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EVALUATION OF ADAPTIVE SIGNAL CONTROL TECHNOLOGY—VOLUME 1: BEFORE-CONDITIONS DATA COLLECTION AND ANALYSIS

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Safety and Efficiency Benefits of Implementing Adaptive Signal Control Technology in Illinois

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16. Abstract

Field evaluation of adaptive signal control technologies (ASCT) is very important in understanding the system's contribution to safety and operational efficiency. Data were collected at six intersections along the Neil Street corridor in Champaign, Illinois, before deployment of SynchroGreen, an ASCT system. The field delay and queue length for "before" conditions were measured and compared with estimates from the Highway Capacity Manual (HCM). The HCM estimates of stopped delay were significantly different in 58.3% of the cases, representing overestimation in 73.5% and underestimation in 26.5% of the cases. On major streets of typical intersections, in 72% of the cases, there were significant differences between HCM delay and field data; in 91% of the cases, HCM overestimated delay by an average by 69%. On the minor streets of typical intersections, in 56% of the cases there were significant differences, and in 94%, the HCM overestimated the delay on average by 52%. The HCM estimates of 50th-percentile queue length were significantly different in 61% of all cases, including overestimation in 56% and underestimation in 44% of the cases. For typical intersections, 52% of the cases had significant differences, including overestimation in 93% and underestimation in 7% of the cases. On the major streets of typical intersections, 68% of the cases had 50th percentile HCM queue lengths similar to those from the field. However, in 28% of the cases it overestimated the queue length on average by 66%; and in 4% of the cases it underestimated it on average by 42%. For minor streets of typical intersections, in only 25% of the cases were the HCM queue lengths similar to those from the field; in 70% of the cases, HCM overestimated the queue length on average by 44%, and in 5% of the cases, HCM underestimated it on average by 20%. In general, it was observed that the trends in the 50th- and 95th-percentile queue length comparisons supported each other. Also, in 91% of the cases, the trends in delay and queue comparisons were either consistent with each other or did not have any significant conflicts. However, in 9% of the cases, significant inconsistencies in trends were observed. Thus, to save time, one may compare the HCM queue length estimates to field data to assess intersection performance, though the delay comparison is preferred.

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EXECUTIVE SUMMARY

Traffic signals in the United States have evolved from fixed-cycle to vehicle-actuated operation to the present-day advanced signal systems and adaptive signal control technology (ASCT). An adaptive traffic signal adjusts its phase plan and signal timing in response to real-time traffic demand. Field evaluation of ASCT is very important in understanding the system's contribution to traffic safety and performance improvement—and, hence, its effectiveness.

The Illinois Department of Transportation (IDOT) is interested in in field evaluation of an ASCT on a corridor. Through a competitive bidding process, a Trafficware product called SynchroGreen® was selected for field implementation. Six intersections along Neil Street in Champaign, Illinois, were selected for this implementation. To evaluate the SynchroGreen system, the corridor's performance prior to ASCT deployment was measured. The data are used as a basis to compare the performance of the system after it is deployed.

This report presents the methodology and outcome of data collection, data reduction, and data analysis of the field conditions before implementation of SynchroGreen in Champaign. Traffic characteristics for four different time periods (AM peak, off peak, noon peak, and PM peak) were obtained from field videotapes. Those traffic characteristics include peak hours, hourly volume, saturation flow rate, signal timing, arrival type, field delay, and queue length. The field delay and queue length measured before implementation are used to evaluate the operational performance of the SynchroGreen system by comparing those characteristics after implementation. Those measures of effectiveness in the "before conditions" were also compared with estimations from the Highway Capacity Manual (HCM) to quantify the effects of volume changes and additional developments at Neil Street and Devonshire Drive through the course of the study.

The HCM estimates of stopped delays were significantly different in 58.3% of the cases, representing overestimation in 73.5% and underestimation in 26.5% of the cases. On major streets of typical intersections, HCM delay estimates and field data were significantly different in 72% of the cases; in 91% of these cases, HCM overestimated the delay by an average by 69%. On minor streets of typical intersections, in 56% of the cases there were significant differences between HCM and field data; in 94% of these cases, HCM overestimated the delay on average by 52%.

HCM estimates of 50th percentile queue length were significantly different in 61% of all cases, including overestimations in 56% and underestimations in 44% of the cases. For typical intersections, 52% of the cases had significant differences, including overestimations in 93% and underestimations in 7% of the cases. On the major streets of the typical intersections, in 68% of the cases, the HCM queue lengths were similar to those from the field. However, in 28% of the cases, HCM overestimated the queue length on average by 66%; in 4% of the cases, it underestimated the queue length on average by 42%. On the minor streets of typical intersections, in only 25% of the cases were the median HCM queue lengths similar to those from the field; however, in 70% of the cases, HCM overestimated the queue length on average by 44%, and in 5% of the cases, it underestimated it on average by 20%.

In addition, a 95th percentile queue length comparison was conducted between HCM estimates and field data. In general, it was observed that trends in the 50th and 95th percentile queue length comparisons supported each other.

The consistency between the results of stopped delay and the 50th percentile queue length comparisons for the 64 overlapping cases was analyzed. In 91% of the cases, the trend in delay and queue comparisons were either consistent with each other or did not have any significant conflicts. However, in 9% of the cases, significant inconsistencies in trends were observed. Thus, to save time one may compare HCM queue length estimates to field data to assess intersection performance, though the delay comparison is preferred.

CONTENTS

CHAPTER 1: INTRODUCTION

Intersection traffic signal control has evolved from pre-timed (fixed-time) operation to vehicle-actuated to the present-day adaptive signal systems. Adaptive signal control technologies (ASCT) are used to make traffic signal operation more responsive to real-time traffic demand. Thus, the technologies have the potential to provide a more efficient and safer operation. In the United States, adaptive systems are relatively new and are increasingly being deployed in different parts of the country.

With existing signal systems, we still face the issue of congestion. In 2014, as a result of congestion, it is estimated that urban Americans traveled 6.9 billion hours more and purchased an extra 3.1 billion gallons of fuel—resulting in total congestion costs of about $160 billion (Schrank et al. 2015). Thus, increased deployment of more-efficient signal systems is necessary to reduce those massive effects of congestion.

The Illinois Department of Transportation (IDOT) has expressed interest in field evaluation of an ASCT for deployment at intersections in the state. Through a competitive process, SynchroGreen® was selected from available ASCTs for field evaluation. It is a real-time ASCT system from Trafficware Inc. (Trafficware 2012). Field evaluations of ASCTs are very important in understanding their contribution to performance improvement—and, hence, their effectiveness. Some field evaluations of SynchroGreen have been reported in the recent past (Stevanovic 2010), at locations such as Seminole County, Florida (Cheek et al. 2011) and Boca Raton, Florida (So et al. 2014).

Therefore a “before and after” study was undertaken on behalf of IDOT to evaluate performance of the SynchroGreen system in terms of traffic safety and traffic operational efficiency.

This report presents traffic data recorded before the deployment of the SynchroGreen system and the analysis performed to evaluate the “before” conditions. The installation of the system began in May 2015 along the Neil Street corridor in Champaign, Illinois, as shown in Figure 1. The six intersections along Neil Street, from north to south, are as follows:

- Neil Street and Stadium Drive
- Neil Street and Kirby Avenue
- Neil Street and St. Mary’s Road
- Neil Street and Devonshire Drive
- Neil Street and Knollwood Drive
- Neil Street and Windsor Road

In addition, the traffic signal at Kirby Avenue and State Street was linked to the traffic signal at Kirby and Neil so that they work in a coordinated manner.

Figure 1. Top view of the deployment location on Neil Street in Champaign, Illinois.

This report is organized as follows:

- Chapter 2 contains a description of the study area and data collection methodology used in the study.
- Chapter 3 presents the methodology and outcomes of data reduction performed following the collection of traffic data.
- Chapter 4 discusses the capacity analyses carried out to obtain HCM (2010) delay and queue estimates and the statistical comparison of HCM estimates with field measurements, as well as the relationships between the results of those comparisons.
- Chapter 5 presents the main findings and conclusions.

CHAPTER 2: DATA COLLECTION

This chapter describes the study area and presents the methodology used for data collection.

2.1 DESCRIPTION OF STUDY AREA

The study area consists of the six intersections along the Neil Street corridor as shown in Figure 2. All decisions regarding system performance are based on those six intersections only (and not the one at Kirby Avenue and State Street).

In the "before system deployment" condition, the six intersections on Neil Street were operating as a coordinated system and provided signal progression for northbound and southbound traffic. The operation was kept as optimal as practically possible. The seventh intersection (i.e., the intersection at Kirby Avenue and State Street) simply had a fixed offset with respect to the intersection at Neil Street and Kirby Avenue. The traffic pattern on Neil Street is such that it has higher volume going northbound (toward downtown Champaign) in the morning and southbound in the afternoon. Neil Street also forms the western boundary of the University of Illinois at Urbana-Champaign.

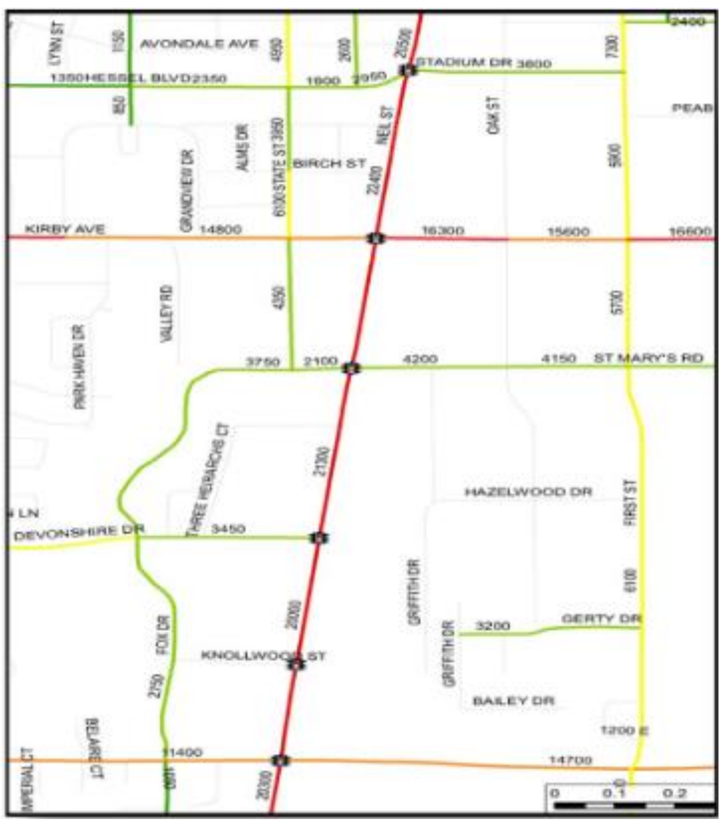


Figure 2. The six study intersections on Neil Street in Champaign, Illinois.

Schematic geometries of the six intersections are shown in Figures 3 through 8 (the drawings are not to scale).

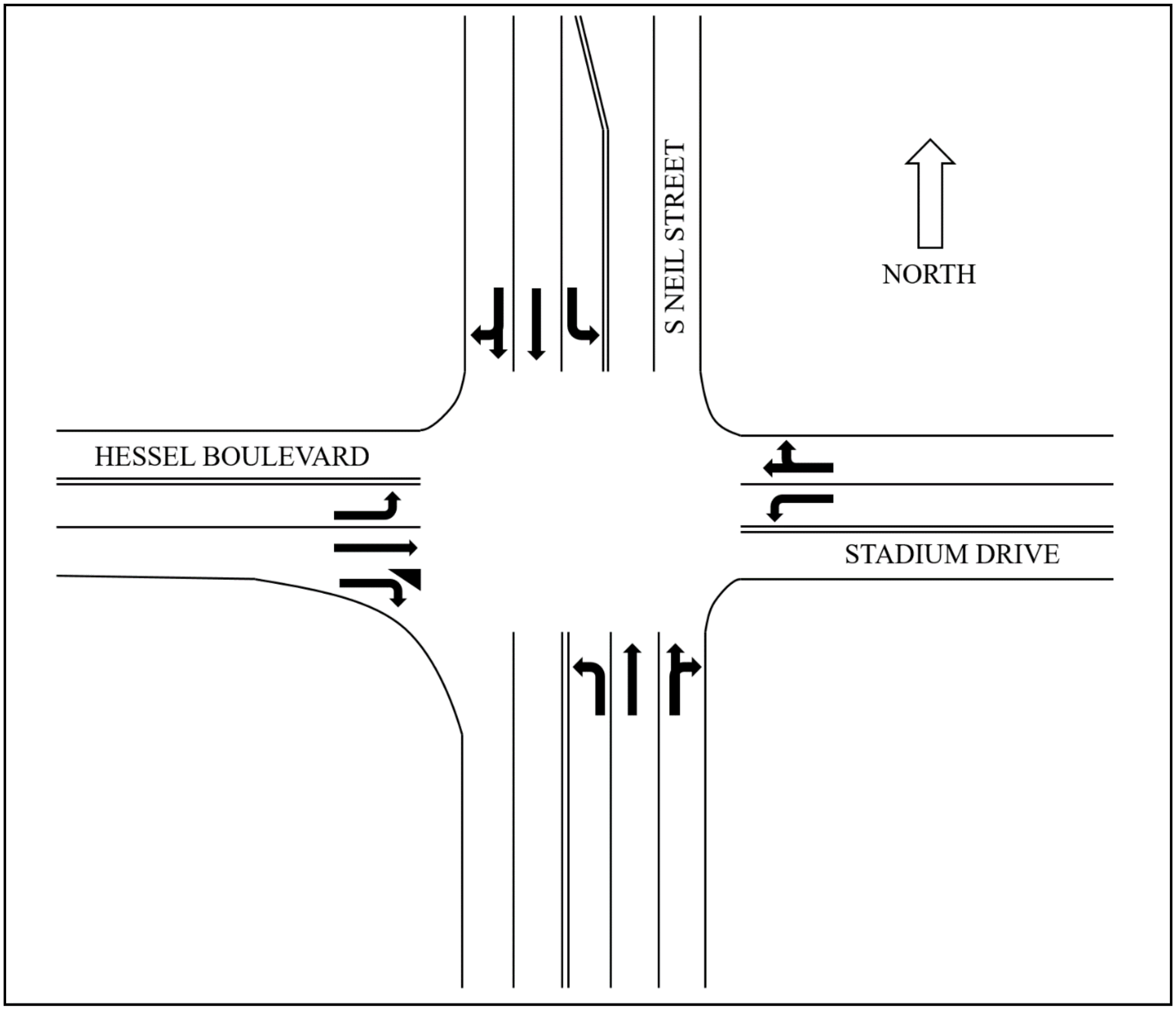


Figure 3. Geometry of the intersection of Neil Street and Stadium Drive.

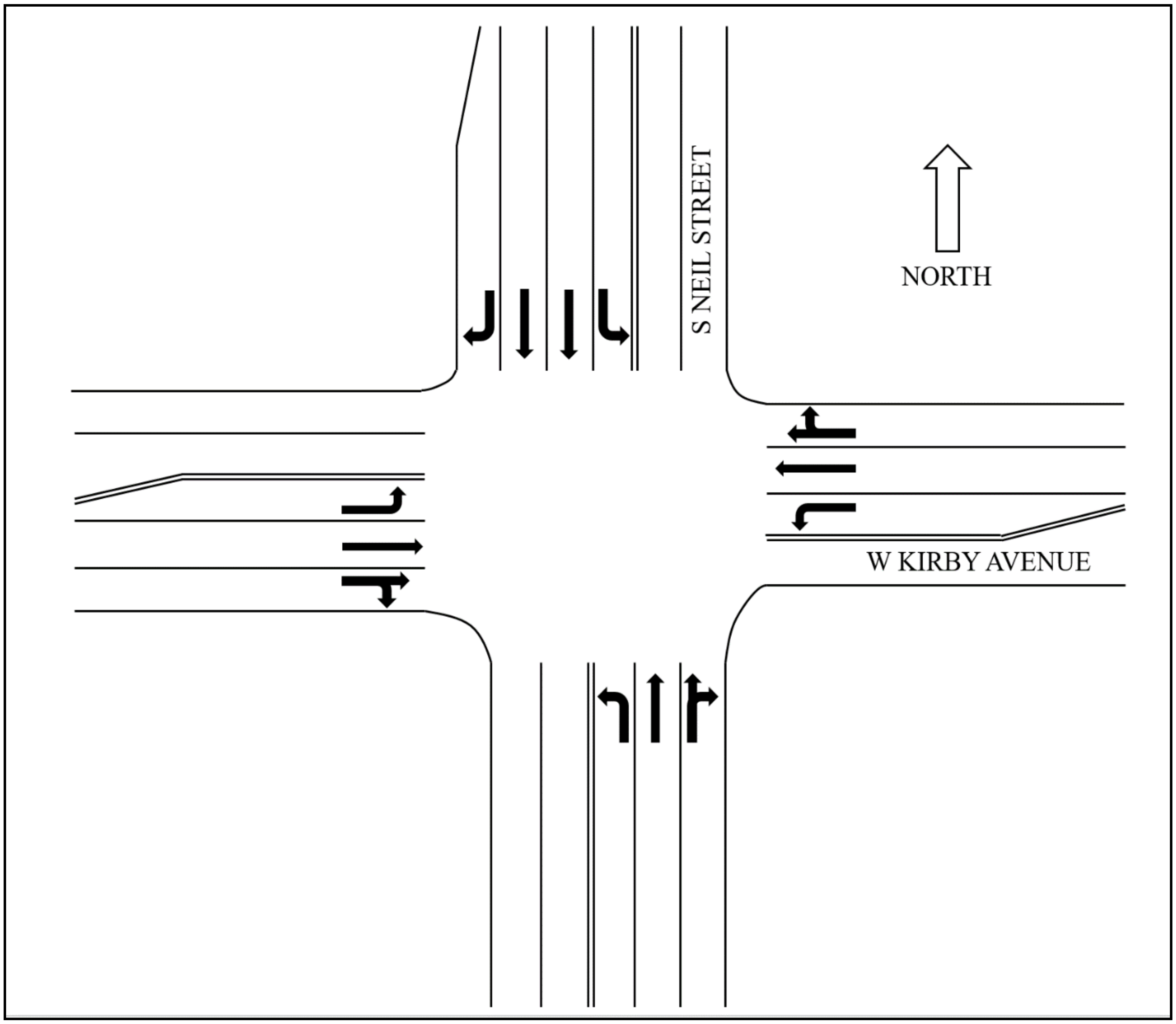


Figure 4. Geometry of the intersection of Neil Street and Kirby Avenue.

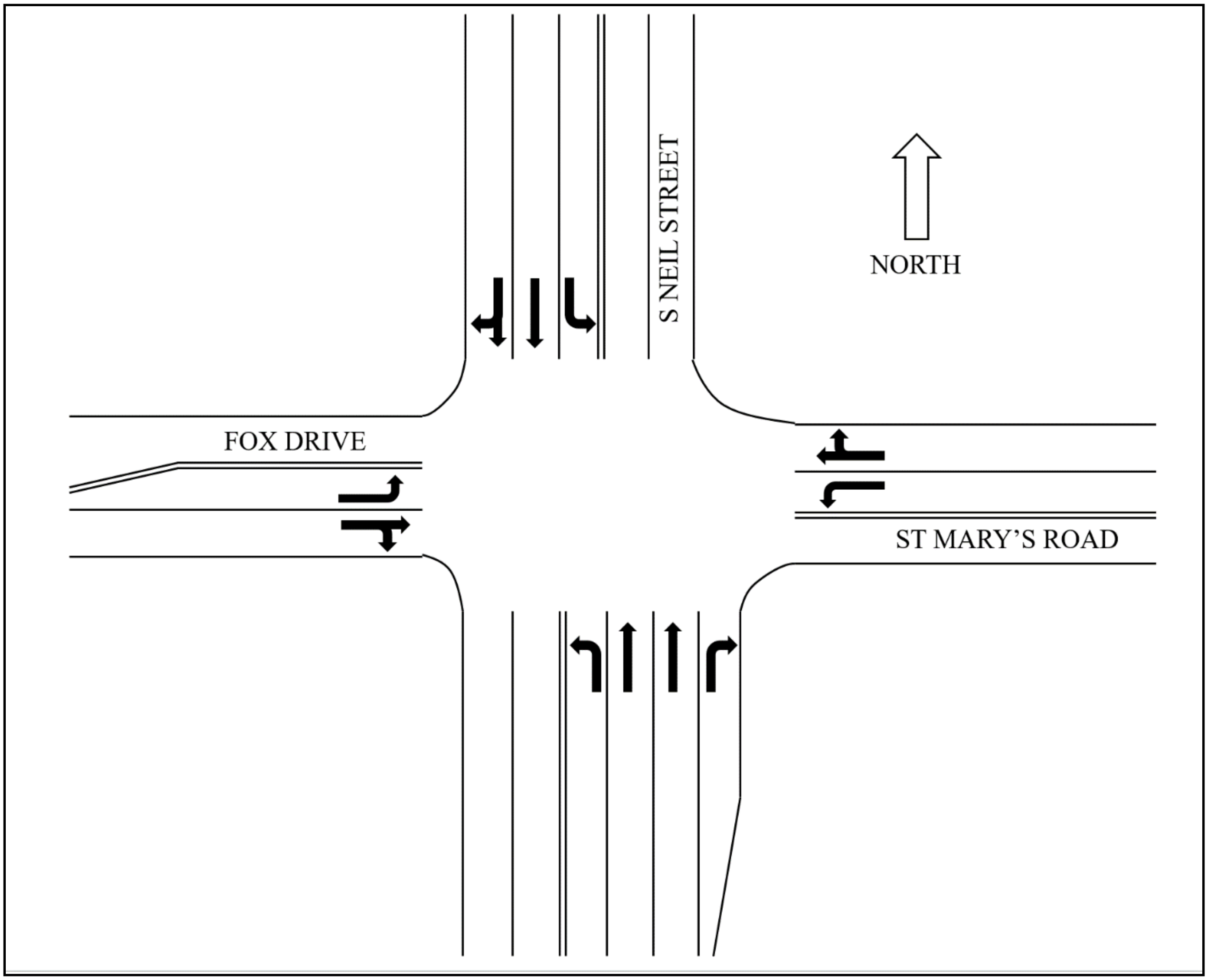


Figure 5. Geometry of the intersection of Neil Street and St. Mary's Road.

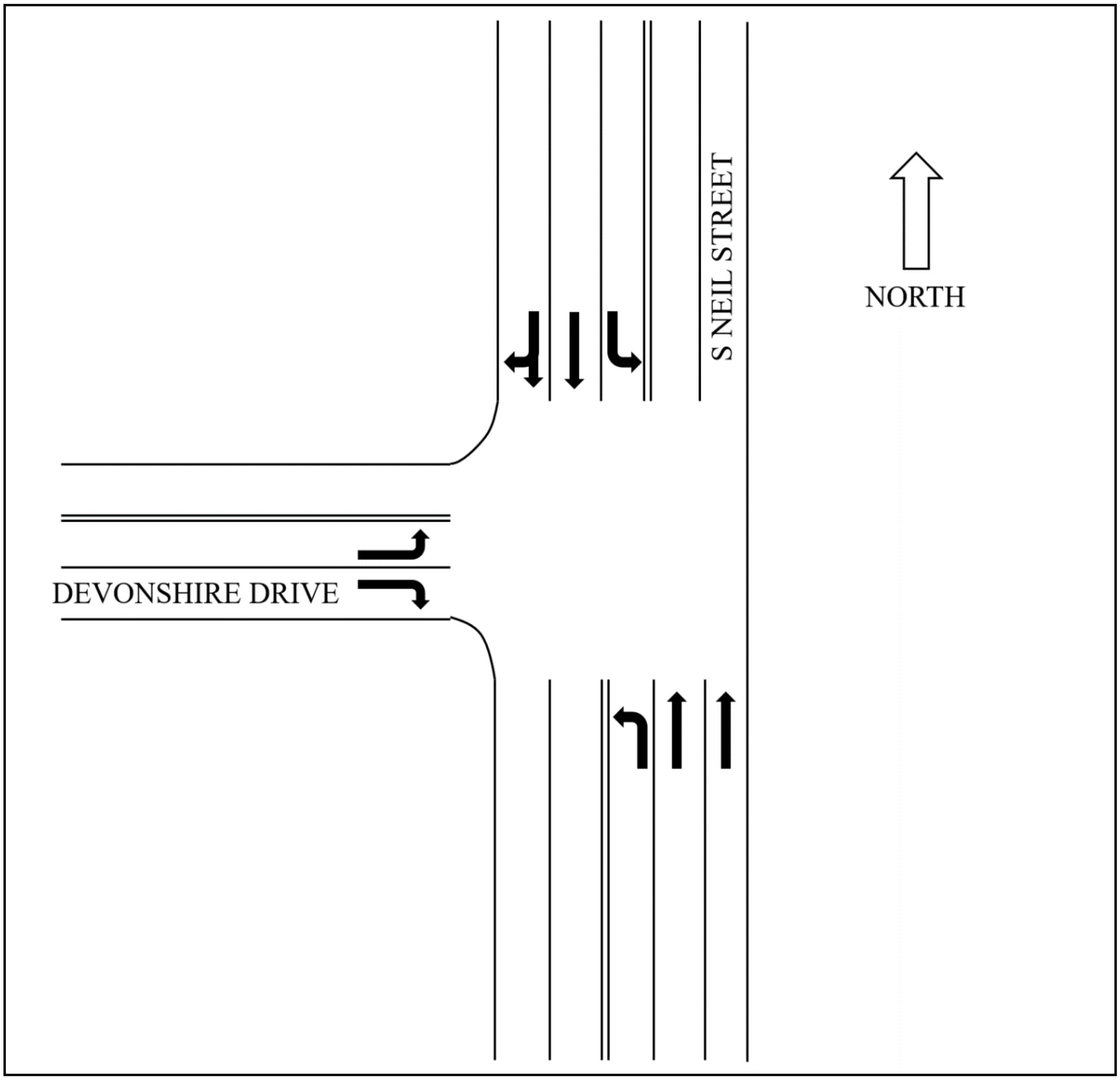


Figure 6. Geometry of the intersection of Neil Street and Devonshire Drive.

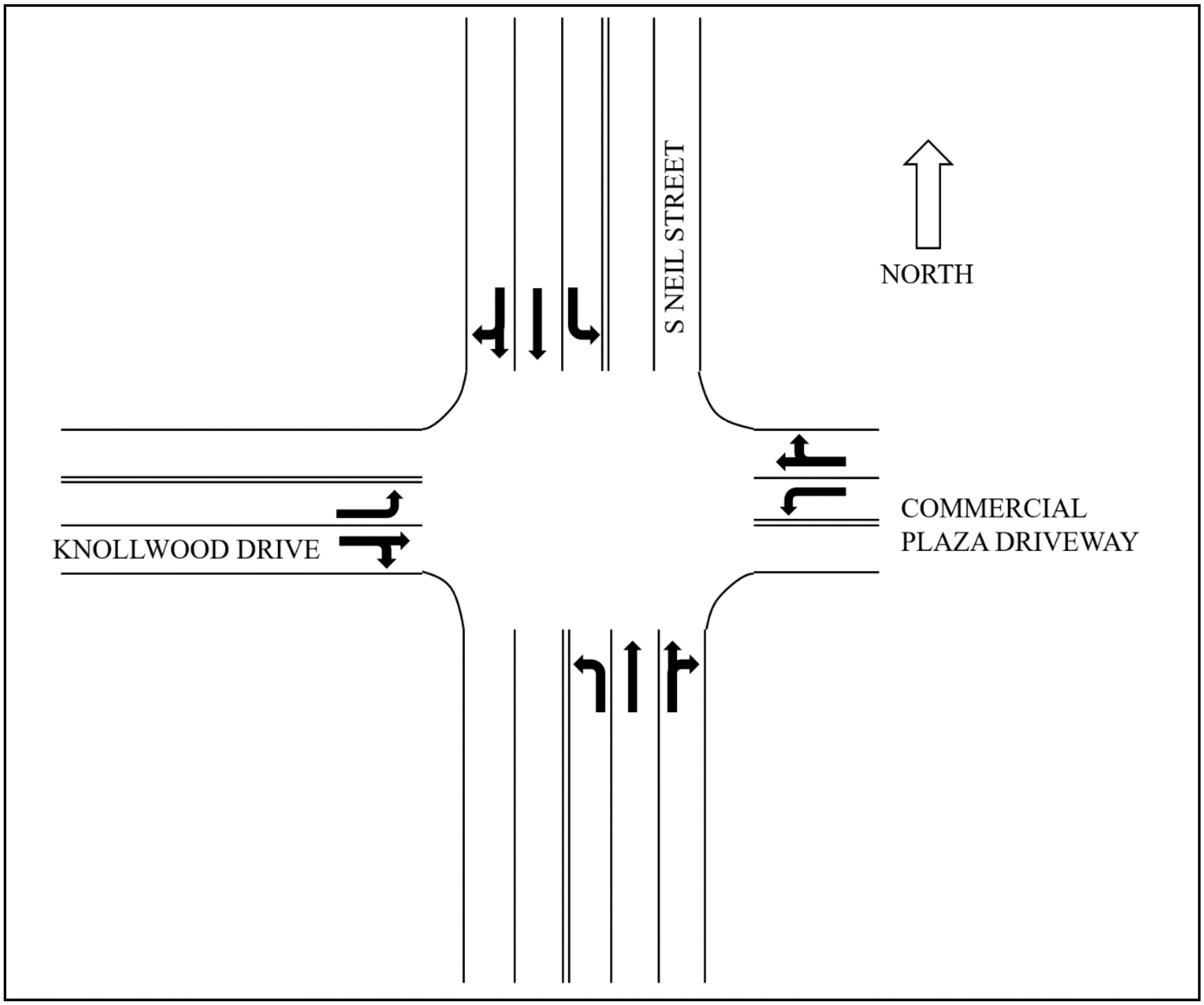


Figure 7. Geometry of the intersection of Neil Street and Knollwood Drive.

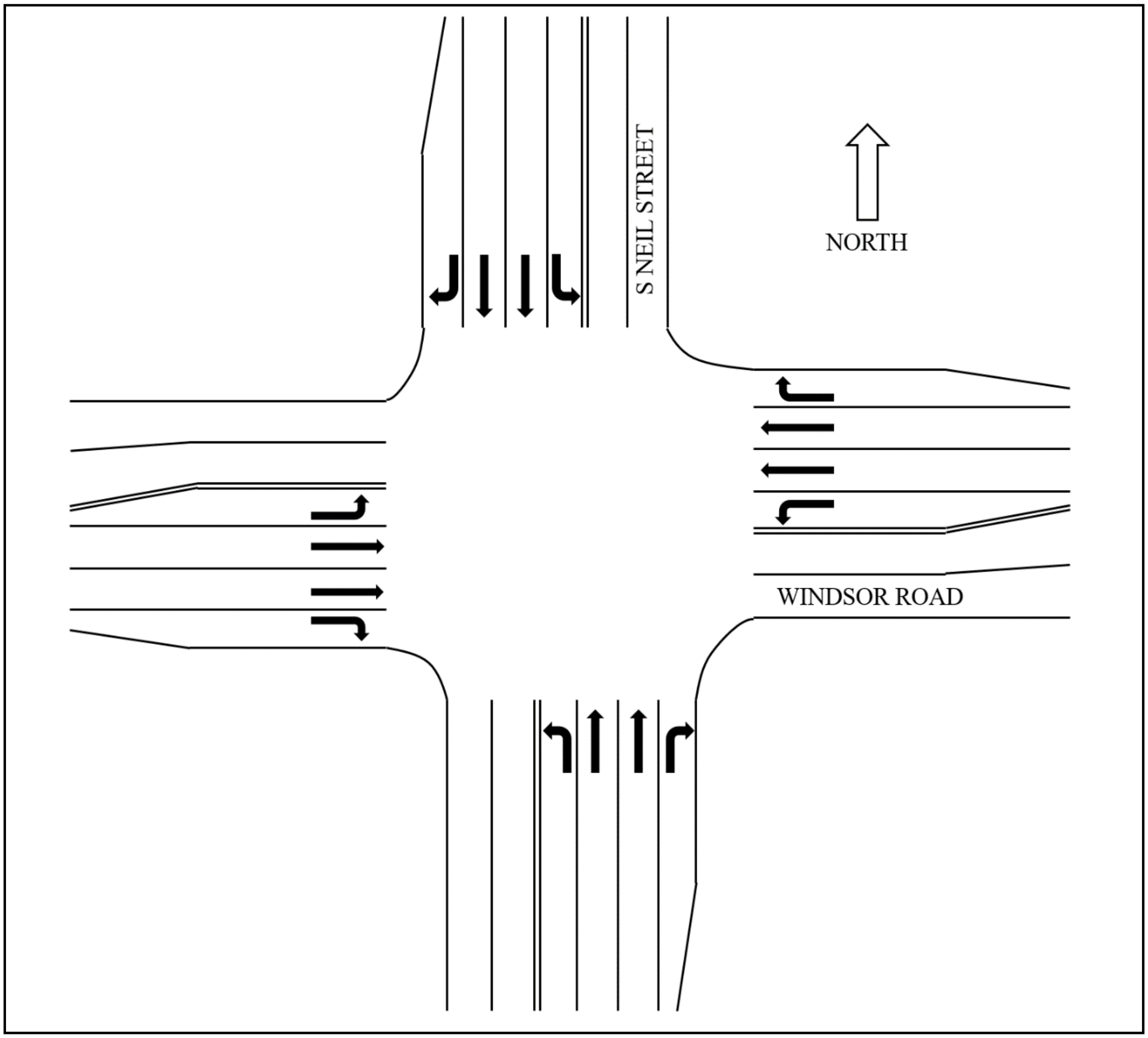


Figure 8. Geometry of the intersection of Neil Street and Windsor Drive.

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 than 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; 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 6. The traffic demand on its eastbound approach is very low compared to Neil Street and even to other cross streets. The westbound approach of the intersection at Neil Street and Knollwood Drive is only a driveway for a small commercial plaza (Figure 7). Thus, the traffic demand on both the east and west approaches of Knollwood Drive is much lower than that of the other streets crossing Neil Street. The remaining four intersections are regular four-legged intersections. Therefore, in the following chapters, the intersections at Neil Street and Knollwood Drive and Neil Street and Devonshire Drive are labeled as atypical intersections, and the remaining four are labeled as typical intersections.

2.2 FIELD DATA COLLECTION METHODOLOGY

The traffic data for the "before" conditions were collected between October 29 and December 11, 2013, at the six intersections on the Neil Street corridor. Two days of data per each intersection were collected on weekdays (avoiding Mondays and Fridays) during normal weather conditions. The dates corresponding to data collection at each intersection are shown in Table 1.

Table 1. Date and Day of Data Collection in the "Before" Condition

Intersection	Date	Day
Neil St. & Stadium Dr.	November 7, 2013	Thursday
	December 3, 2013	Tuesday
Neil St. & Kirby Ave.*	November 13, 2013	Wednesday
	November 19, 2013	Tuesday
Neil St. & St. Mary's Rd.**	November 20, 2013	Wednesday
	October 30, 2013	Wednesday
Neil St. & Devonshire Dr.	October 29, 2013	Tuesday
	December 5, 2013	Thursday
Neil St. & Knollwood Dr.	November 12, 2013	Tuesday
	December 5, 2013	Thursday
Neil St. & Windsor Rd.	November 5, 2013	Tuesday
	November 14, 2013	Thursday

*Additional PM data were collected at Neil Street and Kirby Avenue on December 10, 2013.

**Additional PM data were collected at Neil Street and St. Mary's Road on December 11, 2013.

Data collection was conducted by recording traffic conditions using video cameras. Camcorders were set up such that traffic conditions on each approach of an intersection were visible. Data were recorded during multiple time periods in a day as shown in Table 2 to get data during morning peak, off peak, noon peak, and pm peak. One of the two data collection days was selected for each intersection to perform traffic data reduction which is described in Chapter 3. The data reduction date was chosen considering that there is no presence of activities that could influence traffic like unusual weather conditions, construction, etc., and that the availability of data for the entire time period on that day. The days used in data reduction for each intersection are listed in Table 3.

Table 2. Data Collection Time Periods of a Day

Time Period	Data Collected From/To
Morning	7:00 AM–9:00 AM
Noon	10:30 AM–1:30 PM
Afternoon	4:00 PM–6:00 PM

Table 3. Traffic Data Reduction Dates

Intersection	Date	Day
Neil St. & Stadium Dr.	November 7, 2013	Thursday
Neil St. & Kirby Ave.	November 13, 2013	Wednesday
Neil St. & St. Mary's Rd.*	November 20, 2013	Wednesday
Neil St. & Devonshire Dr.	October 29, 2013	Tuesday
Neil St. & Knollwood Dr.	November 12, 2013	Tuesday
Neil St. & Windsor Rd.	November 5, 2013	Tuesday

*PM data at this intersection were obtained on December 11, 2013, because data were unavailable on November 20, 2013.

CHAPTER 3: DATA REDUCTION

This chapter describes the methodology used for reducing the traffic videos and presents the data obtained for each traffic characteristic of interest. Data reduction was conducted on the following:

- Peak hour
- Hourly volume
- Saturation flow rate
- Signal timing
- Proportion of vehicles stopping
- Arrival type
- Field delay
- Queue length
- Travel time

Using the videos that were recorded, data reduction was performed for the time periods shown in Table 4.

Table 4. Data Reduction Time Periods in the Videos

Time Period	Data Reduced From/To
Morning	7:10 AM–8:40 AM
Noon	10:40 AM–1:15 PM
Afternoon	4:40 PM–6:00 PM

In the following sections, a description of each data reduction item, the methodology used for obtaining it, and the outcome of the data efforts are provided.

3.1 PEAK HOUR

The peak hours during morning, noon, and afternoon time periods were determined. Further data reduction will be conducted for those hours to quantify traffic and perform traffic analyses. From the data collected around noon, an off-peak hour was also selected.

3.1.1 Methodology

The through 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 shown in Table 4, and 2-minute volumes were obtained for each.

The hour in the morning time period with the highest northbound total through volume at the abovementioned four intersections was designated the AM peak hour. Similarly, the hour in the afternoon time period with the highest southbound total through volume at those four intersections was designated the PM peak hour. The noon peak hour was the hour corresponding to the highest total through volume in both north- and southbound directions at those four intersections in the noon time period. The off=peak selected was an hour from the beginning of the data collection time for the noon time period.

3.1.2 Data

The peak hours computed using the above methodology are as shown in Table 5.

Table 5. Peak Hours 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

3.2 HOURLY VOLUME

The left, through, and right-turning movement volumes during the above-described peak hours were determined for all approaches of the six intersections. Those hourly volumes were used in capacity analyses, which will be discussed later in the report.

3.2.1 Methodology

The turning movement volumes were manually counted from the traffic videos recorded at all intersections for the duration of 1.5 hours, in which the peak/off-peak hour started from the 18th minute and ended at the 67th minute. The volume counts were obtained at 15-second intervals for the entire time period. The volume for the 3 peak hours and the off-peak hour were then used for further analysis.

3.2.2 Data

The hourly volume counts during the three peak hours and the off-peak hour are presented in Table 6. 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 the table that the demand on cross streets at the intersections of Neil Street with Devonshire Drive and Knollwood Drive is much lower. The cells with entries of N/A in the 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.

Table 6. Hourly Volume Counts Reduced

Intersection	Time Period	NB			SB			EB			WB		
		LT	Through	RT	LT	Through	RT	LT	Through	RT	LT	Through	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

3.3 SATURATION FLOW RATE

Field saturation flow rates for through lanes were measured for use in the capacity analysis.

3.3.1 Methodology

The field measurement technique for saturation flow rate as described in Chapter 31 of HCM 2010 was adopted to measure the base saturation flow rates for through lanes. For the through lanes, the measured base saturation flow rates did not require any adjustment, but for left- and right-turn lanes, it was adjusted as proposed by HCM.

A minimum of 50 "valid" headways was 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.

3.3.2 Data

The saturation flow rate calculated for the through lanes using the above procedure is presented in Figure 9. As shown in the figure, only the intersections of Neil Street with Kirby Avenue and Windsor Road were saturated enough. The remaining four intersections did not have adequate "valid" headway data to determine the saturation flow rate.

Measured saturation flow rate for northbound and southbound through lanes on Neil Street varied from 1908 to 1961 in the AM and PM peak periods. 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 through lanes on Neil Street at all intersections in all time periods. For EB and WB the measured saturation flow rate varied from 1722 to 1923 pcphgpl. The westbound of Kirby has a 3% upgrade and measured saturation flow rate there was 1722. For the eastbound and westbound through lanes at all intersections, it was decided to use a saturation flow rate of 1750 passenger cars per hour of green per lane (pcphgpl), except for the eastbound and westbound through lanes of Windsor Road, the eastbound through lanes of Kirby, and the eastbound through lanes of Devonshire, where 1900 pcphgpl was used.

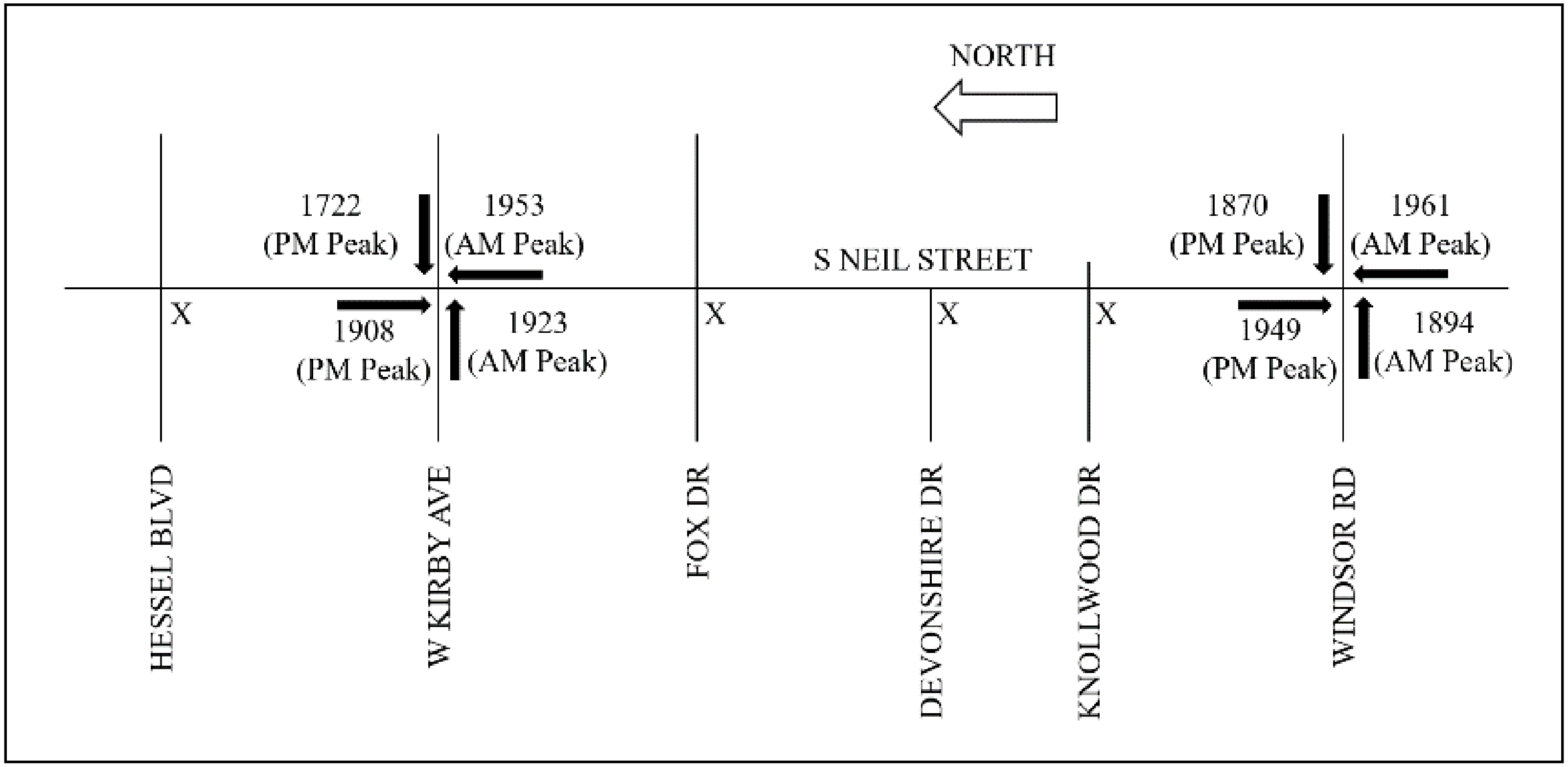


Figure 9. Geometry of the intersection of Neil Street and Windsor Drive.

X = Crossing street did not have adequate headway 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).

3.4 SIGNAL TIMING

This section discusses the methodology used for obtaining the signal timing data at all intersections. The phase sequence, phase splits, and cycle length were reduced from each cycle at an intersection. Owing to the presence of an actuated-coordinated signal control and time-based coordination plan, variable phase sequence and splits were observed, as expected. So, as a common practice, an average signal timing plan was developed for each intersection, which was representative of the behavior of its actuated signal operation during the peak hour. The data were obtained using the field videos. The signal timing information obtained from the field data and the signal controller settings information were used for carrying out the capacity analysis.

3.4.1 Methodology

The average phase sequence built at an intersection was the one that consisted of the most recurring phases in that peak hour. A phase was not considered in the average sequence if it occurred occasionally. The average phase splits were computed either by using the mean or the mode of the observed splits, whichever was more suitable. If a phase had a consistent green split over many cycles (at least more than half) in the peak hour, then the mode of the observed splits was selected as the green split for that phase in the average signal plan. Otherwise, the mean of the observed splits, excluding the outliers, was selected as the green split for that phase.

The yellow change and red clearance interval were obtained directly from the signal controller settings. As mentioned in the discussion of hourly volume reduction (Section 3.2), the traffic data for each intersection was gathered over a period of 1.5 hours for each time period. Thus, there could be a total of 45 cycles per interval if cycle length was 120 seconds. During the data reduction, cycles were observed until a clear phase plan pattern emerged. Between 10 and 39 cycles were analyzed per intersection to determine the average signal timing plan. This plan was used for the signal timing inputs for the capacity analysis.

3.4.2 Data

The cycle lengths during each peak hour are as shown in Table 7. The cycle length was 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 that intersection was operating at half-cycle with respect to the other five intersections.

Table 7. Cycle Length During Each Peak Hour

Peak Hour	Cycle Length
AM Peak	110 seconds
Off Peak	110 seconds
Noon Peak	110 seconds
PM Peak	120 seconds

The yellow change interval values were 3.2, 3.6, or 3.9 seconds; the red clearance was on the order of 2 seconds for different phases. The phase plan, green intervals, yellow change, and red clearance used in the signal timing of each intersection during the 4 peak hours are tabulated in the appendix.

3.5 PROPORTION OF VEHICLES STOPPING

The proportion of vehicles stopping in each lane group was calculated for each peak hour to estimate the arrival type for that lane group.

3.5.1 Methodology

The proportion of vehicles stopping in each lane group is equal to the number of stopped vehicles divided by the total volume of the peak hour for that lane group.

3.5.2 Data

Tables 8 through11 present the proportion of vehicles stopping in each lane group during the 4 peak hours. A blank entry indicates that an exclusive right-turn lane group was not present. The entries for Devonshire Drive WB are N/A because there is no westbound approach present at this intersection.

Table 8. Proportion of Vehicles Stopping During AM Peak Hour

AM Peak	NB			SB			EB			WB		
Neil St at	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
Stadium Dr	58.5%	25.9%		73.3%	39.8%		50.0%	63.9%		90.0%	67.1%	
Kirby Ave	71.6%	64.9%		93.6%	57.0%	27.7%	59.3%	52.4%		86.2%	87.9%	
St Marys Rd	54.5%	22.5%	18.3%	84.8%	53.5%		94.1%	89.5%		81.8%	80.8%	
Devonshire Dr	23.7%	7.8%			8.8%		91.4%		91.7%	N/A	N/A	N/A
Knollwood Dr	32.8%	3.8%		50.0%	7.6%		100.0%	90.9%		100.0%	42.9%	
Windsor Rd	32.1%	41.9%	31.5%	80.5%	29.4%	11.4%	70.5%	52.0%	16.2%	74.6%	59.2%	28.5%

Table 9. Proportion of Vehicles Stopping During Off-Peak Hour

Off Peak	NB			SB			EB			WB		
Neil St at	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
Stadium Dr	39.1%	19.7%		52.8%	34.8%		63.6%	82.9%		84.2%	94.4%	
Kirby Ave	65.3%	64.1%		71.8%	54.3%	23.2%	71.9%	62.9%		70.7%	52.5%	
St Marys Rd	16.7%	19.6%	12.0%	30.2%	13.6%		78.8%	80.4%		75.5%	70.8%	
Devonshire Dr	67.9%	20.3%			6.4%		92.6%		90.3%	N/A	N/A	N/A
Knollwood Dr	27.9%	6.2%		31.4%	2.8%		96.0%	56.5%		94.4%	52.4%	
Windsor Rd	65.0%	44.7%	21.3%	56.1%	28.8%	13.5%	74.1%	65.1%	42.1%	67.6%	62.9%	40.6%

Table 10. Proportion of Vehicles Stopping During Noon Peak Hour

Noon Peak	NB			SB			EB			WB		
Neil St at	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
Stadium Dr	62.2%	19.4%		63.2%	28.6%		68.8%	73.1%		77.6%	60.0%	
Kirby Ave	89.0%	81.0%		80.7%	65.4%	27.0%	73.8%	70.8%		66.7%	57.5%	
St Marys Rd	72.0%	28.6%	34.3%	53.6%	17.0%		88.5%	79.1%		85.5%	74.0%	
Devonshire Dr	23.4%	9.4%			6.9%		89.8%		97.9%	N/A	N/A	N/A
Knollwood Dr	36.1%	8.1%		39.1%	9.0%		92.3%	69.6%		82.1%	61.8%	
Windsor Rd	67.4%	58.5%	18.9%	33.6%	33.6%	63.0%	75.0%	63.6%	42.2%	73.7%	60.8%	27.5%

Table 11. Proportion of Vehicles Stopping During PM Peak Hour

PM Peak	NB			SB			EB			WB		
Neil St at	Left	Through	Right	Left	Through	Right	Left	Through	Right	Left	Through	Right
Stadium Dr	88.9%	35.0%		78.0%	44.3%		93.8%	58.6%		75.4%	58.5%	
Kirby Ave	88.1%	72.3%		94.1%	50.1%	14.3%	69.4%	46.1%		74.8%	87.1%	
St Marys Rd	30.4%	35.6%	35.8%	79.2%	22.0%		97.9%	91.4%		85.5%	77.9%	
Devonshire Dr	54.3%	10.1%			9.9%		86.4%		85.3%	N/A	N/A	N/A
Knollwood Dr	73.9%	4.3%		22.2%	10.9%		81.8%	78.7%		95.2%	38.5%	
Windsor Rd	91.0%	50.9%	26.4%	86.8%	39.1%	38.8%	67.1%	76.6%	74.5%	26.8%	72.2%	76.6%

3.6 ARRIVAL TYPE

Rather than using default values, field arrival types were estimated and used as inputs in the capacity estimation.

3.6.1 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 through movements on Neil Street at all intersections was estimated based on the proportion of vehicles stopped at each intersection and also by viewing the video to check when the platoons arrived during the cycle.

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

3.6.2 Data

The arrival types determined for Neil Street through movements are as shown in Table 12. As previously discussed in the section on methodology, the arrival type of all remaining movements in the study (i.e., Neil Street left-turn movements and all cross-street turning movements) is 3 for all four time periods.

Table 12. Arrival Types Determined from Neil Street Through 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

NBT = Northbound through movement.

SBT = Southbound through movement.

3.7 FIELD DELAY

The control delay and stopped delay in the field were calculated from the video data. The field measurements presented in this section will later be compared with their estimates obtained through capacity analysis (in Chapter 4).

3.7.1 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 identifying 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 the survey period was essentially equal to 1 hour for each peak hour and the off-peak hour. The count interval of 15 seconds was selected for this study because it is an integral divisor of the duration of survey period (1 hour) as required by the HCM.

3.7.2 Data

The control delay and stopped delay obtained for each lane group in the study (using the HCM field measurement methodology) are presented in Table 13 and Table 14, respectively. The cells with entries of N/A in signify that the respective lane group was not present at the subject approach.

Table 13. Control Delay at Lane Group Level Calculated Using the HCM 2010 Field Measurement Technique

Intersection	Time Period	NB			SB			EB			WB		
		LT	THROUGH	RT	LT	THROUGH	RT	LT	THROUGH	RT	LT	THROUGH	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 14. Stopped Delay at Lane Group Level Calculated Using the HCM 2010 Field Measurement Technique

Intersection	Time Period	NB			SB			EB			WB		
		LT	THROUGH	RT	LT	THROUGH	RT	LT	THROUGH	RT	LT	THROUGH	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

3.8 QUEUE LENGTH

The queue lengths in the field were calculated from the video data. The field measurements presented in this section will later be compared to their estimates obtained through capacity analysis (in Chapter 4).

3.8.1 Methodology

The queue length of a through-lane group of an intersection was calculated by manually counting the number of stopped vehicles at the beginning of the green light on a cycle-by-cycle basis for each peak hour. This count also includes vehicles that joined the queue after the end of the red light and came to a complete stop. Only the lane groups with a maximum queue length of at least were vehicles were considered for further analysis.

3.8.2 Data

The 50th, 85th, 90th, and 95th percentiles of the queue data were calculated from the raw data. Those values are as shown in Table 15.

Table 15. Queue Length Percentiles Calculated from Field Data

			50th Percentile	85th Percentile	90th Percentile	95th Percentile
Neil St. & Stadium Dr.	NB	AM Peak	2	6	6.5	7
		Noon Peak	1	2	2	2
		PM Peak	2	6	8.5	10
	SB	AM Peak	2.25	6.5	8	11
		Noon Peak	2	5	5	6
		PM Peak	3	6	7	7
	EB	AM Peak	2	4	5	5
		PM Peak	0	1	2	2
	WB	AM Peak	0	1	1	1
		PM Peak	2	5	7	9
Neil St. & Kirby Ave.	NB	AM Peak	9.5	21	22.5	23
		Off Peak	7	10	10	11
		Noon Peak	9	11	12.5	15
		PM Peak	11	14	14	18
	SB	AM Peak	4.5	6	7.5	8
		Off Peak	3.5	6	7	7
		Noon Peak	8.5	10.5	11	11.5
		PM Peak	8	14	14	14
	EB	AM Peak	6.5	10.5	11.5	12.5
		Off Peak	3	5	5	10
		Noon Peak	4	7	8.5	9
		PM Peak	3.75	5.5	6.25	6.5
	WB	AM Peak	3.5	5	5.25	7
		Off Peak	1.5	3.5	4.25	4.5
		Noon Peak	3	4.5	5	5.5
		PM Peak	9	10.5	10.5	11

(Table 15 continues next page)

Table 15 (continued)

			50th Percentile	85th Percentile	90th Percentile	95th Percentile
Neil St. & St. Mary's Road	NB	AM Peak	2	6	6	8
		Noon Peak	3.25	4.5	5	5.5
		PM Peak	3	6	7	7
	SB	AM Peak	4.25	6.5	6.75	8
		Noon Peak	1.5	3	4	6.5
		PM Peak	3	4	5	6
	EB	AM Peak	2	4	4	6
		PM Peak	2	4	5	5
	WB	AM Peak	1	2	3	4
		PM Peak	5	7	10	12
Neil St. & Devonshire Dr.	NB	AM Peak	1	2	3	3
		Noon Peak	1	2	2	3
		PM Peak	1	2	2	2
	SB	AM Peak	1	1	2	2
		Noon Peak	1	2	3	3
		PM Peak	1	2	3	4
Neil St. & Knollwood Dr.	NB	AM Peak	1	2	2	3
		Noon Peak	1	2	2.5	3
		PM Peak	1	1	2	2
	SB	AM Peak	1	3	3	6
		Noon Peak	1	2	2	3
		PM Peak	2	4	5	5
Neil St. & Windsor Rd.	NB	AM Peak	5	13	14	15
		Off Peak	3.25	6	7.75	9.5
		Noon Peak	5.25	7	7.25	8.5
		PM Peak	3.5	6	6.5	7
	SB	AM Peak	1	2	3	3
		Off Peak	2	4	4	5
		Noon Peak	3.25	4.5	5.75	6
		PM Peak	4.5	10	10	12
	EB	AM Peak	6	11	11	12
		Off Peak	2	4	5	6
		Noon Peak	3	5	5	6
		PM Peak	5	7	9	12
	WB	AM Peak	3.5	8	9	10
		Off Peak	3	5	5	5
		Noon Peak	3	5	5	6
		PM Peak	7.5	10	10	10

3.9 TRAVEL TIME

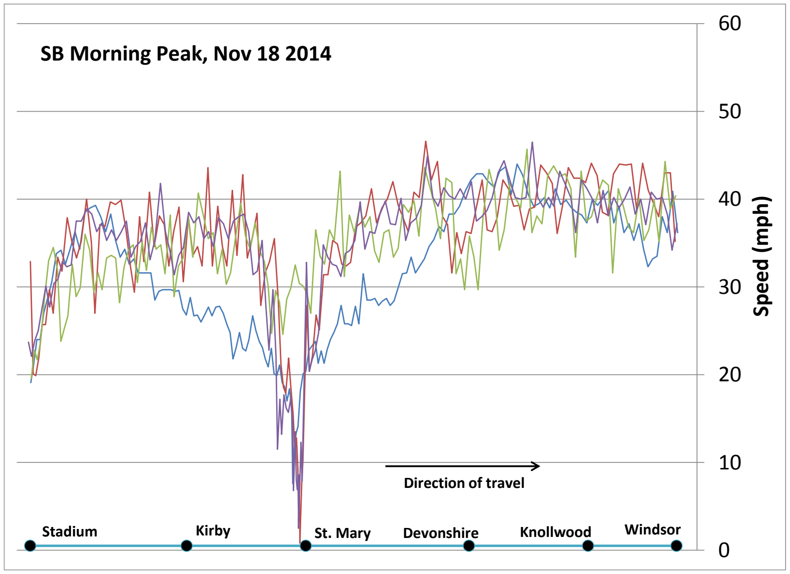
Travel-time data collection was performed in good weather conditions using the floating car method. A GPS unit was used to record the trajectory data for the test vehicle. The data were collected on 6 days in 2014 (October 28 and 29, November 18 and 19, and December 3 and 4), and 16 to 24 round trips were collected for each peak hour.

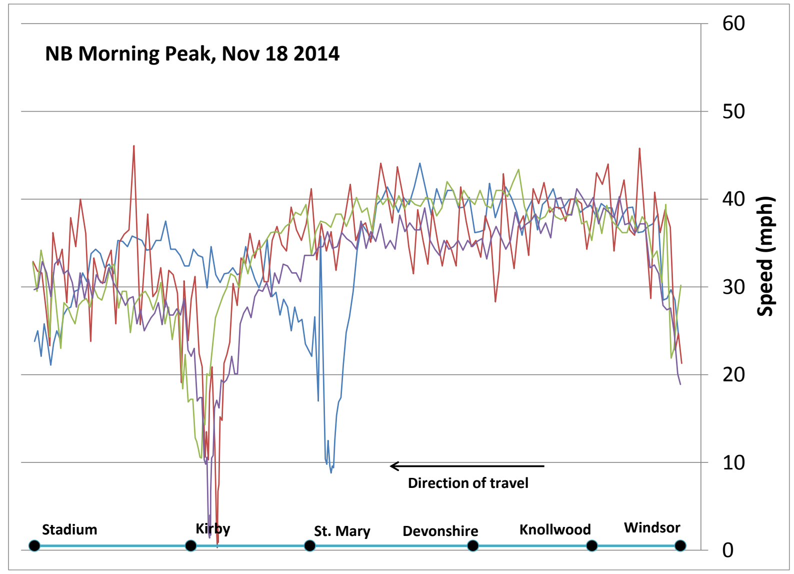
3.9.1 Methodology

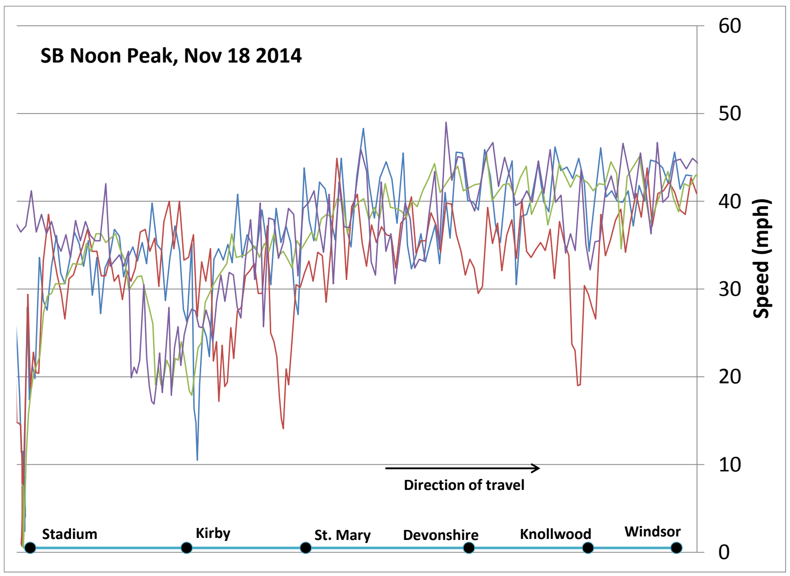
Travel time was found by driving along the corridor and recording the coordinates, speed, and time of the test vehicle every second using a GPS unit (mobile phone). An Android phone application called GeoTracker was used to record the vehicle trajectory in each run. This recorded file was later processed to obtain the travel time of the corridor. GPS Track Editor was the computer software used to process the .gpx files recorded in the field.

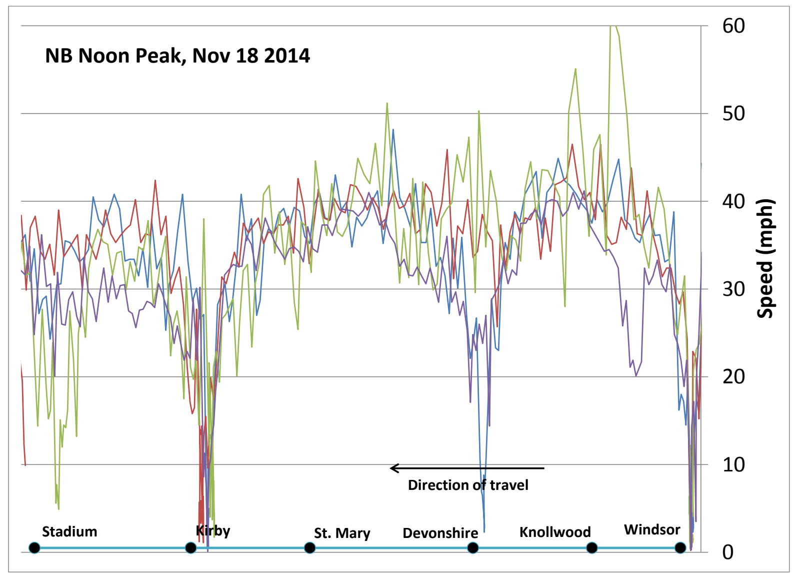
3.9.2 Data

The results of the travel-time data collection are presented on the following pages. Figure 10 shows the sample speed profiles for southbound and northbound trips during the AM, noon, and PM peaks on November 18, 2014. Figure 11 shows minimum, maximum, and average values of all travel-time runs performed for each link during the AM, noon and PM peaks. Table 16 shows minimum, maximum, and average values of all travel-time runs performed for an approach during each time period.


SB Morning Peak, Nov 18 2014
60
50
40
30
20
10
0
Speed (mph)
Direction of travel
Stadium
Kirby
St. Mary
Devonshire
Knollwood
Windsor


NB Morning Peak, Nov 18 2014
60
50
40
30
20
10
0
Speed (mph)
Direction of travel
Stadium
Kirby
St. Mary
Devonshire
Knollwood
Windsor


SB Noon Peak, Nov 18 2014
60
50
40
30
20
10
0
Speed (mph)
Direction of travel
Stadium
Kirby
St. Mary
Devonshire
Knollwood
Windsor


NB Noon Peak, Nov 18 2014
60
50
40
30
20
10
0
Speed (mph)
Direction of travel
Stadium
Kirby
St. Mary
Devonshire
Knollwood
Windsor

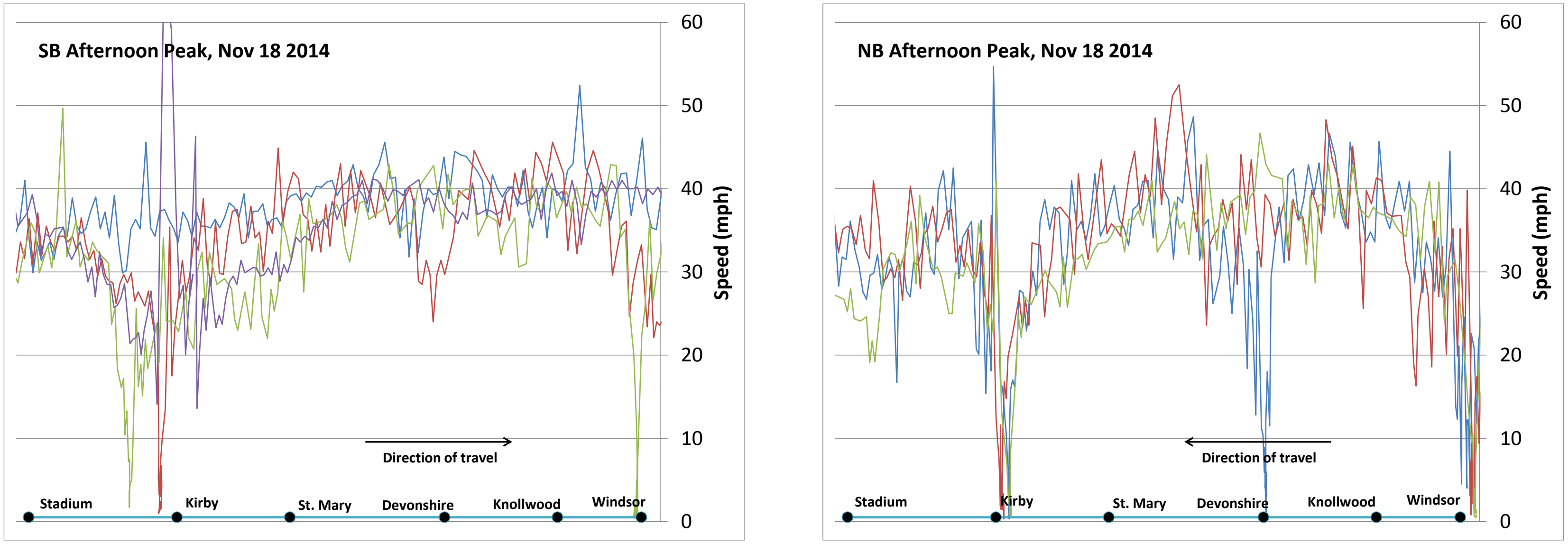


Figure 10. Sample speed profiles for southbound and northbound during the AM, noon, and PM peaks on November 18, 2014.

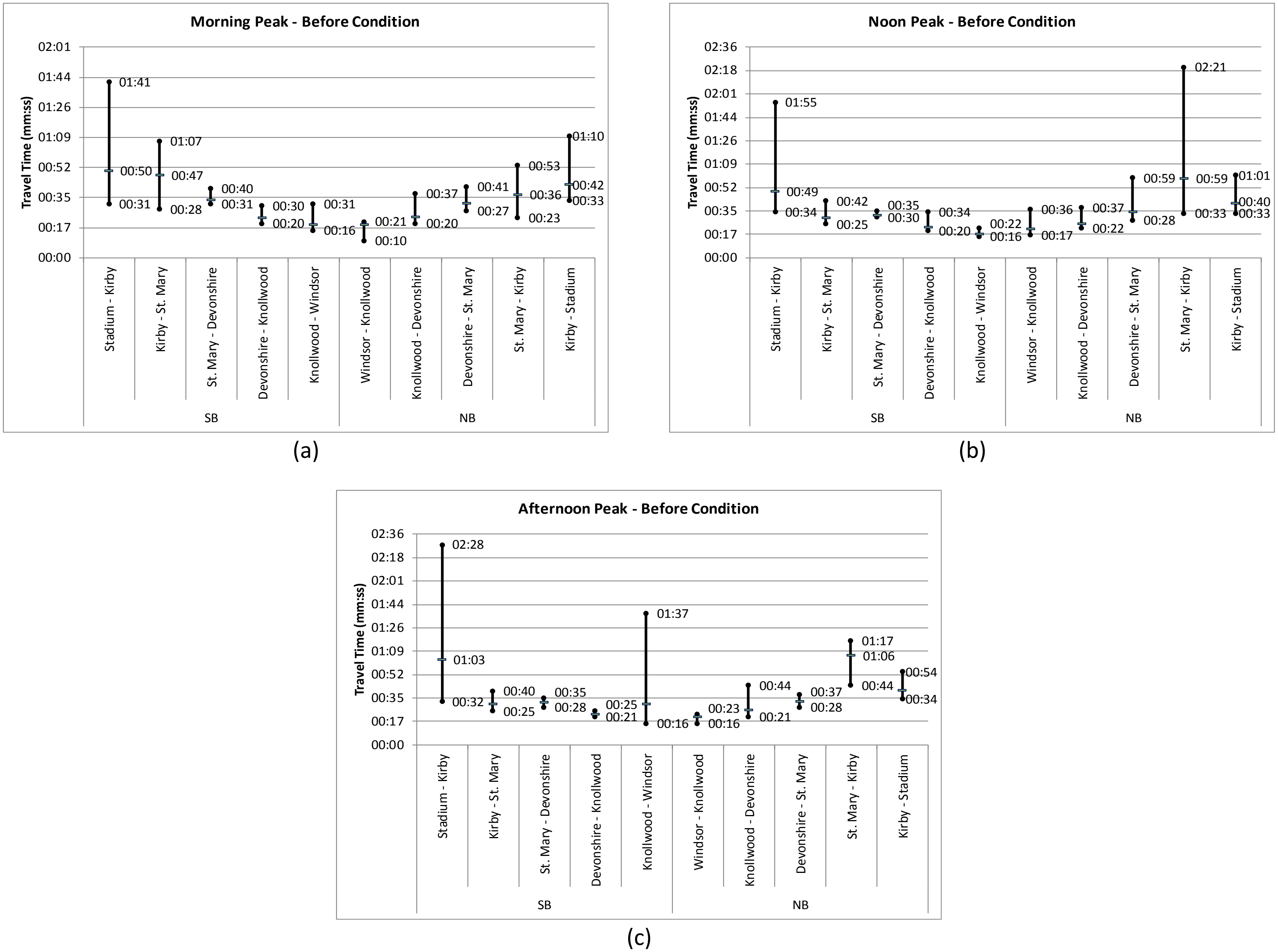


Figure 11. Maximum, minimum, and average travel time for each link: (a) AM peak, (b) noon peak, (c) PM peak.

Table 16. Results of Travel-Time Data Collection (in minutes:seconds)

		Minimum	Maximum	Average
AM Peak	SB	02:15	03:58	02:52
	NB	02:13	03:11	02:33
Off Peak	SB	02:07	03:24	02:36
	NB	02:36	03:36	03:01
Noon Peak	SB	02:08	03:37	02:33
	NB	02:29	04:16	02:59
PM Peak	SB	02:05	04:26	02:55
	NB	02:49	03:38	03:04

CHAPTER 4: DATA ANALYSIS

This chapter explains three data analyses: HCM estimates vs. field stopped delay, HCM estimates vs. field queue length, and exploring the relationships between the results of the two comparisons. For the first two analyses, the methodology used for comparison is first explained. It is then followed by statistical comparison, detailed results, and discussion. For the third analysis, the methodology is briefly explained, after which the results are discussed.

4.1 DELAY COMPARISON

4.1.1 Methodology

The delay comparison in this study was made between HCM stopped delay estimates and respective field measurements on a lane-group basis. The only lane groups considered are protected left-turn lanes, through lanes, and protected right-turn lanes. 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.

4.1.1.1 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 the correction factor can be obtained from Exhibit 31-48 in Chapter 31 of HCM 2010.

The stopped delay value of a lane group is more directly obtained from field and does not contain any corrections, as in control delay calculation. Also, the HCM estimate of control delay includes an adjustment factor of 1.3 for uniform delay and incremental delay components. This factor is essentially meant to increase the stopped delay by 30% to account for 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 related to corrections.

4.1.1.2 Permitted and Protected-Permitted Lane Groups

The field delay for permitted and protected-permitted turning movements highly depends on the availability of gaps in the opposing traffic stream. The delay for permitted and protected-permitted left turn is especially correlated with the volume and arrival pattern of the opposing through movements. It is thus believed that those 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.

4.1.1.3 Capacity Analysis

Highway Capacity Software (HCS) 2010 version 6.70 was used to perform capacity analysis for all intersections. 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 corridor. For the major street approaches, arrival types were determined based on field data and used in HCS to reflect effects of signal coordination.

Figure 12 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 correspond to the AM peak period (7:30 AM–8:30 AM). 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. A pre-timed signal analysis was performed for each intersection for the four time periods in the HCS.

The reduced field data (discussed in Chapter 3) were used as inputs for demand, saturation flow rate, phasing, signal timing, and arrival types in the HCS runs. On the minor approaches at Neil Street with Stadium Drive and St. Mary's Road, right-turning and through traffic share the same lanes, but the shared lanes are wide enough to allow right-turning vehicles to skip the through-vehicle queue and make a turn right on red (RTOR). Therefore, RTOR volumes were determined (Table 17) and then excluded from the right-turn volume in HCS runs for those two intersections (as suggested by HCM).

The demand in the multi-period analysis was equal to the 15-minute aggregated volume counts for each movement. As shown in Figure 12, the cycle length at the Stadium Drive intersection is 55 seconds, instead of 110 seconds, because it operates at a half-cycle with respect to the other five intersections, as previously stated. The signal can be coded as a fixed operation by checking the Pre-Timed Signal box in the Phasing section, as done in Figure 12. The signal timings in the Timing section (such as phase split, yellow change, red clearance, and minimum green) were set equal to the values presented in the previous chapter.

After the HCS was run, the hourly 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% (i.e., dividing the latter by a factor of 1.3). Those stopped delays were used in the statistical analysis as described in the next section.

Capacity analysis in HCS was also used in the queue length comparison. The details will be discussed in the queue length comparison section.

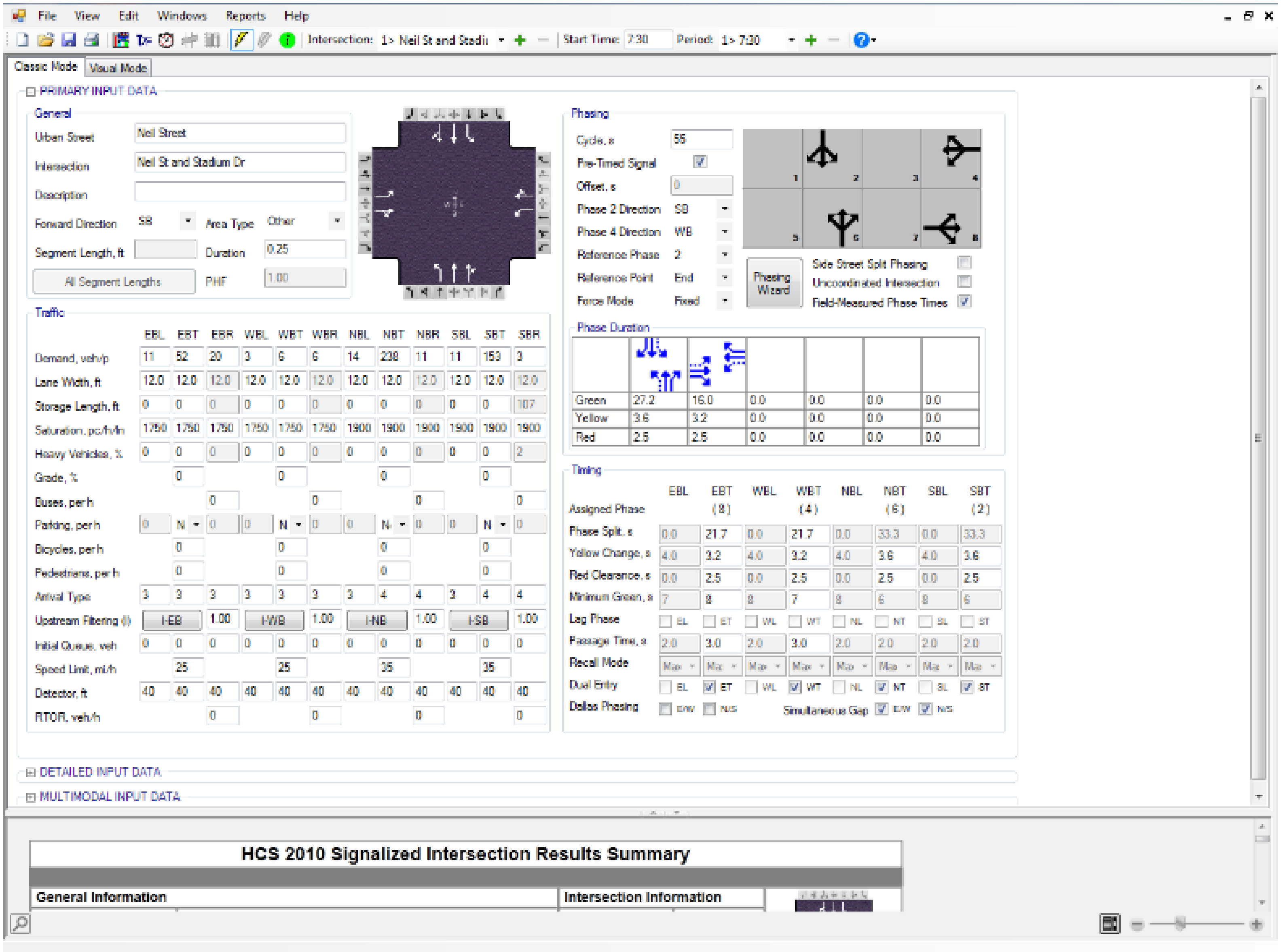


Figure 12. Screenshot of HCS run showing the input data at Neil Street and Stadium Drive for AM peak period.

Table 17. RTOR Volume for Stadium Drive and St. Mary's Road

Intersection	Approach	Time Period	RTOR Volume				
			0:00–0:15	0:15–0:30	0:30–0:45	0:45–1:00	Total
Neil St. & Stadium Dr.	EB	AM Peak	11	11	10	11	43
		Off Peak	4	8	9	2	23
		Noon Peak	7	10	4	2	23
		PM Peak	9	4	6	1	20
	WB	AM Peak	3	2	3	5	13
		Off Peak	1	3	8	7	19
		Noon Peak	7	10	4	5	26
		PM Peak	16	7	10	11	44
Neil St. & St. Mary's Rd.	EB	AM Peak	3	9	3	4	19
		Off Peak	10	3	3	12	28
		Noon Peak	2	13	7	11	33
		PM Peak	7	10	8	6	31
	WB	AM Peak	10	6	1	4	21
		Off Peak	3	11	14	25	53
		Noon Peak	14	11	19	15	59
		PM Peak	30	23	15	18	86

4.1.1.4 One-Sample T-Test

Statistical comparison was performed using the 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 the equation below.

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

In this equation, μ0 is the HCM stopped delay estimate of the subject lane group. $\bar{x}$ is the average stopped delay per vehicle of that lane group observed from the field, and s^2 is its variance. The field variance of stopped delay of a lane group was obtained by measuring average 3-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 those 20 observations is equal to the field variance s^2. The observation time of 3 minutes was deliberately chosen in order to capture traffic data of at least one complete cycle (110 or 120 seconds) in each time interval.

Thus, using this methodology, the differences are tested to determine whether they are statistically significant.

4.1.2 Statistical Delay Comparison

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

The details of the t-tests performed are presented in Table 18. The column heading “n” in the table stands for the number of 3-minute delay observations obtained from the field for the subject lane group. The column heading “df” stands for degrees of freedom of the t-test, which is 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, southbound, eastbound, and westbound through-lane groups, respectively. EBL stands for eastbound left-lane group. Those abbreviations will also be used in subsequent tables and graphs. Some tests in the table had the number of delay observations (n) less than 20 because the data for those time periods were available for less than 1 hour.

There were a total of 84 tests performed over the four time periods: 80 for the through-lane groups and four 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 10%. The tests in which HCM significantly overestimated the stopped delay with respect to the field measurement are highlighted with red in Table 18 and the underestimations with blue.

Table 18. Statistical Comparison Between HCM Stopped Delay Estimates and Field Measurements

			HCS Delay	Field N	Field Mean	Field Variance	df	T-statistic	P-value
Neil St. & Stadium Dr.	AM Peak	NBT	6.218	20	3.693	3.315	19	6.2036	< .00001
		SBT	4.873	20	5.362	8.186	19	–0.7643	0.454251
		EBT	14.446	19	10.925	15.999	18	3.8376	0.001206
		WBT	11.301	19	10.500	95.765	18	0.3568	0.72539
	Off Peak	NBT	2.270	20	2.339	4.542	19	–0.1436	0.887796
		SBT	2.277	20	5.013	12.128	19	–3.5137	0.002326
		EBT	15.412	20	20.250	328.494	19	–1.1938	0.247555
		WBT	15.191	20	14.159	150.708	19	0.3762	0.710936
	Noon Peak	NBT	4.445	20	2.009	1.758	19	8.2173	< .00001
		SBT	4.621	20	2.817	1.846	19	5.9356	0.00001
		EBT	13.083	20	14.798	87.858	19	–0.8181	0.423498
		WBT	13.527	20	13.050	91.207	19	0.2234	0.825685
	PM Peak	NBT	6.912	19	4.063	5.381	18	5.3523	0.000044
		SBT	8.240	19	6.313	6.971	18	3.1813	0.005171
		EBT	11.496	14	10.241	125.672	13	0.4188	0.682199
		WBT	14.063	18	10.549	17.359	17	3.5780	0.002317
Neil St. & Kirby Ave.	AM Peak	NBT	36.505	20	17.666	123.228	19	–7.5894	< .00001
		SBT	18.0473	20	15.959	53.706	19	–1.2742	0.217993
		EBT	30.4821	20	18.682	80.037	19	–5.8989	0.000011
		WBT	21.4812	20	35.226	185.857	19	4.5087	0.000241
	Off Peak	NBT	24.4022	20	17.992	34.790	19	–4.8606	0.000109
		SBT	18.8242	20	16.704	35.814	19	–1.5842	0.129674
		EBT	22.6023	20	21.992	38.105	19	–0.4422	0.663408
		WBT	22.1956	20	16.820	91.563	19	–2.5125	0.021179
	Noon Peak	NBT	32.0777	20	23.131	46.721	19	–5.8537	0.000012
		SBT	26.7305	20	22.945	53.732	19	–2.3098	0.032297
		EBT	24.4675	20	20.537	41.863	19	–2.7168	0.013684
		WBT	23.0274	20	17.262	88.695	19	–2.7376	0.013082
	PM Peak	NBT	26.1591	20	27.449	82.068	19	0.6366	0.532363
		SBT	19.8123	20	20.682	110.902	19	0.3694	0.716208
		EBT	34.5816	20	21.756	98.346	19	–5.7836	0.000014
		WBT	36.8361	20	33.028	122.086	19	–1.5413	0.139734

(Table 18 continues next page)

Table 18 (continued)

			HCS Delay	Field N	Field Mean	Field Variance	df	T-statistic	P-value
Neil St. & St. Mary's Rd.	AM Peak	NBT	8.085	20	5.807	12.571	19	2.8734	0.009731
		SBT	11.057	20	8.746	11.963	19	2.9886	0.00755
		EBT	41.512	20	34.247	1228.195	19	0.9271	0.365553
		WBT	37.482	20	22.154	230.998	19	4.5102	0.00024
	Off Peak	NBT	1.462	20	2.691	2.671	19	–3.3635	0.003267
		SBT	1.679	20	1.990	2.792	19	–0.8304	0.416846
		EBT	40.477	20	29.641	573.941	19	2.0227	0.057408
		WBT	42.707	20	29.813	746.569	19	2.1104	0.048308
	Noon Peak	NBT	12.865	20	5.664	12.400	19	9.1447	< .00001
		SBT	3.029	20	2.771	4.100	19	0.5684	0.57642
		EBT	37.202	20	23.686	174.334	19	4.5778	0.000206
		WBT	42.053	20	17.175	175.417	19	8.4005	< .00001
	PM Peak	NBT	8.540	20	7.717	17.959	19	0.8685	0.396012
		SBT	4.865	20	4.958	12.667	19	–0.1172	0.908088
		EBT	39.947	20	32.516	773.410	19	1.1950	0.246829
		WBT	46.296	20	26.196	133.494	19	7.7803	< .00001
Neil St. & Devonshire Dr.	AM Peak	NBT	0.339	20	0.875	0.459	19	3.5430	0.002177
		SBT	0.200	20	1.254	1.035	19	4.6351	0.000181
		EBL	40.662	20	43.971	314.163	19	0.8351	0.414094
	Off Peak	NBT	0.099	20	2.873	8.776	19	4.1874	0.0005
		SBT	0.272	20	0.989	1.137	19	3.0071	0.007248
		EBL	37.707	20	37.000	324.628	19	–0.1754	0.862699
	Noon Peak	NBT	0.154	20	1.111	1.151	19	3.9909	0.000784
		SBT	0.343	20	0.796	0.470	19	2.9582	0.008078
		EBL	37.219	20	44.471	700.865	19	1.2250	0.235546
	PM Peak	NBT	0.137	20	1.067	1.883	19	3.0325	0.006858
		SBT	0.687	20	0.938	1.427	19	0.9394	0.359516
		EBL	41.903	20	43.557	1078.859	19	0.2252	0.824381
Neil St. & Knollwood Dr.	AM Peak	NBT	0.491	13	0.331	0.155	12	–1.4726	0.166597
		SBT	0.154	15	1.033	8.475	14	1.1690	0.261923
	Off Peak	NBT	0.155	20	0.495	0.657	19	1.8796	0.075674
		SBT	0.095	20	0.095	0.048	19	0.0083	0.9937
	Noon Peak	NBT	0.232	20	0.717	0.443	19	3.2577	0.004148
		SBT	0.192	20	0.674	1.145	19	2.0160	0.058277
	PM Peak	NBT	0.231	20	0.335	0.447	19	0.6983	0.493632
		SBT	0.722	20	0.691	0.735	19	–0.1594	0.875114

(Table 18 continues next page)

Table 18 (continued)

			HCS Delay	Field			df	T-statistic	P-value
				N	Mean	Variance			
Neil St. & Windsor Rd.	AM Peak	NBT	19.753	20	9.999	54.620	19	–5.9019	0.000011
		SBT	14.249	20	6.539	23.301	19	–7.1426	< .00001
		EBT	30.543	20	15.228	20.447	19	–15.1467	< .00001
		WBT	31.452	20	23.207	113.032	19	–3.4684	0.002573
	Off Peak	NBT	14.702	20	9.199	37.656	19	–4.0106	0.000748
		SBT	14.835	20	6.310	14.173	19	–10.1270	< .00001
		EBT	25.150	20	23.778	144.227	19	–0.5110	0.615238
		WBT	23.887	20	20.419	126.743	19	–1.3775	0.184369
	Noon Peak	NBT	19.538	20	14.538	45.497	19	–3.3148	0.003643
		SBT	15.645	20	8.194	23.824	19	–6.8271	< .00001
		EBT	26.160	20	24.089	91.220	19	–0.9694	0.344525
		WBT	26.012	20	21.288	214.936	19	–1.4409	0.165891
	PM Peak	NBT	15.758	20	16.081	71.054	19	0.1713	0.866032
		SBT	15.821	16	12.317	45.556	15	–2.0762	0.055481
		EBT	29.281	20	27.073	124.702	19	–0.8841	0.387693
		WBT	35.347	9	26.481	54.594	8	–3.5998	0.006985

4.1.3 Results and Discussion

The delay comparisons in Table 18 were classified into four categories: (1) typical intersections, (2) atypical intersections, (3) major streets at typical intersections, and (4) minor streets at typical intersections. Summaries of the data in Table 18 for all categories combined and for each category are shown in Tables 19 through 23. For each table, the column heading "%" stands for the percentage of cases with significant discrepancy. The table cells with entries of "—" signify that the respective data were not applicable and are not presented. The table cells with entries of "N/A" signify that the respective data were unavailable.

For typical intersections, cases with significant discrepancies were plotted in both number of vehicles and percentage, as shown in Figure 13. The average rates of significant overestimation (extreme cases excluded, if any) were computed for major and minor streets and were also plotted in the graphs as average lines.

Table 19. Summary of Delay Comparison

Overall			
	No. of Cases	**%**	**Range of (HCM – Field)/Field%**
Total	84	—	—
Significant Discrepancy	49	58%	(97) – 145%
Overestimation	36	73%	17 – 145%
Underestimation	13	27%	(97) – (39%)

Table 20. Summary of Delay Comparison for Typical Intersections

Category 1: Typical Intersections			
	No. of Cases	**%**	**Range of (HCM – Field)/Field%**
Total	64	—	—
Significant Discrepancy	39	61%	(55) – 145%
Overestimation	36	92%	17 – 145%
Underestimation	3	8%	(55) – (39%)

Table 21. Summary of Delay Comparison for Atypical Intersections

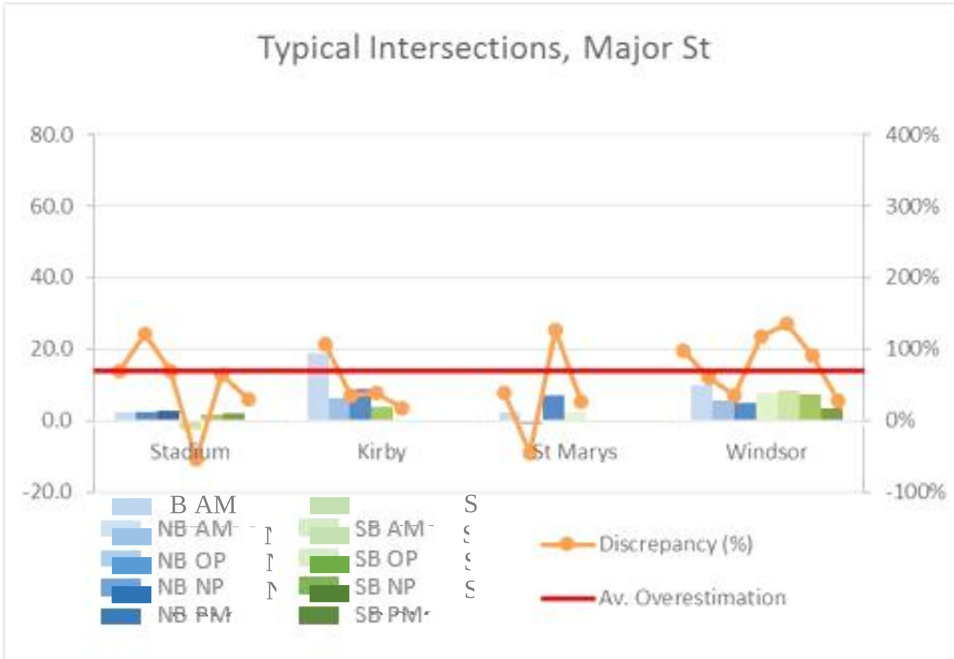
Category 2: Atypical Intersections				
	No. of Cases	**%**	**Range of (HCM – Field)/Field%**	**Average Discrepancy %**
Total	20	—	—	—
Significant Discrepancy	10	50%	(97) – (57)%	—
Overestimation	0	0%	N/A	N/A
Underestimation	10	100%	(97) – (57)%	(75)%

Table 22. Summary of Delay Comparison for Major Street Cases at Typical Intersections

Category 3: Typical Intersections, Major Street				
	No. of Cases	**%**	**Range of (HCM – Field)/Field%**	**Average Discrepancy %**
Total	32	—	—	—
Significant Discrepancy	23	72%	(55) – 135%	—
Overestimation	21	91%	17 – 135%	69%
Underestimation	2	9%	(55)-(46)%	–50%

Table 23. Summary of Delay Comparison for Minor Street Cases at Typical Intersections

Category 4: Typical Intersections, Minor Streets				
	No. of Cases	**%**	**Range of (HCM – Field)/Field%**	**Average Discrepancy %**
Total	32	—	—	—
Significant Discrepancy	18	56%	(39) – 145%	—
Overestimation	17	94%	19 – 145%	52%
Underestimation	1	6%	(39)%	(39)%



(a).

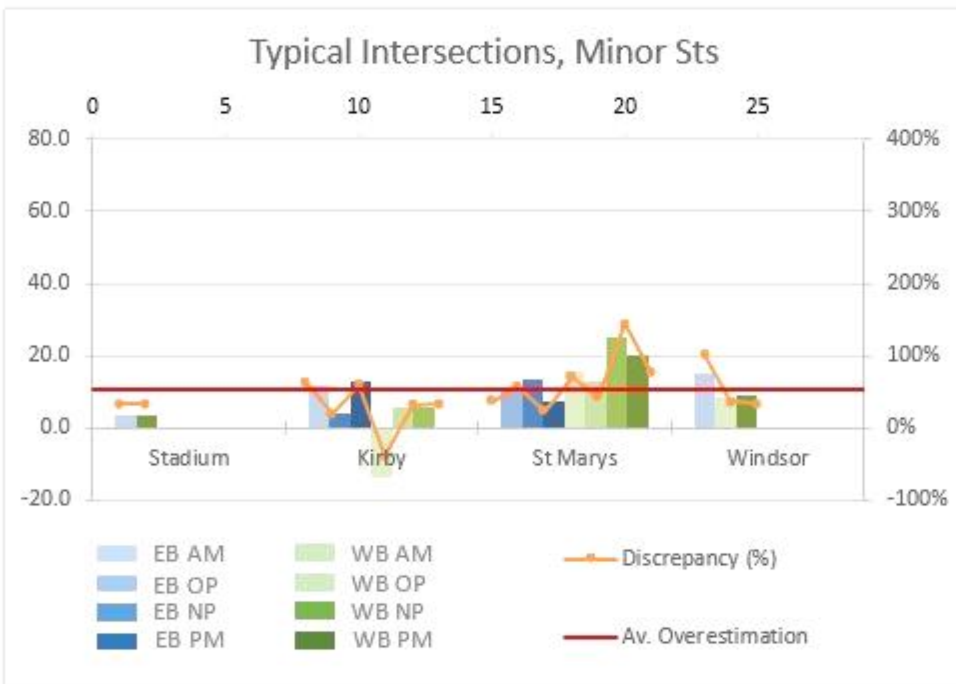


Figure 13. Cases with significant discrepancies for typical intersections in delay comparison: (a) major street cases, (b) minor street cases.

The following are the findings of the delay comparison:

- Of 84 total tests, 49 had statistically significant discrepancies in the estimation. In other words, the HCM estimates of stopped delay were not accurate in 58.3% of the cases.
- Of the 49 significant errors, HCM overestimated the stopped delay in 36 cases (73.5%) and underestimated it in 13 cases (26.5%) with respect to field measurements.
- Of the 20 cases at atypical intersections, HCM estimates in ten (50%) were significantly different from the field, all of which were underestimations.
- For the 64 cases at typical intersections, 39 (61%) had statistically significant discrepancies in the estimations, including 36 (92%) overestimations and three (8%) underestimations. Similar trends were observed in the cases of streets classified as major and minor. For the major street cases, 23 of 32 (72%) had significant discrepancies, of which 21 (91%) were overestimations and two (9%) were underestimations. For minor streets, 18 of 32 (56%) had significant discrepancies, with 17 (94%) overestimations and 1 (6%) underestimation.
- For the 21 major street cases at typical intersections that were significantly overestimated, the discrepancies ranged from 17% to 135%, and the average discrepancy was 69%. For the 17 significant overestimations for minor streets at typical intersections, the discrepancies ranged from 19% to 145%, and the average discrepancy was 52%.

Shaik (2016) compared the delay estimates from HCM to the field data, assuming that the traffic signals were actuated (used the same cycle length measured in the field but allowing the computer to decide the phase plan and green time allocation). The proportion of discrepancies observed was similar (58.3% vs. 61.9%). However, the proportion of overestimations and underestimations observed was not similar (73.5% and 26.5% vs. 59.6% and 40.4%). It should be noted that it is more meaningful to compare field data to the HCS results that closely represent the traffic operation condition at the time of data collection. This is accomplished when the HCS delays are obtained using average signal timing information observed in field rather than the HCS-calculated average phase durations for actuated signals.

4.2 QUEUE LENGTH COMPARISON

4.2.1 Methodology

The queue length comparison in this study was done between the HCM back-of-queue estimates and the respective field measurements for through-lane groups. Only the lane groups with a maximum queue length of at least 2 vehicles were considered in the comparison. Statistical comparison was performed using the single-sample Wilcoxon signed-rank test with 90% confidence level in order to perform a non-parametric median (50th percentile) comparison.

Similar to the delay comparison, HCS was used to compute queue length. The same HCS models that were used for delay were used for queue estimation, except that the queue length estimates were obtained using hourly volumes with a peak hour factor of 1.0. The RTOR volumes for minor streets at the intersections of Neil Street and Stadium Drive and Neil Street and St. Mary’s Road were also excluded from the input demand. The back-of-queue estimates corresponding to the 50th, 85th, 90th, and 95th percentile were obtained. The median estimate (50th percentile) was later statistically compared with the median of the field queue length data. Graphical comparisons were made between the estimated 95th percentile queue lengths and field measurements.

4.2.2 Statistical Queue Length Comparison

Using the aforementioned methodology for comparison, Wilcoxon tests were performed for all through-lane groups in the study area for the four time periods that had at least two vehicles in the queue. There were 64 cases in total that met this criterion.

For the 50th percentile queue length comparison, the details of the Wilcoxon tests performed are presented in Table 24. A comparison was considered significant only if the p-value of its Wilcoxon test was less than 10%. The tests in which HCM significantly overestimated the queue length with respect to the field measurement are highlighted with red in the table and the underestimations with blue. The table cells with entries of “N/A” signify that the respective discrepancy in percentage was unavailable because the corresponding field data was zero and could not be divided.

The 95th percentile queue length comparison is presented in Figure 14. Because eastbound and westbound cases did not have data during the noon peak period at the intersections of Neil Street with Stadium Drive and Neil Street with St. Mary’s Road, the northbound and southbound noon peak cases at those two intersections were not presented in the graphs. The columns represent the discrepancies between HCM estimation and field queue length in terms of numbers of vehicles, and the curves show those discrepancies in percentages. The cases where significant discrepancies were observed in 50th percentile queue length comparison are highlighted in the graphs (overestimations with red triangles and underestimations with blue rectangles).

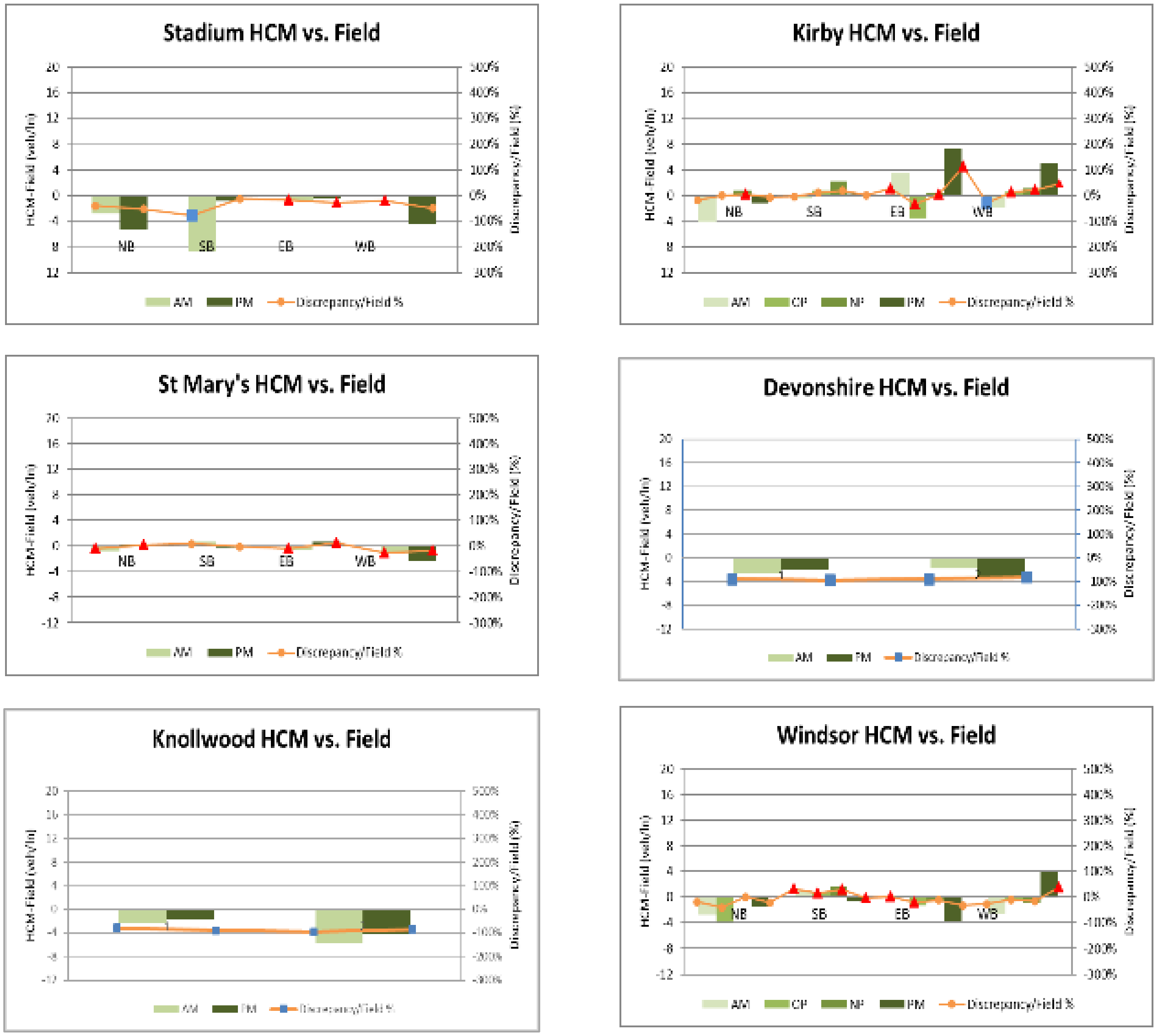
Table 24. Statistical Comparison Between HCM 50th Percentile Back-of-Queue Estimates and Field Measurements

			Field Data	HCS	Discrepancy (veh)	Discrepancy (%)	P-Value
Neil St. & Stadium Dr.	NBT	AM Peak	2	2.3	0.3	15%	0.88500
		Noon Peak	1	1.5	0.5	50%	0.02670
		PM Peak	2	2.6	0.6	30%	0.46922
	SBT	AM Peak	2.25	1.3	–0.95	–42%	0.01345
		Noon Peak	2	1.7	–0.3	–15%	0.30180
		PM Peak	3	3.4	0.4	13%	1.00000
	EBT	AM Peak	2	2.3	0.3	15%	0.00057
		PM Peak	0	0.8	0.8	#DIV/0!	0.00015
	WBT	AM Peak	0	0.4	0.4	#DIV/0!	0.00003
		PM Peak	2	2.5	0.5	25%	0.13119
Neil St. & Kirby Ave.	NBT	AM Peak	9.5	13	3.5	37%	0.15592
		Off Peak	7	6.7	–0.3	–4%	0.16104
		Noon Peak	9	10.5	1.5	17%	0.00743
		PM Peak	11	11.3	0.3	3%	0.83348
	SBT	AM Peak	4.5	4.3	–0.2	–4%	0.95058
		Off Peak	3.5	4.4	0.9	26%	0.22287
		Noon Peak	8.5	8.8	0.3	4%	0.22435
		PM Peak	8	9.3	1.3	16%	0.20548
	EBT	AM Peak	6.5	10.6	4.1	63%	0.00001
		Off Peak	3	3.6	0.6	20%	0.08665
		Noon Peak	4	5.6	1.6	40%	0.00277
		PM Peak	3.75	8.9	5.15	137%	<0.00001
	WBT	AM Peak	3.5	2.8	–0.7	–20%	0.00319
		Off Peak	1.5	2.9	1.4	93%	0.00243
		Noon Peak	3	3.7	0.7	23%	0.03563
		PM Peak	9	10.7	1.7	19%	0.00001
Neil St. & St. Mary's Rd.	NBT	AM Peak	2	4	2	100%	0.03361
		Noon Peak	3.25	6.2	3.2	107%	<0.00001
		PM Peak	3	4	1	33%	0.26403
	SBT	AM Peak	4.25	5	2	67%	0.15751
		Noon Peak	1.5	1.8	0.8	80%	0.28357
		PM Peak	3	3.2	0.2	7%	0.29175
	EBT	AM Peak	2	2.9	0.9	45%	0.00055
		PM Peak	2	3.1	1.1	55%	0.00006
	WBT	AM Peak	1	1.6	0.6	60%	0.00017
		PM Peak	5	5.7	0.7	14%	0.00005

(Table 24 continues next page)

Table 24 (continued)

			Field Data	HCS	Discrepancy (veh)	Discrepancy (%)	P-Value
Neil St. & Devonshire Dr.	NBT	AM Peak	1	0.2	–0.8	–80%	0.00006
		Noon Peak	1	0.1	–0.9	–90%	0.00004
		PM Peak	1	0.1	–0.9	–90%	0.00007
	SBT	AM Peak	1	0.1	–0.9	–90%	0.00033
		Noon Peak	1	0.2	–0.8	–80%	0.00002
		PM Peak	1	0.4	–0.6	–60%	0.00001
Neil St. & Knollwood Dr.	NBT	AM Peak	1	0.3	–0.7	–70%	0.01744
		Noon Peak	1	0.1	–0.9	–90%	0.00009
		PM Peak	1	0.1	–0.9	–90%	0.00049
	SBT	AM Peak	1	0.1	–0.9	–90%	0.01253
		Noon Peak	1	0.1	–0.9	–90%	0.00002
		PM Peak	2	0.4	–1.6	–80%	<0.00001
Neil St. & Windsor Rd.	NBT	AM Peak	5	7.7	2.7	54%	0.13796
		Off Peak	3.25	3.1	–0.15	–5%	0.14944
		Noon Peak	5.25	5	–0.25	–5%	0.39180
		PM Peak	3.5	3	–0.5	–14%	0.16754
	SBT	AM Peak	1	2.2	1.2	120%	0.00002
		Off Peak	2	3.2	1.2	60%	0.00087
		Noon Peak	3.25	4.3	1.05	32%	0.00113
		PM Peak	4.5	7.1	2.6	58%	0.03719
	EBT	AM Peak	6	7.6	1.6	27%	0.00014
		Off Peak	2	2.6	0.6	30%	0.01896
		Noon Peak	3.5	3	–0.5	–14%	0.20193
		PM Peak	5	4.6	–0.4	–8%	0.14036
	WBT	AM Peak	3.5	4.1	0.6	17%	0.54991
		Off Peak	3	2.5	–0.5	–17%	0.42366
		Noon Peak	3	2.9	–0.1	–3%	0.35105
		PM Peak	7.5	9	1.5	20%	0.04486



Columns: Discrepancies between estimation and field queue length (no. of vehicles).

Curves: Percentage of HCM estimation over field queue length.

▲ : HCM significantly overestimates 50th percentile queue length.

■ : HCM significantly underestimates 50th percentile queue length.

Note: The results for the northbound and southbound lanes at Neil Street and Stadium Drive and Neil Street and St. Mary's Road during the noon peak are not displayed.

Figure 14. 95th percentile queue length comparison between HCM estimates and field measurement.

4.2.3 Results and Discussion

Similar to delay comparison, for the 50th percentile queue length comparison, cases for typical intersections were divided into major and minor streets. The comparison results for typical intersections are summarized in Table 25 and Table 26, and cases with significant discrepancies are plotted for both number of vehicles and percentage in Figure 15. For each table, the column heading "%" stands for either the percentage of cases with a significant discrepancy across the total number of cases or the percentage that the over/underestimation cases occupied in the cases with significant discrepancy. The table cells with "—" signify that the respective data were not applicable and are not presented. The table cells with N/A signify that the respective data were unavailable.

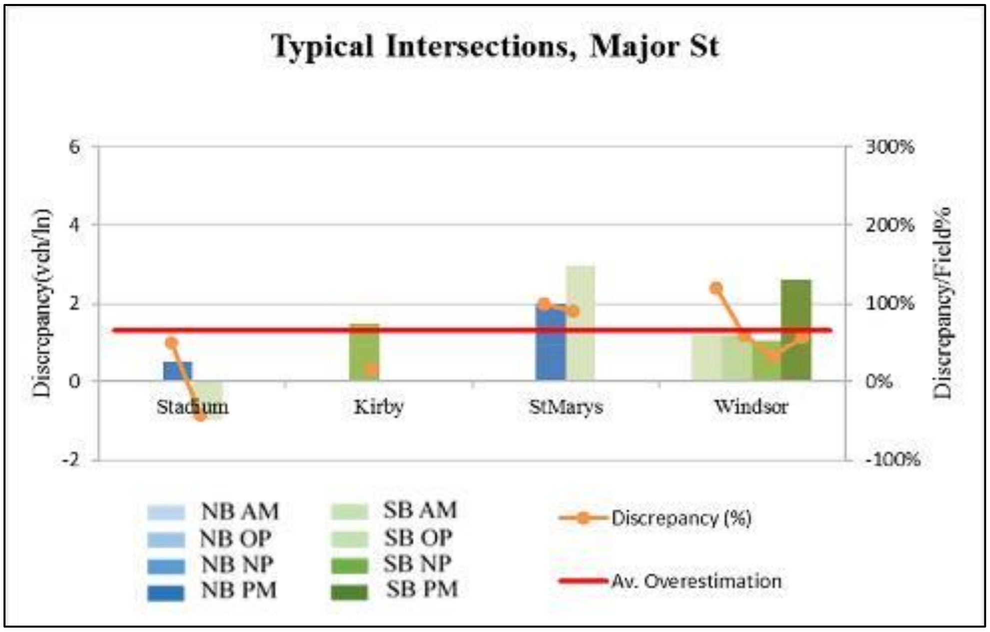
Table 25. Summary of 50th Percentile Queue Length Comparison for Typical Intersections

Overall			
Categories	**No. of Cases**	**%**	**Range of (HCM-Field)/Field %**
Total	52	—	—
Significant Discrepancies	27	52%	(42) – 137%
Significant Overestimation	25	93%	14 – 137%
Significant Underestimation	2	7%	(42) – (20%)

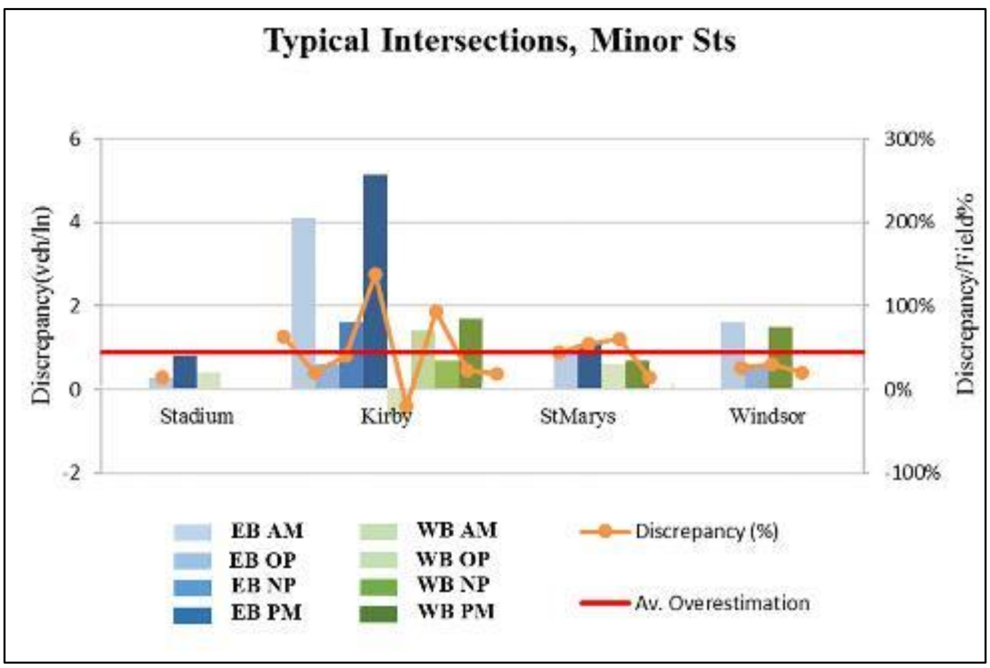
Table 26. Summary of 50th Percentile Queue Length Comparison for Typical Intersections: Major vs. Minor Streets

Major Street (NB/SB)				
Categories	**No. of Cases**	**%**	**Range of (HCM-Field)/Field %**	**Average Discrepancy (%)**
Total	28	—	—	—
Significant Discrepancies	9	32%	(42) - 120%	—
Significant Overestimation	8	89%	17 - 120%	66%
Significant Underestimation	1	11%	(42)%	(42)%
Minor Street (EB/WB)*				
Total	24	—	—	—
Significant Discrepancies	18	75%	(20) - 137%	—
Significant Overestimation	17	94%	14 - 137%	44%
Significant Underestimation	1	6%	(20) %	(20)%

*(HCM – Field)/Field % for eastbound PM and westbound AM at Neil Street and Stadium Drive are unavailable because the field queue lengths of those two cases were zero vehicles per lane.



(a)



(b)

Figure 15. Cases with significant discrepancies for typical intersections in 50th percentile queue length comparison: (a) major street cases, (b) minor street cases.

The following are the findings of the queue length comparison:

- In the 50th percentile queue length comparison, 39 of 64 cases had significant discrepancies in the estimation. In other words, the HCM estimates of median queue length were not accurate in 61% of the cases.
- For the two atypical intersections of Neil Street with Devonshire Drive and Neil Street with Knollwood Drive, HCM significantly underestimated the median queue length in all 12 tests on average by 83% .
- For the four typical four-legged intersections, significant discrepancies were observed in 52% of the cases, of which 93% were overestimations and 7% were underestimations. For the major street, in 32% of the cases, the HCM estimates were significantly different than field data; in 89% of the cases, HCM overestimated the queue length on average by 66%. However, in 11% of them (one case only), field median queue length was underestimated on average by 42%. For minor streets, the HCM queue lengths in 75% of the cases were significantly different than those from the field, and in 94% of those cases, HCM overestimated the queue length on average by 44%. However, in 6% of the cases (one case only), it underestimated queue length on average by 20%.
- The relationship between of the 50th and 95th percentile queue length comparisons can be observed in Figure 14. Among the 26 cases with significant discrepancies in median estimates, the two significant underestimations by HCM in median queue length comparison were still underestimated in the 95th percentile comparison. For the other 24 cases, 14 (58%), showed overestimation in the 50th and 95th comparisons; in ten cases, the 50th percentile queue length showed significant overestimation, but 95th percentile showed underestimation by HCM. The ten discrepancies may be due to the normal distribution assumption HCM makes in its back-of-queue factor calculation, which may not be valid for the queue length data collected in the field. In general, among all the cases with significant discrepancies in median estimates, 16 of 26 cases (67%) in the 95th percentile queue length supported the 50th percentile trend. In other words, in those 16 cases, the over/underestimation trends of HCM were the same for both the 50th and 95th percentile queue lengths.

4.2.4 Comparison Between Red-Time Formula, HCM, and Field Queue Length

Another method practitioners use for queue length estimation is the red-time formula (RTF). RTF is recommended by AASHTO as the conventional rule of thumb for the storage length design of left-turn lanes (Qi et al. 2011). It is based on the assumption that doubling the number of vehicles stored (queued) during the red phase would provide enough length to handle most of the traffic conditions. The storage length in RTF is computed as follows:

$$Storage\ Length\ (ft) = \frac{\left(1-\frac{G}{C}\right)(DHV)(1+\%trucks\ )(25\times 2)}{(\#\ cycles\ per\ hour)(\#\ traffic\ lanes)}$$

In this formula, G is the green time (second) for the studied lane group, and C is the cycle length (second) for the intersection. DHV is the design hourly volume of the studied lanes (veh/h), and in this study, it was the total volume of the through-lane group under consideration. $\%trucks$ is the fraction of the volume that is heavy trucks.

This storage length was considered to be comparable to the 95th percentile queue length values; in the formula, the average storage length is converted to the 95th percentile storage length by a safety parameter of 2. Thus, it was used to estimate the 95th percentile queue length for each through-lane group. For comparison purposes, the unit of computed storage length was changed from feet to vehicles per lane by dividing it by the assumed average vehicle length of 25 ft.

Liu, Benekohal, and Shaik (2016) compared queue length estimates from RTF and HCM to field data at the four typical intersections (using the 95th percentile queue length estimates from HCM and field data). They found that the results varied for different intersections. At Neil Street and Stadium Drive, both of the methods (RTF and HCM) showed estimations close to the field data. At Neil Street and Windsor Road, the two methods showed similar estimations for the northbound, eastbound, and westbound cases. At the intersections of Neil Street with Kirby Avenue and Neil Street with St. Mary's Road, as well as for the southbound cases at Neil Street and Windsor Road, more deviations were observed between the RTF estimates and field queue lengths than in the HCM ones. Considering all four intersections, the HCM 2010 procedure, in general, estimated the 95th percentile queue length better than the RTF.

4.3 RELATIONSHIPS BETWEEN RESULTS OF DELAY AND QUEUE COMPARISONS

4.3.1 Methodology

After comparing the estimated delay and queue length from HCM to the corresponding field delay and queue length, further analyses were conducted to explore the consistency in the results of delay and queue comparisons (to see whether the trend of over/underestimation by HCM is the same for both delay and queue).

In this section, the results of stopped delay comparisons and 50th percentile queue length comparisons are analyzed. All 64 cases in the 50th percentile queue length comparison were among the 84 cases in the stopped delay comparison. Thus, in those 64 cases, the relationships between delay and queue comparison results were further analyzed.

4.3.2 Results and Discussion

Table 27 summarized the study results. All 64 cases were grouped into one of two categories: cases consistent in over/underestimation trend, and cases inconsistent in over/underestimation trend. For each category, the cases were further classified into three subcategories based on the significant level of discrepancies in queue and delay comparisons. Those include cases with significant discrepancies in *both* queue and delay comparisons, cases with only one significant discrepancy in *either* queue or delay comparison, and cases with *no* significant discrepancies in both queue and delay comparisons. For each table, the column heading "%" stands for either the percentage of cases in the subject category over the total, or the percentage of cases in the subcategory over the total in the subject category. The table cells with "–" signify that the respective data were not applicable and are not presented.

Table 27. Summary of Relationships Between Results of Delay and Queue Comparison

Overall		
	No. of Cases	**%**
Total	64	—
Consistent trend	50	78%
Inconsistent trend	14	22%
Category 1: Cases with Consistent Trend		
Total	50	—
Significant discrepancies both in queue and delay	27	54%
One significant discrepancy (either queue or delay)	19	38%
No significant discrepancies in queue or delay	4	8%
Category 2: Cases with Inconsistent Trend		
Total	14	—
Significant discrepancies both in queue and delay	0	0%
One significant discrepancy (either queue or delay)	6	43%
No significant discrepancies in queue or delay	8	57%

As shown in Table 27, in 50 of the overall 64 cases (78%), the trends of over/underestimation by HCM were the same for delay and queue length (i.e., if HCM overestimated the field delay, it also overestimated the field queue length, and vice versa). Among those cases, 46 (92%) had at least one significant discrepancy in delay or queue length comparisons. Among the 46 cases, 27 (54%) had significant discrepancies in both delay and queue length, and 19 (38%) had only one significant discrepancy. For only four of the 50 cases (8%), the HCM estimates were not significantly different from both field delay and field queue length.

On the other hand, for 14 of 64 cases (22%), the comparisons of delay and queue lengths were not consistent with each other in the over/underestimation trend. In six of 14 cases (43%), there was only one significant discrepancy in either delay or queue comparison. Those six cases belonged to the major street cases. Four of them showed significant overestimations in delay comparison but no significance observed in the queue comparison, and they were the northbound cases during the noon peak and the off-peak periods at the intersections of Neil Street with Stadium Drive, Kirby Avenue, and Windsor Road. For the other two cases, significant underestimations were observed in queue comparison but no significant differences were seen in delay comparison. Both underestimations were at Neil Street and Knollwood Drive, one for northbound during the AM peak and the other southbound during the PM peak. In the remaining eight cases (57%), there were no significant discrepancies in either delay or queue length comparisons. This finding indicates that the conflict of the over/underestimation trend in the latter eight cases was not significant enough to represent the inconsistency between the results of delay and queue comparisons.

In summary, in 58 of 64 cases (91%), the HCM's over/underestimation of delay and queue length was consistent or there was no significantly conflict between them. However, in six of the 64 cases (9%), there were significant inconsistencies between the delay comparisons and queue length comparisons.

CHAPTER 5: CONCLUSIONS

This report presents the methodology and outcome of data collection, data reduction, and data analysis of the field conditions before implementation of the SynchroGreen adaptive signal control technology system on Neil Street in Champaign, Illinois. One day per intersection was selected to reduce the field videos and obtain traffic characteristics of interest for four different time periods: AM peak, off peak, noon peak, and PM peak. Those traffic characteristics were peak hours, hourly volume, saturation flow rate, signal timing, arrival type, field delay, and queue length.

The field delay and queue length measured in the “before” conditions will be used to evaluate the operational performance of the SynchroGreen system by comparing them with “after” conditions. Those measures of effectiveness in the “before” conditions were also compared with the HCM estimations to quantify the effects of volume changes and additional developments at Neil Street and Devonshire Drive throughout the course of the study.

The HCM estimates of stopped delay were significantly inaccurate in 49 of 84 cases (58.3%), representing overestimation in 73.5% of the cases and underestimation in 26.5%. For typical intersections on the major street, 72% of the cases had significant discrepancies between HCM delay estimates and field data—in 91% of the cases, HCM overestimated delay by an average by 69%. On minor streets, 56% of the cases had significant discrepancies, and, in 94% of them, HCM overestimated the delay on average by 52%.

For the 50th percentile queue length, HCM estimates were significantly inaccurate in 39 of 64 cases (61%), of which 56% were overestimations and 44% were underestimations. For typical intersections, 52% of the cases had significant discrepancies, including 93% overestimations and 7% underestimations. For the major street at typical intersections, in 68% of the cases, the 50th percentile HCM queue lengths were similar to those from the field. However, in 28% of the cases, HCM overestimated the median queue length on average by 66%, and in 4% of the cases, it underestimated the median queue length on average by 42%. For minor streets, in only 25% of the cases were the median HCM queue lengths similar to those from the field. In 70% of the cases, HCM overestimated the median queue length on average by 44%, and in 5% of the cases, it underestimated it on average by 20%.

In addition, a 95th percentile queue length comparison was conducted between HCM estimates and field data. In general, it was observed that the trends in 50th and 95th percentile queue length comparisons supported each other.

The results of the delay comparison and 50th percentile queue length comparison for the 64 overlapping cases were compared. In 91% of the cases, the trends in delay and queue comparisons were consistent with each other or had no significant conflicts; however, in 9% of the cases, significant inconsistencies were observed. Thus, to save time, one may compare the HCM queue length estimates to field data to assess intersection performance, although the delay comparison is preferred.

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APPENDIX

The signal timing data used in performing the capacity analysis in HCS are presented in this appendix. Those data include the average green intervals, yellow change, and red clearance at each intersection for the four time periods. Columns 1 through 8 in the Tables A-1 through A-3 correspond to the phases shown in Figure A-1. For reference, the phase in column 1 is a north- and southbound protected left turn.

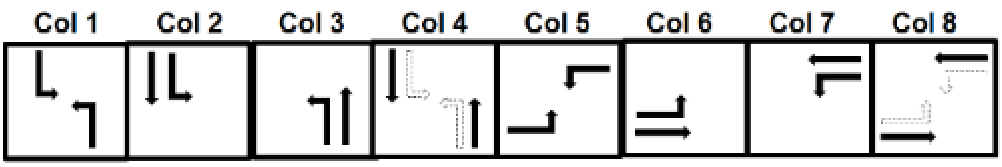

Figure A-1. Signal phases observed in the field.

A.1 GREEN INTERVALS

Table A-1 presents the representative green intervals (reduced from field videos) that were used to perform the capacity analysis in HCS. Columns 1 through 8 represent the green intervals of the corresponding phases as described in Figure A-1. All values are in seconds.

Table A-1. Representative Green Intervals Observed in Field and Used in Capacity Analysis

	Neil Street at	Col 1	Col 2	Col 3	Col 4	Col 5	Col 6	Col 7	Col 8	*Cycle Length*
	Stadium Dr				27.2				16	55
	Kirby Ave	8	2		36	8.7			35	110
AM Peak	St Mary Rd		9.2		58	8			13	110
	Devonshire Dr				88				10.9	110
	Knollwood Dr				89				9.4	110
	Windsor Rd	9			41.9	8	5		22	110
	Stadium Dr				29				14.2	55
	Kirby Ave	9			35.7	8			35	110
Off Peak	St Mary Rd		8		57.7	8			14.5	110
	Devonshire Dr				86.9				12	110
	Knollwood Dr				89				9.4	110
	Windsor Rd	8.1			42	9			28	110
	Stadium Dr				28.2				20	60
	Kirby Ave	8		2	47.5	7		2	32	120
Noon Peak	St Mary Rd				74	8.2		2	18	120
	Devonshire Dr				97				11.9	120
	Knollwood Dr				96				12.4	120
	Windsor Rd	8	2		45.9	9		2	31	120
	Stadium Dr				33				10.2	55
	Kirby Ave	8			36.7	8			35	110
PM Peak	St Mary Rd				75	7.4			11	110
	Devonshire Dr				86.9				12	110
	Knollwood Dr				89.4				9	110
	Windsor Rd	8			42.1	8		2	29	110

A.2 YELLOW CHANGE

Table A-2 presents the yellow change data (obtained from the signal controller settings of all intersections) that were used to perform the capacity analysis in HCS. Columns 1 through 8 represent the yellow change of the corresponding phases as described in Figure A-1. All values are in seconds.

Table A-2. Yellow Change Used in Capacity Analysis

	Neil Street at	Col 1	Col2	Col 3	Col 4	Col 5	Col 6	Col 7	Col 8
AM Peak	Stadium Dr				3.6				3.2
	Kirby Ave	3.2	0		3.6	3.2			3.6
	St Mary Rd		3.2		3.6	3.2			3.2
	Devonshire Dr				3.9				3.6
	Knollwood Dr				3.9				3.2
	Windsor Rd	3.2			3.9	3.2	3.2		3.6
Off Peak	Stadium Dr				3.6				3.2
	Kirby Ave	3.2			3.6	3.2			3.6
	St Mary Rd		3.2		3.6	3.2			3.2
	Devonshire Dr				3.9				3.6
	Knollwood Dr				3.9				3.2
	Windsor Rd	3.2			3.9	3.2			3.6
Noon Peak	Stadium Dr				3.6				3.2
	Kirby Ave	3.2		3.2	3.6	3.2			3.6
	St Mary Rd				3.6	3.2		3.2	3.2
	Devonshire Dr				3.9				3.6
	Knollwood Dr				3.9				3.2
	Windsor Rd	3.2	3.2		3.9	3.2			3.6
PM Peak	Stadium Dr				3.6				3.2
	Kirby Ave	3.2			3.6	3.2			3.6
	St Mary Rd				3.6	3.2		0	3.2
	Devonshire Dr				3.9				3.6
	Knollwood Dr				3.9				3.2
	Windsor Rd	3.2	0		3.9	3.2			3.6

A.3 RED CLEARANCE

Table A-3 presents the red clearance data (obtained from the signal controller settings of all intersections) that were used to perform the capacity analysis in HCS. Columns 1 through 8 represent the red clearance of the corresponding phases as described in Figure A-1. All values are in seconds.

Table A-3. Red Clearance Used in Capacity Analysis

	Neil Street at	Col 1	Col2	Col 3	Col 4	Col 5	Col 6	Col 7	Col 8
AM Peak	Stadium Dr				2.5				2.5
	Kirby Ave	0	0		2.4	2			2.3
	St Mary Rd		2		2.1	2			2.5
	Devonshire Dr				1.5				2.1
	Knollwood Dr				2				2.5
	Windsor Rd	2			2.5	0	0		2.5
Off Peak	Stadium Dr				2.5				2.5
	Kirby Ave	2			2.4	2			2.3
	St Mary Rd		2		2.1	2			2.5
	Devonshire Dr				1.5				2.1
	Knollwood Dr				2				2.5
	Windsor Rd	2			2.5	2			2.5
Noon Peak	Stadium Dr				2.5				2.5
	Kirby Ave				2.4				2.3
	St Mary Rd				2.1				2.5
	Devonshire Dr				1.5				2.1
	Knollwood Dr				2				2.5
	Windsor Rd	0			2.5	0			2.5
PM Peak	Stadium Dr				2.5				2.5
	Kirby Ave	2			2.4	2			2.3
	St Mary Rd				2.1	2			2.5
	Devonshire Dr				1.5				2.1
	Knollwood Dr				2				2.5
	Windsor Rd	2			2.5	0	0		2.5

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