, red clearance, minimum green, and passage time for each phase were obtained directly from the signal controller settings data. Cycle length measured from the field were also verified with the ones in the controller settings. All recurring signal phases observed in the field videos were allowed in the phasing section of the HCS runs to be given a green time.

4.5.2 Data

The cycle lengths during the each peak hour are as shown in Table 4.6 below. The cycle length was indeed equal for all intersections in a given peak hour because of the coordinated signalized operation, except for the intersection of Neil Street and Stadium Drive. The signal at this intersection was operating at half-cycle with respect to the other five intersections.

The yellow change values were 3.2, 3.6, or 3.9 seconds and the red clearance was in the order of 2 seconds for different phases. The passage time for each phase was directly taken from the controller settings.

Table 4.6 Cycle length during each peak hour

PEAK HOUR	CYCLE LENGTH
AM Peak	110 seconds
Noon Peak	110 seconds
PM Peak	120 seconds
Off Peak	110 seconds

4.6 Arrival Type

The field arrival types were estimated to be used as inputs in the capacity estimation instead of default values.

4.6.1 Methodology

Random arrival, i.e. arrival type 3, was assumed for all movements on the cross streets and for left turn movements from Neil Street at all intersections. The arrival type for thru movements on Neil Street at all intersections was estimated based on the proportion of vehicles stopped at each intersection and also by viewing of video to check when the platoons arrived during the cycle.

Based on field observation, arrival type of 1, 5, and 6 were practically not present on Neil Street thru movements at any intersection. Thus only arrival type of 2, 3, and 4 were considered for these movements. The proportion of vehicle stopped for a subject thru movement was used to compute the platoon ratio and thus obtain the arrival type using Exhibit 18-8 of HCM 2010.

4.6.2 Data

The arrival types determined for Neil Street thru movements are as shown in Table 4.7 below. In the table, NBT stands for northbound thru movement and SBT stands for southbound thru movement. As mentioned in the methodology above, the arrival type of all remaining movements in the study, which are Neil Street left turn movements and all cross street turning movements, is 3 for all four time periods.

Table 4.7 Arrival types determined on Neil Street thru movements

Intersection	AM Peak		Off Peak		Noon Peak		PM Peak	
	NBT	SBT	NBT	SBT	NBT	SBT	NBT	SBT
Neil St & Stadium Dr	4	4	4	4	4	4	4	4
Neil St & Kirby Ave	3	4	3	4	2	3	2	4
Neil St & St Mary's Rd	4	2	4	4	3	4	3	4
Neil St & Devonshire Dr	4	4	4	4	4	4	4	4
Neil St & Knollwood Dr	4	4	4	4	4	4	4	4
Neil St & Windsor Rd	4	4	4	4	3	4	4	4

CHAPTER 5

DATA ANALYSIS

This chapter explains the data analysis performed in the study. The analysis consists of two main parts – capacity analyses and statistical comparison. First the methodology used for comparison is explained. It is followed by the demonstration of capacity analysis runs and the statistical comparison. It then goes on to discuss the results and check the validity of the general trend for each intersections and time of day.

5.1 Methodology for Comparison

The comparison in this study was done between HCM stopped delay estimates and respective field measurements on a lane-group basis. The lane groups considered are protected left-turn lanes, thru lanes, and protected right-turn lanes only. The reasoning for using stopped delay rather than control delay for the purpose of comparison, and not considering permitted and protected-permitted left- and right-turn lane groups is explained later in this section.

Statistical comparison was performed using one-sample *t*-test at a level of significance of 0.10 for a two-tailed hypothesis. The null hypothesis of the test was that the HCM estimate was equal to the field measurement. The *t*-statistic used to perform the test is as shown in equation 5.1 below.

$$t = \frac{\bar{x} - \mu 0}{s/\sqrt{n}} \tag{5.1}$$

In this equation, $\mu 0$ is the HCM stopped delay estimate of the subject lane group. $\overline{x}$ is the average stopped delay per vehicle of that lane group observed from field and s^2 is its variance. The field variance of stopped delay of a lane group was obtained by measuring average three-minute stopped delays during each peak hour and then computing the variance. So, each lane group ideally had 20 stopped delays during every peak hour (60 minutes), and the variance of these 20 observations is equal to the field variance s^2. The observation time of three minutes was deliberately chosen in order to contain traffic data of at least one complete cycle (110 or 120 seconds) in each one.

Thus, using the above methodology, the differences of comparison were tested for significance. The following subsections provide the reasoning for performing stopped delay comparison and for not considering permitted and protected-permitted turns in the analysis.

5.1.1 Comparison using Stopped Delay

In the procedure of HCM field delay measurement, time-in-queue per vehicle or stopped delay is first estimated for a subject lane group. Then HCM recommends the use of a correction factor to adjust stopped delay for deceleration and acceleration delay and thus obtain the estimate of control delay for that lane group. The value of correction factor can be obtained by looking-up Exhibit 31-48 in Chapter 31 of HCM 2010.

Stopped delay value of a lane group is more directly obtained from field and does not contain any corrections as that in control delay calculation. Also, the HCM estimate of control delay includes an adjustment factor of 1.3 in uniform delay and incremental delay components. This factor is essentially meant to increase the stopped delay by 30 % to account for the deceleration and acceleration delay. It was thus decided to compare stopped delay between field and HCM estimates because it is more meaningful for assessing the accuracy of the HCM delay model and appropriate to avoid unnecessary error due to corrections.

5.1.2 Permitted and Protected-Permitted Lane Groups

The field delay of permitted and protected-permitted turning movements is highly determined by their arrival and the time to find a gap to depart. Especially, the delay of permitted and protected-permitted left turn is correlated with volume and arrival of the opposing thru movements and thus easily varies with the possibility to find a gap during green. The delay of permitted right-turn verily depends on the right turn on red (RTOR) volume and the possibility of finding gaps for the vehicles to leave during red. It is thus believed that these influencing factors can cause substantial error in the comparison. Therefore, it was decided not to consider permitted and protected-permitted lane groups for this study.

5.2 Capacity Analyses

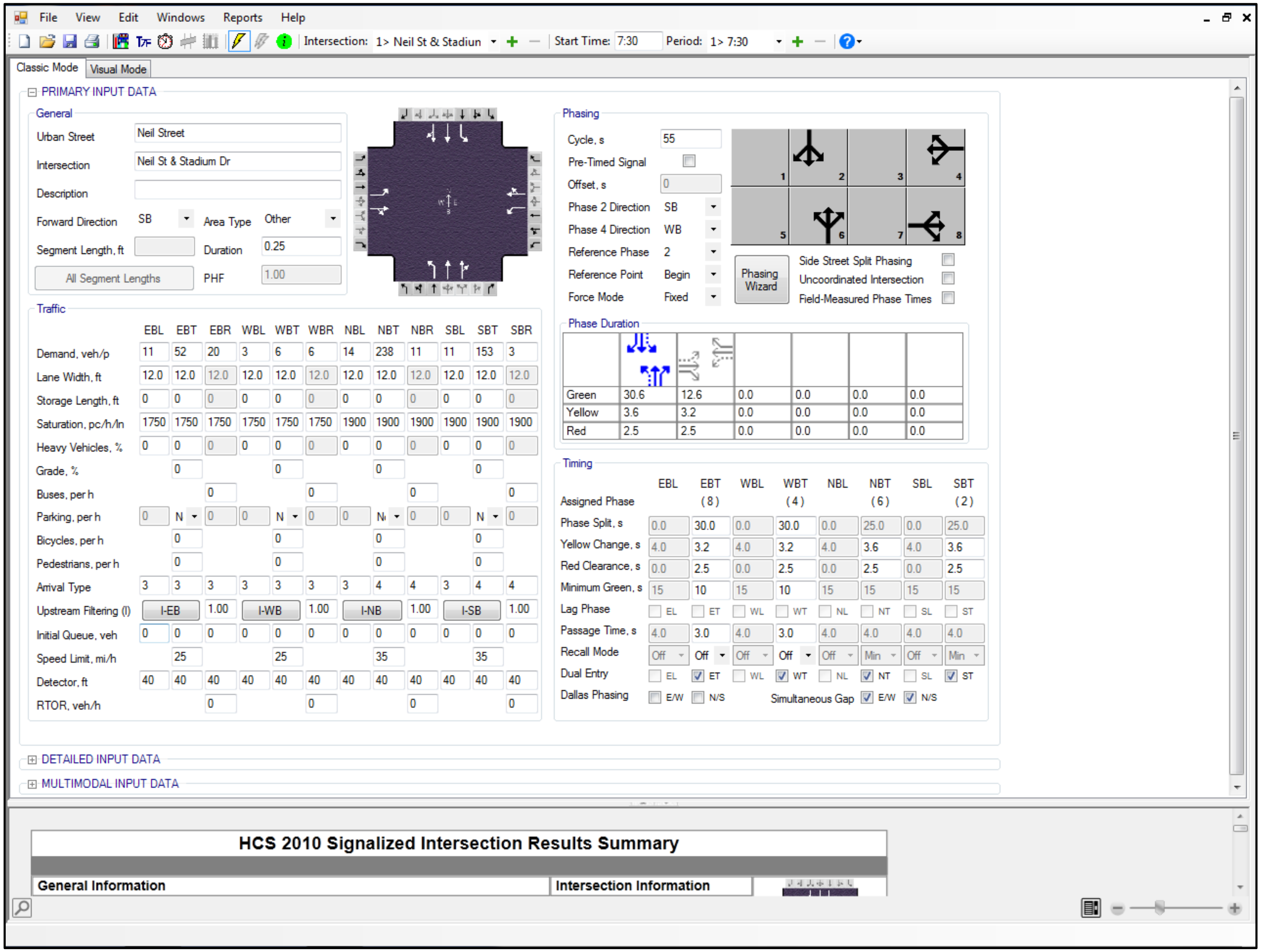
HCS 2010 version 6.70 was used to perform capacity analysis for all intersections as per the HCM 2010 methodology. Individual HCS models were developed for each intersection instead of a single corridor model because the current HCS cannot accommodate intersections operating at different cycle lengths in the same model. Thus, although coordinated in the field, the intersections were analyzed in HCS assuming isolated actuated-signalized operation for the existing field conditions.

Figure 5.1 below is a screenshot of a typical HCS run of an intersection carried out in the analysis. The intersection shown in the figure is at Neil Street and Stadium Drive and the input data corresponds to the AM peak period, i.e. 7:30 AM – 8:30 AM. As per the recommendation of the HCM, a multi-period analysis was performed in HCS for an analysis duration of 15 minutes or 0.25 hour.

The field data reduced in the previous chapter was used as the inputs for demand, saturation flow rate, phasing, signal timing, and arrival types in the HCS runs. The demand of multi-period analysis was equal to the 15-minute aggregated volume counts for each movement. In Figure 5.1, the cycle length at this intersection is 55 seconds, instead of 110 seconds, because it operates at a half-cycle with respect to the other five intersections as mentioned earlier. The signal can be coded as an actuated operation by leaving the ‘Pre-Timed Signal’ box in the ‘Phasing’ section unchecked as done in the figure. The signal settings in the ‘Timing’ section such as yellow change, red clearance, minimum green, and passage time, were set directly equal to the values as programmed in the field controllers.

When the analysis is run, HCS determines a green time allocation based on the iterative procedure for determining average duration of an actuated phase in Chapter 31 of HCM 2010, which is believed to be representative of the actuated signal operation. This signal timing is used to further carry out the capacity analysis. The HCM control delay estimate of each movement was calculated as the volume-weighted average of the 15-minute control delays, for all time periods. Furthermore, the corresponding stopped delay was obtained by reducing the control delay estimate by 30 percent, i.e. dividing the latter by a factor of 1.3. These stopped delays were used in the statistical analysis as described in the next section.

Figure 5.1 Screenshot of HCS run showing the input data at Neil St and Stadium Dr for AM peak period



5.3 Statistical Comparison

Using the aforementioned methodology for comparison, the *t*-tests were performed for all thru and protected left-turning lane groups in the study area for the four time periods. There were 20 thru lane groups present at the six intersections (the lane groups on Devonshire Drive and Knollwood Drive do not classify as thru). There was only 1 protected left-turning lane group present on the eastbound approach of the intersection at Neil Street and Devonshire Drive.

The details of the *t*-tests performed are presented in Table 5.1 below. The column heading "n" in the table stands for the number of 3-minute delay observations obtained from field for the subject lane group. The column heading "df" stands for degrees of freedom of the *t*-test which is basically equal to the number of observations minus one, i.e. n-1. The other columns show the HCM estimates, field measurements, *t*-statistics, and p-values. NBT, SBT, EBT, and WBT stand for northbound thru, southbound thru, eastbound thru, and westbound thru lane groups respectively. EBL stands for eastbound left lane group. Some tests in the table had the number of delay observations (n) less than 20 because the data for these time periods was available for less than one hour.

There were a total of 84 tests performed over the 4 time periods out of which 80 were for the thru lane groups and the remaining 4 were for the protected left-turning lane group. An observed error in a comparison was significant only if the p-value of its *t*-test was less than 5 percent. The tests in which HCM significantly overestimated the stopped delay with respect to the field measurement are highlighted with red color in the table and the underestimations with blue. The results of the comparison are discussed in the next section.

Table 5.1 Statistical comparison between HCM stopped delay estimates and field measurements

			HCS Delay	Field Delay					
				n	Mean	Variance	df	*t*-statistic	p-value
Neil St & Stadium Dr	AM Peak	NBT	3.716	20	3.693	3.315	19	-0.0566	0.9555
		SBT	2.909	20	5.362	8.186	19	3.8338	0.0011
		EBT	17.960	19	10.925	15.999	18	-7.6669	< .00001
		WBT	14.277	19	10.500	95.765	18	-1.6823	0.1089
	Off Peak	NBT	1.890	20	2.339	4.542	19	0.9423	0.3580
		SBT	1.896	20	5.013	12.128	19	4.0023	0.0008
		EBT	15.614	20	20.250	328.494	19	1.1439	0.2672
		WBT	15.686	20	14.159	150.708	19	-0.5564	0.5844
	Noon Peak	NBT	2.532	20	2.009	1.758	19	-1.7627	0.0940
		SBT	2.599	20	2.817	1.846	19	0.7182	0.4815
		EBT	15.921	20	14.798	87.858	19	-0.5357	0.5984
		WBT	16.282	20	13.050	91.207	19	-1.5135	0.1466
	PM Peak	NBT	3.717	19	4.063	5.381	18	0.6509	0.5239
		SBT	4.354	19	6.313	6.971	18	3.2337	0.0046
		EBT	14.525	14	10.241	125.672	13	-1.4296	0.1764
		WBT	18.036	18	10.549	17.359	17	-7.6237	< .00001
Neil St & Kirby Ave	AM Peak	NBT	21.809	20	17.666	123.228	19	-1.6691	0.1115
		SBT	11.384	20	15.959	53.706	19	2.7918	0.0117
		EBT	40.690	20	18.682	80.037	19	-11.0016	< .00001
		WBT	27.938	20	35.226	185.857	19	2.3908	0.0274
	Off Peak	NBT	10.475	20	17.992	34.790	19	5.6990	0.000017
		SBT	5.966	20	16.704	35.814	19	8.0247	< .00001
		EBT	38.126	20	21.992	38.105	19	-11.6886	< .00001
		WBT	36.775	20	16.820	91.563	19	-9.3263	< .00001
	Noon Peak	NBT	20.646	20	23.131	46.721	19	1.6255	0.1206
		SBT	15.449	20	22.945	53.732	19	4.5728	0.0002
		EBT	37.515	20	20.537	41.863	19	-11.7353	< .00001
		WBT	31.033	20	17.262	88.695	19	-6.5390	< .00001
	PM Peak	NBT	26.103	20	27.449	82.068	19	0.6642	0.5147
		SBT	15.976	20	20.682	110.902	19	1.9983	0.0602
		EBT	41.302	20	21.756	98.346	19	-8.8142	< .00001
		WBT	45.970	20	33.028	122.086	19	-5.2384	0.000047

Table 5.1 (cont.)

			HCS Delay	Field Delay					
				n	Mean	Variance	df	*t*-statistic	p-value
Neil St & StMarys Rd	AM Peak	NBT	0.616	20	5.807	12.571	19	6.5487	< .00001
		SBT	8.401	20	8.746	11.963	19	0.4460	0.6606
		EBT	43.286	20	34.247	1228.195	19	-1.1535	0.2631
		WBT	38.468	20	22.154	230.998	19	-4.8005	0.0001
	Off Peak	NBT	0.912	20	2.691	2.671	19	4.8680	0.0001
		SBT	1.173	20	1.990	2.792	19	2.1862	0.0415
		EBT	39.780	20	29.641	573.941	19	-1.8926	0.0738
		WBT	41.930	20	29.813	746.569	19	-1.9834	0.0620
	Noon Peak	NBT	5.659	20	5.664	12.400	19	0.0063	0.9953
		SBT	1.771	20	2.771	4.100	19	2.2099	0.0397
		EBT	37.829	20	23.686	174.334	19	-4.7901	0.0001
		WBT	42.203	20	17.175	175.417	19	-8.4509	< .00001
	PM Peak	NBT	8.206	20	7.717	17.959	19	-0.5160	0.6118
		SBT	4.816	20	4.958	12.667	19	0.1786	0.8606
		EBT	38.674	20	32.516	773.410	19	-0.9903	0.3345
		WBT	47.159	20	26.196	133.494	19	-8.1142	< .00001
Neil St & Devonshire Dr	AM Peak	NBT	0.339	20	0.875	0.459	19	3.5430	0.0022
		SBT	0.600	20	1.254	1.035	19	2.8764	0.0097
		EBL	39.138	20	43.971	314.163	19	1.2194	0.2378
	Off Peak	NBT	0.099	20	2.873	8.776	19	4.1874	0.0005
		SBT	0.432	20	0.989	1.137	19	2.3375	0.0305
		EBL	38.524	20	37.000	324.628	19	-0.3783	0.7094
	Noon Peak	NBT	0.154	20	1.111	1.151	19	3.9909	0.0008
		SBT	0.697	20	0.796	0.470	19	0.6458	0.5266
		EBL	37.792	20	44.471	700.865	19	1.1282	0.2734
	PM Peak	NBT	0.098	20	1.067	1.883	19	3.1592	0.0052
		SBT	0.882	20	0.938	1.427	19	0.2105	0.8359
		EBL	41.443	20	43.557	1078.859	19	0.2878	0.7772

Table 5.1 (cont.)

			HCS Delay	Field Delay					
				n	Mean	Variance	df	*t*-statistic	P-value
Neil St & Knollwood Dr	AM Peak	NBT	0.551	13	0.331	0.155	12	-2.0212	0.0662
		SBT	0.210	15	1.033	8.475	14	1.0938	0.2928
	Off Peak	NBT	0.383	20	0.495	0.657	19	0.6201	0.5426
		SBT	0.335	20	0.095	0.048	19	-4.9262	0.0001
	Noon Peak	NBT	1.159	20	0.717	0.443	19	-2.9712	0.0078
		SBT	1.269	20	0.674	1.145	19	-2.4840	0.0225
	PM Peak	NBT	0.569	20	0.335	0.447	19	-1.5681	0.1334
		SBT	1.184	20	0.691	0.735	19	-2.5727	0.0186
Neil St & Windsor Rd	AM Peak	NBT	13.854	20	9.999	54.620	19	-2.3324	0.0308
		SBT	10.400	20	6.539	23.301	19	-3.5773	0.0020
		EBT	33.760	20	15.228	20.447	19	-18.3287	< .00001
		WBT	32.050	20	23.207	113.032	19	-3.7196	0.0015
	Off Peak	NBT	6.562	20	9.199	37.656	19	1.9214	0.0699
		SBT	6.437	20	6.310	14.173	19	-0.1505	0.8820
		EBT	34.551	20	23.778	144.227	19	-4.0118	0.0007
		WBT	33.215	20	20.419	126.743	19	-5.0833	0.0001
	Noon Peak	NBT	12.613	20	14.538	45.497	19	1.2768	0.2173
		SBT	7.663	20	8.194	23.824	19	0.4864	0.6325
		EBT	35.014	20	24.089	91.220	19	-5.1153	0.0001
		WBT	33.633	20	21.288	214.936	19	-3.7657	0.0013
	PM Peak	NBT	12.626	20	16.081	71.054	19	1.8335	0.0825
		SBT	11.911	16	12.317	45.556	15	0.2410	0.8128
		EBT	36.808	20	27.073	124.702	19	-3.8987	0.0010
		WBT	45.988	9	26.481	54.594	8	-7.9204	0.000047

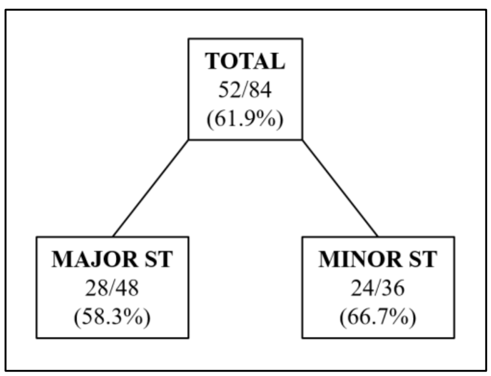
5.4 Results and Discussion

The following are the findings in the statistical comparison:

a) There were 52 out of 84 tests that had statistically significant errors in the estimation. In other words, the HCM estimates of stopped delay were not accurate 61.9% of the time.

b) In the 84 comparisons, 48 correspond to major street lane groups and 36 correspond to minor street lane groups. There were 28 out of 48 comparisons (58.3%) that had significant errors on major street, and 24 out of 36 comparisons (66.7%) were significant on minor street, as shown in Figure 5.2 below. Thus, the proportion of comparisons having significant discrepancies was in the same order for both major and minor street.

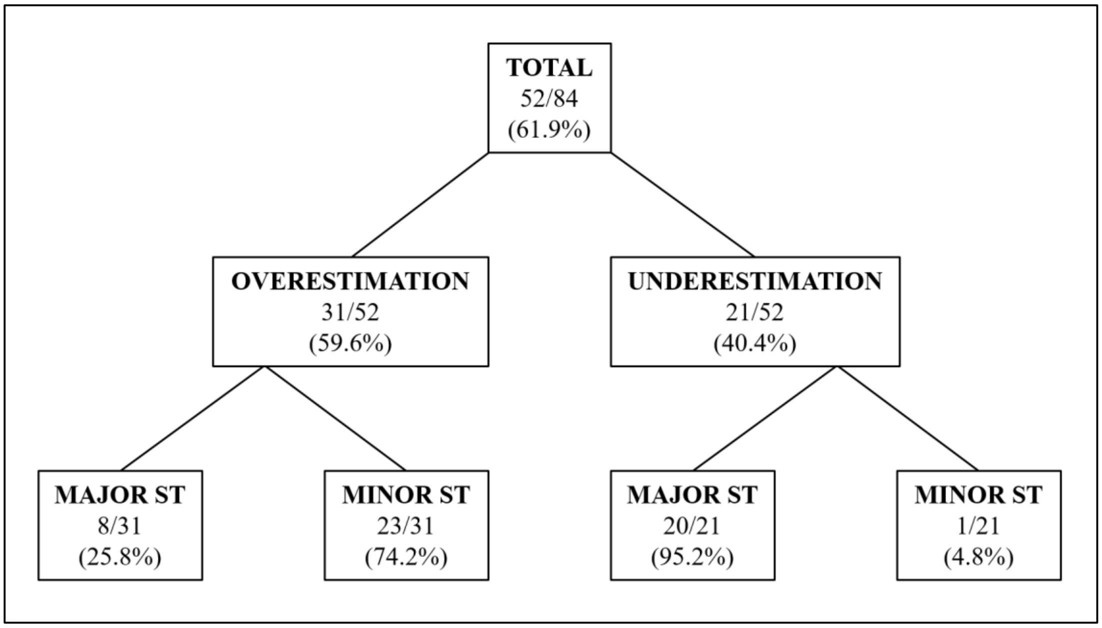
Figure 5.2 Proportion of significant discrepancies: major vs. minor street



c) Out of the 52 significant errors, HCM overestimated the stopped delay in 31 cases (59.6%) and underestimated in 21 cases (40.4%) with respect to field measurements, as shown in Figure 5.3 below. Thus, there is more significant overestimation than underestimation.

d) In the 31overestimations, 8 out of 31 (25.8%) were on major street and 23 out of 31 (74.2%) were on minor street. In the 21 underestimations, 20 out of 21 (95.2%) were on major street and 1 out of 21 (4.8%) was on minor street. This is illustrated in Figure 5.3 below.

Figure 5.3 Division of all comparisons having significant discrepancies



Although the proportion of comparisons having significant discrepancies is similar on both major and minor streets from Figure 5.2, there is a particular trend observed from Figure 5.3. It is evident that most of the significant overestimation is on the minor street (74.2%) and almost all of the significant underestimation is on the major street (95.2%). Thus, overall, HCM seems to significantly overestimate delay on minor streets and underestimate on major streets of actuated-signalized intersections in comparison to field measurements.

Now, it is sought to determine if this overall trend is valid at all intersections and during all time periods. The following sections will analyze the results of the statistical comparison based on intersection and time of day.

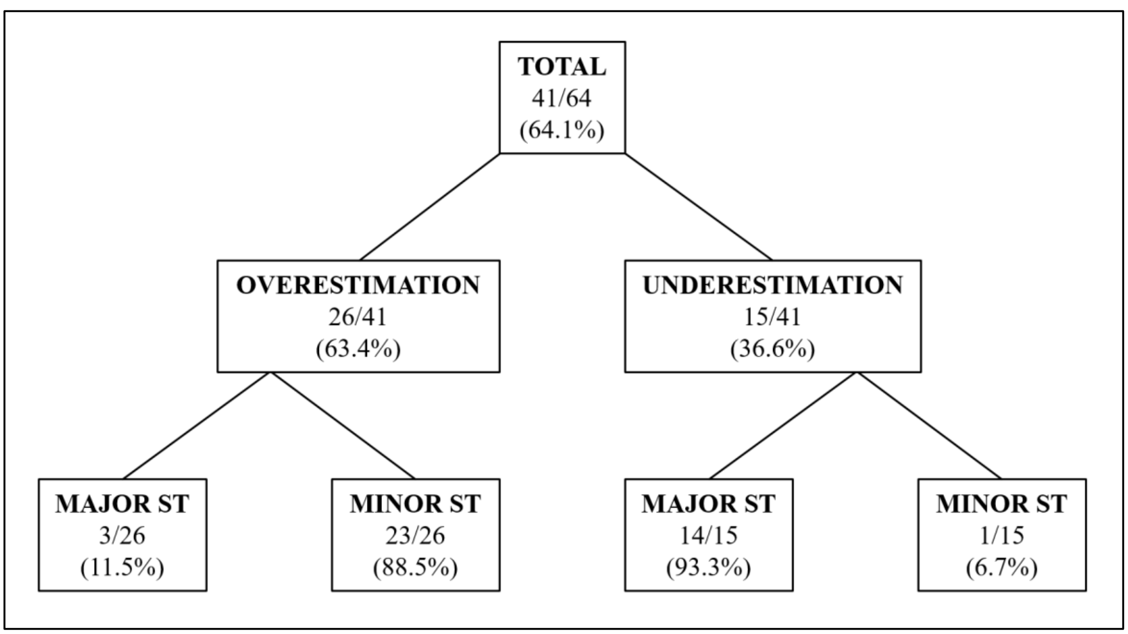
5.5 Analysis by Intersection

The intersections were divided into two groups based on whether or not all cross-street thru lane groups were present. The four-legged intersections were placed in group 1 and the intersections of Neil St. with Devonshire Dr. and Knollwood Dr. were placed in group 2. The statistical comparisons of the two groups are separately analyzed in the following subsections.

5.5.1 Group 1

In group 1, 41 out of 64 comparisons had statistically significant discrepancies (64.1%) over all time periods together. Out of the 41 comparisons with significant discrepancies, HCM overestimated the stopped delay in 26 cases (63.4%) and underestimated in 15 cases (36.6%). Clearly, there is more overestimation present than underestimation. In the 26 overestimations, 3 were on major street (11.5%) and 23 were on minor street (88.5%). In the 15 underestimations, 14 were on major street (93.3%) and 1 was on minor street (6.7%). This is illustrated in Figure 5.4.

Figure 5.4 Division of group 1 comparisons having significant discrepancies



From the above figure for group 1, most of the overestimations are on the minor street (88.5%) and almost all underestimations are on the major street (93.3%). Thus, clearly, the trend observed in the previous section holds well for group 1 as well. Now, individual intersections of group 1 are examined for this trend.

Table 5.2 below presents the information of comparisons with significant discrepancies for each intersection with respect to over- and underestimation, as well as major and minor street. At each of the 4 intersections, again, the overestimations are mostly on minor street and the underestimations on major street. The proportion of comparisons on major vs. minor street of each intersection in Table 5.2 is, although, not exactly equal to that in the overall comparisons, they are very similar to each other.

Table 5.2 Comparisons with significant discrepancies per intersection of group 1

<table>
<tr><td colspan="4">NEIL STREET & STADIUM DRIVE</td><td colspan="4">NEIL STREET & KIRBY DRIVE</td></tr>
<tr><td colspan="4">Total</td><td colspan="4">Total</td></tr>
<tr><td colspan="4">6/16
(37.5%)</td><td colspan="4">13/16
(81.25%)</td></tr>
<tr><td colspan="2">Overestimation</td><td colspan="2">Underestimation</td><td colspan="2">Overestimation</td><td colspan="2">Underestimation</td></tr>
<tr><td colspan="2">3/6
(50%)</td><td colspan="2">3/6
(50%)</td><td colspan="2">7/13
(53.8%)</td><td colspan="2">6/13
(46.2%)</td></tr>
<tr><td>Major St.</td><td>Minor St.</td><td>Major St.</td><td>Minor St.</td><td>Major St.</td><td>Minor St.</td><td>Major St.</td><td>Minor St.</td></tr>
<tr><td>1/3
(33.3%)</td><td>2/3
(66.6%)</td><td>3/3
(100%)</td><td>0/3
(0%)</td><td>0/7
(0%)</td><td>7/7
(100%)</td><td>5/6
(83.3%)</td><td>1/6
(16.7%)</td></tr>
<tr><td colspan="8"></td></tr>
<tr><td colspan="4">NEIL STREET & ST. MARY'S ROAD</td><td colspan="4">NEIL STREET & WINDSOR ROAD</td></tr>
<tr><td colspan="4">Total</td><td colspan="4">Total</td></tr>
<tr><td colspan="4">10/16
(62.5%)</td><td colspan="4">12/16
(75%)</td></tr>
<tr><td colspan="2">Overestimation</td><td colspan="2">Underestimation</td><td colspan="2">Overestimation</td><td colspan="2">Underestimation</td></tr>
<tr><td colspan="2">6/10
(60%)</td><td colspan="2">4/10
(40%)</td><td colspan="2">10/12
(83.3%)</td><td colspan="2">2/12
(16.7%)</td></tr>
<tr><td>Major St.</td><td>Minor St.</td><td>Major St.</td><td>Minor St.</td><td>Major St.</td><td>Minor St.</td><td>Major St.</td><td>Minor St.</td></tr>
<tr><td>0/6
(0%)</td><td>6/6
(100%)</td><td>4/4
(100%)</td><td>0/4
(0%)</td><td>2/10
(20%)</td><td>8/10
(80%)</td><td>2/2
(100%)</td><td>0/2
(0%)</td></tr>
</table>

Thus, it can be concluded that the trend of underestimation on major street and overestimation on minor street of actuated signalized intersections by HCM exists at the intersection level for group 1. In general, there is more significant overestimation present than underestimation.

From Table 5.2, it is also observed that the number of comparisons with significant discrepancies at the intersection of Neil St. and Stadium Dr. is much lower than the other three intersections. The signal control at this intersection is a simple two-phase operation, whereas the other intersections have more signal phases. The HCM methodology for delay estimation of actuated signalized intersections, which involves the iterative procedure for estimating average actuated-phase durations, might work better for simple phase plans, and thus could be the reason for the lower number of significant discrepancies.

5.5.2 Group 2

The intersections of group 2 did not have cross-street thru lane groups. There were 12 comparisons made for the intersection at Neil St. and Devonshire Dr., which included the major street thru lane groups and a protected left-turning lane group on the minor street. There were 8 comparisons made for the intersection at Neil St. and Knollwood Dr., all of which were major-street thru lane groups.

Table 5.3 below presents the information of comparisons with significant discrepancies for each intersection with respect to over- and underestimation, as well as major and minor street. It is observed that, in total, both the intersections have similar proportion of comparisons with significant discrepancies. However, at Neil St. and Devonshire Dr., all of them are underestimations on major street and, at Neil St. and Knollwood Dr., all of them are overestimations on major street. It is also noted that none of the protected left-turning lane group comparisons at Neil St. and Devonshire Dr. are significant.

Table 5.3 Comparisons with significant discrepancies per intersection of group 2

NEIL STREET & DEVONSHIRE DRIVE				NEIL STREET & KNOLLWOOD DRIVE			
Total				**Total**			
6/12 (50%)				5/8 (62.5%)			
Overestimation		**Underestimation**		**Overestimation**		**Underestimation**	
0/6 (0%)		6/6 (100%)		5/5 (100%)		0/5 (0%)	
Major St.	**Minor St.**	**Major St.**	**Minor St.**	**Major St.**	**Minor St.**	**Major St.**	**Minor St.**
0/0 (0%)	0/0 (0%)	6/6 (100%)	0/6 (0%)	5/5 (100%)	n/a	0/5 (0%)	n/a

Thus, the HCM delay estimates at Neil St. and Devonshire Dr. follow the trend of underestimation on major street, whereas at Neil St. and Knollwood Dr., they are overestimated and thus do not fit in the trend. Also, as there are no cross-street thru lane groups present in this group, nothing can be inferred about minor streets at these intersections.

It should be noted that, in group 2, average stopped delay of the thru lane groups itself is in the order of 1 second, if not lower, due to very low demand on the cross streets. Thus, the trend observed in discrepancies by HCM at regular four-legged intersection might not be evident at such atypical intersections.

The next section will analyze the results of the statistical comparison for each time period.

5.6 Analysis by Time of Day

The results of the statistical comparison were grouped for each of the 4 time periods, i.e. AM peak, off peak, noon peak and PM peak, over all intersections together. Table 5.4 below presents the information of comparisons with significant discrepancies for each time period with respect to over- and underestimation, as well as major and minor street.

Table 5.4 Comparisons with significant discrepancies for each time period

AM PEAK				OFF PEAK			
Total				**Total**			
14/21 (66.7%)				15/21 (71.4%)			
Overestimation		**Underestimation**		**Overestimation**		**Underestimation**	
8/14 (57.1%)		6/14 (42.9%)		7/15 (46.7%)		8/15 (53.3%)	
Major St.	**Minor St.**	**Major St.**	**Minor St.**	**Major St.**	**Minor St.**	**Major St.**	**Minor St.**
3/8 (37.5%)	5/8 (62.5%)	5/6 (83.3%)	1/6 (16.7%)	1/7 (14.3%)	6/7 (85.7%)	8/8 (100%)	0/8 (0%)
NOON PEAK				PM PEAK			
Total				**Total**			
12/21 (57.1%)				11/21 (52.38%)			
Overestimation		**Underestimation**		**Overestimation**		**Underestimation**	
9/12 (75%)		3/12 (25%)		7/11 (63.6%)		4/11 (36.4%)	
Major St.	**Minor St.**	**Major St.**	**Minor St.**	**Major St.**	**Minor St.**	**Major St.**	**Minor St.**
3/9 (33.3%)	6/9 (66.7%)	3/3 (100%)	0/3 (0%)	1/7 (14.3%)	6/7 (85.7%)	4/4 (100%)	0/4 (0%)

In the above table, the proportion of comparisons with significant discrepancies is similar for each time period. Within each time period, it can be seen that there is more overestimation present than underestimation, except for off peak wherein it is almost equal. Moreover, most of the overestimation in each time period is on the minor street and almost all underestimation is on the major street. Thus, clearly, the trend observed in the overall comparisons holds well for each time period.

Hence, it can be concluded that HCM underestimates stopped delay on major street and overestimates on minor street of actuated signalized intersection for all the four time periods in comparison to field measurements. The potential causes of the observed trend in discrepancies are discussed in the next section.

5.7 Potential Causes

The errors in delay estimation by HCM 2010 can be attributed to its green allocation for the actuated-signalized operation. The general trend of underestimating delay in major direction (Neil Street) and overestimating it in the minor direction (cross streets) clearly suggests that the green allocation is biased because, obviously, the delay estimates directly depend on the g/C ratio (effective green time to cycle length ratio). Thus the accuracy of delay estimation can certainly be improved by reallocating the green intervals appropriately. The reallocation can be done by reducing the green time from the underestimated direction and adding it to the overestimated direction.

The thru movement volume on Neil Street is always higher than that of the cross streets, and in fact much higher in most cases. Observing the under- and overestimation trend consistently at all intersections during all time periods indicates its correlation with the volume trend. HCM seems to overemphasize the high volume direction (or major street) as compared to the low demand direction (or minor street). This is believed to possibly cause the bias in green proportions by the HCS. Thus, this highlights the role of the iterative procedure for estimating the average duration of actuated phases (Chapter 31 of HCM 2010) in the accuracy of delay estimation. The scope of this study is limited only to the comparison of the HCM estimations and field measurements. A future direction is to study the role of the iterative green allocation procedure and its sensitivity to changes in traffic characteristics.

An inherent source of error in the capacity analysis of actuated signalized intersections is the delay calculation procedure itself. It is known that actuated signals are responsive to traffic demand. They have the ability to terminate green intervals early, skip phases, and so forth. The green intervals of such signals are varying in field. Thus, representing the actuated signal control as a fixed-cycle operation with 'average' phase durations leads to some error in the delay estimation.

The next chapter summarizes the thesis and provides recommendations for future studies.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

In summary, the accuracy of HCM 2010 delay estimation methodology for actuated-signalized intersections was studied in this thesis. The stopped delay estimates of thru and protect left-turning lane groups were statistically compared with field measurements. The field data was collected at six actuated-signalized intersections in Champaign, Illinois. The field stopped delay was measured using the field measurement technique of intersection control delay as described in Chapter 31 of HCM 2010. The HCM delay estimates were obtained using the existing field conditions via the HCS 2010 (version 6.70). The statistical analysis was performed using *t*-tests to examine the significance of the stopped delay discrepancies.

It was found that the HCM stopped delays were significantly different from the field data for 61.9% of the comparisons (52 cases out of 84) at 0.10 level. In the significant discrepancies noted, the HCM overestimated 59.6% of the comparisons (31 cases out of 52) and underestimated 40.4% of the comparisons (21 cases out of 52). Most of the significant overestimations were on the cross streets (74.2%) and almost all of the significant underestimations were on the major street (95.2%). The proportion of comparisons with significant discrepancies on Neil Street (58.3%) and the cross streets (66.7%) was, however, similar.

Thus, overall, HCM significantly overestimated the stopped delay on minor streets and underestimated on major street of actuated-signalized intersections in comparison to field measurements. Also, there was more significant overestimation present than underestimation. The observed trend of discrepancies was also verified for each intersection as well as each time period in the study. The trend was consistent at regular four-legged intersection, but might not be evident at atypical intersections that have very low demand on minor street. It was also observed that the proportion of significant discrepancies was lower (almost half) for an intersection with a simple two-phase signal control as compared to others that had more signal phases. This suggests that HCM 2010 methodology of capacity estimation, which involves the iterative procedure of estimating average actuated-phase durations, might work better for simple two-phase signalized intersections. Further work is recommended to corroborate this idea.

In conclusion, the HCM 2010 procedure of delay estimation for actuated-signalized intersections requires improvement for better accuracy. In light of the under- and overestimation trend found in this study, green allocation by the HCS is believed to be a potential cause of discrepancy in the delay estimation. This highlights the role of the iterative procedure for average phase duration described in Chapter 31 of HCM 2010 towards the delay discrepancies. The methodology seems to overemphasize the high volume direction (or major street) as compared to the low demand direction (or minor street). Representing the actuated signal timing as a fixed-cycle with ‘average’ phase durations in delay calculation is also believed to induce inherent error in the estimation.

It is thus recommend to study the role of HCM 2010 iterative procedure of estimating the average actuated-phase duration with regards to delay estimation accuracy. Another future direction is to study the HCM delay estimation accuracy for permitted and protected-permitted left-turning lane groups, and permitted right turning lane groups with high right turn on red (RTOR) volume.

A validation study of the HCM correction factors to adjust stopped delay for deceleration and acceleration delay, in order to obtain control delay, is also recommended using field data.

REFERENCES

Anusha, S. P., Sharma, A., Vanajakshi, L., Subramanian, S. C., & Rilett, L. R. (2016). Model-Based Approach for Queue and Delay Estimation at Signalized Intersections with Erroneous Automated Data, *ASCE Journal of Transportation Engineering*, 1-10. http://doi.org/10.1061/(ASCE)TE.1943-5436.0000835

Benekohal, R. F., Elzohairy, Y. M., & Saak, J. E. (2002). Comparison of Delays from Highway Capacity Software, Synchro, PASSER II and IV, and CORSIM for Urban Arterials. *Transportation Research Record*, *1802*(1), 133–144. http://doi.org/10.3141/1802-16

Braun, S. M., & Ivan, J. N. (1996). Estimating Intersection Approach Delay Using 1985 and 1994 Highway Capacity Manual Procedures, *Transportation Research Record*, 1555, 23-32.

Dion, F. (2004). Comparison of delay estimates at under-saturated and over-saturated pre-timed signalized intersections, *Transportation Research Part B*, *38*, 99-122. http://doi.org/10.1016/S0191-2615(03)00003-1

McTrans Center, University of Florida. (2010). *Highway Capacity Software*. Gainesville, Florida.

Petraglia, K. J. (1999). Field Validation of MOE’s for Signalized Intersection Analysis. In *69th Annual Meeting of the Institute of Transportation Engineers*. Las Vegas, Nevada.

Prevedouros, P. D., Koga, C. A. (1996). Comparison of 1985 and 1994 HCM: Signalized Intersection Delay Estimates, *ITE Journal*, 26-30. (July).

Saito, M., Walker, J., & Zundel, A. (2001). Use of Image Analysis to Estimate Average Stopped Delays per Vehicle at Signalized Intersections, *Transportation Research Record*, 1776, 106-113.

T. R. B. (2010). *Highway Capacity Manual* (Fifth). Washington, D.C.

Wang, Z., Chen, S., Yang, H., Wu, B., & Wang, Y. (2015). Comparison of delay estimation models for signalised intersections using field observations in Shanghai, *IET Intelligent Transport Systems*, 1-10. http://doi.org/10.1049/iet-its.2014.0280