DISCHARGE HEADWAY AT SIGNALIZED

INTERSECTIONS IN MONTERREY,

NUEVO LEON, MEXICO

by

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Presented to the Faculty of the Graduate School of

The University of Texas at Arlington in Partial Fulfillment

of the requirements

for the Degree of

DOCTOR OF PHILOSOPHY

THE UNIVERSITY OF TEXAS AT ARLINGTON

May 2006

To: my Mom, Dad (+) and brothers; my beloved wife Silvia Catalina; my children David, María and Roberto de Jesús, marvelous gifts; and the only one that makes everything possible, Jesus, my Lord.

ACKNOWLEDGMENTS

November 11, 2005

ABSTRACT

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Publication No. _____

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The University of Texas at Arlington, 2006

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Driver behavior has been the subject of study for almost 100 years. As a result, driver characteristics have been determined and used in the design of highway elements, both for interrupted and for free flow facilities. Signalized intersections represent the most typical interrupted flow facility, with driver reaction time and aggressiveness playing a key role in signal timing design and also in the assessing of intersection performance. These driver characteristics

were first determined in the United States in the late 1940's, identifying an average headway (in seconds) between consecutive vehicles as they react to light change from red to green and pass a common reference point. The additional time required to start the queue moving, known as the start-up lost time in seconds, was also identified.

After many studies of these characteristics over time and despite the wide variability of the average headway as a result of varying conditions from site to site, it has been possible to adopt a specific value of average headway for conditions defined as base conditions. As a result, adjustment factors have been applied since the site conditions differ from the base ones. The vast majority of such studies have been conducted for United States conditions; however, it was worthwhile to determine these parameters for other conditions, specifically for Monterrey, México, and also to evaluate the effect of time of day and lane use variables in the subject approach of three different signalized intersections.

The general result of the study was that average headway for Monterrey conditions is slightly higher to that of United States conditions. However only two or three vehicles incur start-up lost time, instead of five in the United States. Also, an important difference resulted in comparing two different procedures for headway calculation.

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CHAPTER 1

INTRODUCTION

This chapter consists of four sections. The first one is a background that briefly touches on what could be viewed as the farthest origin of what is known today as the traffic problem, then providing a quick history of automobile industry, and finishing up with a few statistics relevant to a clear identification of this traffic problem.

In the second section, traffic engineering is defined, and how its focus has evolved from the study of traffic crashes and the way to avoid them, to the current concern on mobility, natural resources and the environment. The section ends commenting on the role of computational tools for the quick and precise solution of the various models built to describe the physical processes taking place amongst the driver, the vehicle and the roadway.

The third section presents the starting point of this study and how it shifted to the objective of determining a driver parameter for a Mexican setting, in particular for the city of Monterrey, Nuevo León. Finally, the fourth section outlines how this written report is organized.

1.1   Background

Man was surely far away from guessing the serious trouble his wheel discovery, pooled with other inventions taking place through time, would bring about around 60 centuries later. Of course, both health and elusive life value put aside, the benefits of such a discovery have been far more important and transcendent to human advancement.

The invention of the steam engine in the early XVIII century allowed for a number of trials with little success, of applying this source of power to self-propelled road vehicles. French engineer Nicolas J. Cougnot successfully invented the first automobile using a steam engine in 1770 and by 1803, U. S. inventor Oliver Evans built the first self-propelled vehicle that traveled through the United States roadways.

Over the next two decades or so, even though improvement in steam engines continued to the extent of using them to regular daily movement of passenger coaches in England, restrictive British legislation forced the steam coaches off the roads. By 1860, development of self-propelled steam engine vehicles ceased. In the meantime, other countries like France and Germany turned their attention to the internal-combustion engine. The first internal-combustion engine was designed to be fueled with gunpowder but it was never built. By 1894, continuous improvement of this type of engine fueled with gasoline, resulted in a clear superiority over the then highly developed steam

engine, as it turned out at the famous Paris-Bordeaux race of 1895, where the first four cars arriving the finish line had high-speed Daimler gasoline engines.

A number of US pioneer automobile manufacturers were very active in the 1890s, with Henry Ford producing his experimental model in 1893. In the early 1910 decade, over 600,000 automobiles were circulating all over US roads, some powered by steam, some by gasoline and still others by electricity but almost all of them were touring-type. Before that time motoring had been regarded primarily as a sport; afterwards, it was increasingly considered a means of transportation.

So, just like around 60 centuries ago when man did not guess the problems his wheel invention would bring about, the first auto makers from the late XIX and early XX centuries most likely neither foresaw what today, at the onset of the XXI century, we continue referring to as the *traffic problem***.**

Its daily consequences are endured now for at least 10 decades in terms of crashes, delays and, more recently, pollution. The ever-growing consumption of a nonrenewable natural resource like oil, out of which gasoline and diesel, among other fuels are obtained, is also a concern.

But, how did this *traffic problem* start? At the beginning of the 20th century the number of vehicles was few and their relatively low speeds (barely above the animal-drawn ones') allowed that century's first decade to go by without major concern. Things were rather different in the following decades,

though. From the outset, the mileage of existing roads was not enough for the number of vehicles that was rapidly growing year after year, and the conditions of such roads did not match with operating features of the fast automotive vehicle. As auto-makers incorporated higher speeds to their vehicles and the overall quantity of autos skyrocketed, consequences immediately appeared in the form of pitiable crashes and saturated roadways that were not built for such vehicles.

As it is logical to think, the congestion or saturation problem was more noticeable within towns, just as it continues to be nowadays since the vehicle accumulation is greater in an urban setting. On the other hand, it was not easy to widen and improve precarious roadways due to existing development and to the high cost of necessary land.

By the same token, safety became a problem in rural settings due to the fact that vehicles with their continually higher speeds traveled along the inadequate roads of that time causing a lot of crashes. As a consequence, they resulted in fatalities, numerous injured people and property damage.

In order to gain a better understanding of the traffic problem and its mentioned consequences, some statistics are now introduced. Early records show four motor vehicles registered in the United States in 1895 (ITE 1965). Table 1 on next page gathers motor vehicle registration (all types but motorcycles), highway fatalities, public road miles, and fuel consumption between years 1900 and 2000 in 10-year periods (USDOT 2004).

Table 1.1-- Some statistics about the traffic problem in the USA

Year	Vehicles x 1,000	Fatalities	Highwy miles x 1,000,000	Fuel gallons x 1,000,000
1900	8	36	2,320	
1910	469	1,599	2,430	
1920	9,239	12,155	3,105	3,448
1930	26,750	31,204	3,259	15,778
1940	32,453	32,914	3,287	23,969
1950	49,162	33,186	3,313	39,839
1960	73,858	36,399	3,546	63,212
1970	108,418	53,816	3,730	96,332
1980	155,796	51,091	3,860	118,614
1990	188,798	44,599	3,867	134,831
2000	221,475	41,821	3,936	165,232

1950 year based indices

Year	Vehicles x 1,000	Fatalities	Highwy miles x 1,000,000	Fuel gallons x 1,000,000
1900	0.00	0.00	0.70	0.00
1910	0.01	0.37	0.94	0.00
1920	0.19	0.37	0.94	0.09
1930	0.54	0.94	0.98	0.40
1940	0.66	0.99	0.99	0.60
1950	1.00	1.00	1.00	1.00
1960	1.50	1.10	1.07	1.59
1970	2.21	1.62	1.13	2.42
1980	3.17	1.54	1.17	2.98
1990	3.84	1.34	1.17	3.38
2000	4.51	1.26	1.19	4.15

Based on information in Table 1.1, Figure 1.1 shows how these indicator indices have evolved from years 1900 to 2000, adopting year 1950 as a base.
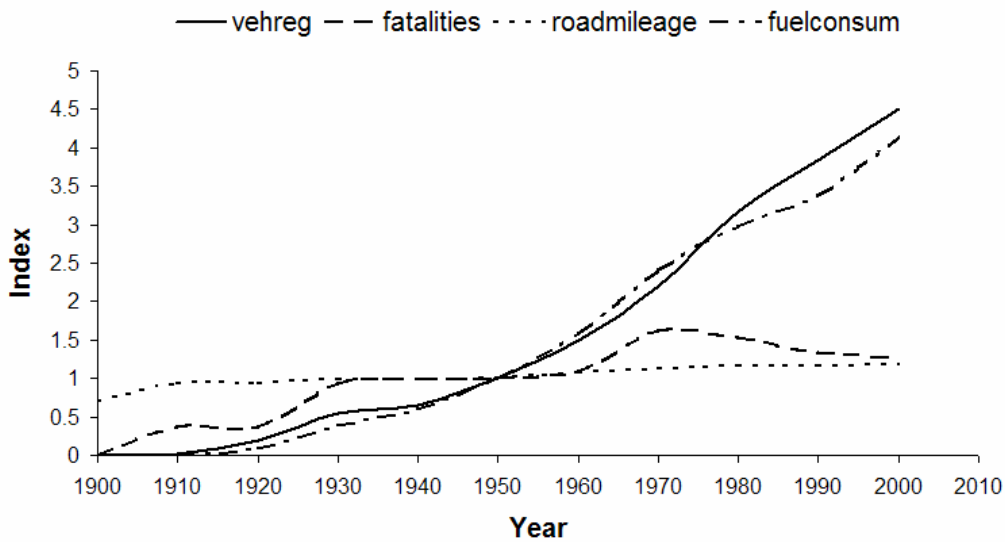
Figure 1.1 Growth rate of traffic problem indicators in the US.

A big difference in the growth rate between the number of vehicles and the public road miles can be seen, and an almost similar growth rate between the number of vehicles and the motor-fuel consumption is also evident. As for the number of highway fatalities, a generalized lower tendency has been registered after its historical maximum of 55,600 in 1972.

Though most of the attention is given to the quantity by itself, it is also common in speaking of highway fatalities to present them as rates related to some variables. Highest fatality rate per 1,000 miles of road, 14.84 was reached in year 1969 and lowest was 0.02 in 1900 and 1901; as for the rate per 100 million annual vehicle miles of travel (VMT), the highest value was 45.33 in 1909 with the

lowest of 1.72 in 1995; the last rate available in the same 95 year span is per 100,000 registered motor vehicles, with its highest 450.00 value in 1900 and its lowest 20.61 in 1992 (USDOT 2004).

Highway fatality rate per 1,000 licensed drivers began to be recorded by this source in year 1949, with a maximum value of 54.79 in year 1951 and a minimum of 22.66 in year 1992 (dot.gov@ 2004).   The October 2003 Transportation Statistics Annual Report (USDOT 2003) presents some other figures on fatalities and pollution.

> There were 45,130 fatalities related to transportation in 2001, almost 16 fatalities per 100,000 U.S. residents. This is a decline of 11% from 18 fatalities per 100,000 residents in 1991, when there were 44,320 fatalities. Nearly 93% of all transportation fatalities in 2001 were highway-related. Most of these people who died were occupants of passenger cars or light trucks (including pickups, sport utility vehicles, and minivans). Air, rail, transit, water, and pipeline transportation resulted in comparatively few deaths per capita. Transit, for instance, led to about 0.11 deaths per 100,000 residents in 2001.

> Transportation in 2001 emitted 66% of the nation's carbon monoxide (CO), 47% of nitrogen oxides (NO_x), 35% of volatile organic compounds (VOC), 5% of particulates, 6% of ammonia, and 4% of sulfur dioxide. Highway vehicles emitted almost all of transportation's share of CO in 2001, 80% of the NO_x, and 75% of all VOC.

Since this work uses field data from Monterrey, México, a few numbers of highway transportation history for this country and city are presented. In 1898, the first vehicle that entered Mexican territory was registered in Monterrey though did not remain in the city, but was taken to Guadalajara, the capital city of the State of Jalisco. During that time, the Nuevo León State railroad network

satisfied most of the necessities of moving goods and products required in Monterrey. The most important track then was the México-Laredo route (Ferrocarriles y Vialidad 2004).

By 1925, the movement of people and goods among cities was done through railroad because highways had not been built yet. In fact, the first highway in the country was to connect México city with the city of Puebla, inaugurated in September 1926. The first traffic jam took place during the event due to the presence of 2,000 vehicles. The national highway network (both paved and unpaved) was developed from 1930 to 1973 as Table 1.2 shows ("Asociación Mexicana de Caminos" 1974). The network reached 151,560 miles in 1992 and 211,595 miles by year 2001 (Secretaría de Comunicaciones y Transportes 2004).

Table 1.2-- National highway evolution in Mexico

Year	Mileage of paved and unpaved roads
1930	890
1940	6,170
1950	13,310
1960	26,070
1969	42,030
1979	96,130

Regarding Monterrey, Nuevo León, Table 1.3 shows the evolution of its total metropolitan population, total urban area, and amount of this area devoted to development and to road network from 1950 to 2000.

Table 1.3-- Population and area evolution in Monterrey, Mexico

Year	Pop.	Total area, acres	Block area, acres	Roadway area, acres
1950	389,629	20,109	15,204	4,905
1960	723,739	21,091	15,980	5,111
1970	1;254,691	40,330	29,973	10,357
1980	2;011,936	47,443	35,137	12,306
1990	2;988,081	60,797	46,133	14,664
2000	3;243,466	109,250	106,191	33,611

Table 1.4 contains vehicle registration (all types but motorcycles) and growth percentage from 1994 to 2001. Total trips in the metro area during 1993 amounted to +4.7 million, with 60% by transit, 35% by car and 5% by other means; for year 2002, total trips grew to more than 6 million with almost same distribution between the modes. It is interesting to note in Table 1.4, that the average trip time grew from 33.2 minutes in 1993 to almost 39.9 minutes in 1999 (ITESM 2003).

1.2   Traffic Engineering

Information gathered from the answer given by men at the beginning of the past century was the basis for what is known today as Traffic Engineering. The beginning of Traffic Engineering is sumarized by Radelat (2003):

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Table 1.4-- Vehicle registration growth in Monterrey, México

Year	Vehicles	Growth, %
1994	567,920	-
1995	575,101	1.26
1996	596,076	3.65
1997	629,294	5.57
1998	676,452	7.49
1999	732,493	8.28
2000	825,051	12.64
2001	970,186	17.59

The problem needed a technical approach, not only starting from studies about the movement of vehicles on the roadway, but also about the direct originators of such movements that happened to be the vehicle drivers themselves. It was necessary to create a real technique of circulation that simultaneously accounted for physical laws and human factors; and such a technique started to take form in the country where traffic problems were acquiring bigger dimensions, that is to say the United States.

A little bit later than year 1920 when the amount of automotive vehicles in that country grew very fast, a knowledge branch started to gestate, a professional specialty within the civil engineering field guided towards traffic. In 1930, the new profession was completely defined and its members launched the Institute of Traffic Engineers. And, just what was the role of these brand new traffic engineers?

At the beginning, their activity was mainly focused to the study of traffic crashes and the way to avoid them, but gradually its field stretched trying to address the numerous factors composing the complex traffic phenomenon. At the same time, purely empirical practices were giving way to more rational procedures, to the extent that nowadays it can be said that a technological profession known as traffic engineering is definitely established.

However, in regards to the above time frame of such a specialty, it is worth mentioning that a number of traffic engineering practices had been applied some centuries before (Pignataro 1975):

> ... many of the profession's antecedents are rooted in ancient history. For example, one-way streets were known in ancient Rome, and special off-street parking facilities were provided to get chariots off the traveled way. Vehicles were prohibited from entering the business districts of large cities in the Roman Empire during certain hours of the day because of traffic congestion. It is most likely that similar traffic rules and regulations were necessary to control vehicular flow on the paved streets of Babylon in 2000 B.C. Modern traffic islands and rotaries have their origins in the monuments and public squares erected in roadways of centuries past. Pavement markings were used as early as 1600 A.D. on a road, beginning in México City, which had incorporated a built-in centerline of contrasting color.

Traffic engineering today has to do with the modern problems of safety, mobility, natural resources and the environment, making a combined use of specific knowledge areas such as traffic characteristics, geometric design, traffic studies, capacity and level of service analysis, traffic control, and traffic operations and management.

One of the traffic engineer's tasks is assessing the performance of the roadway network. In view of traffic flow, two types of roadway network facilities are defined: interrupted, which is the surface street system with intersections and connections to and from abutting properties; and uninterrupted, the freeway system where no stops occur due to intersecting roadways or connections.

The use of computational tools by the traffic engineer in the performance assessment has an important role. It solves the various models built to describe the physical processes taking place among the driver, the vehicle and the roadway network, either along interrupted, uninterrupted or the combination of both traffic flow roadway facilities.

Computer simulation models play a major role in the analysis and assessment of the highway transportation system and its components. These components may be individual signalized or unsignalized intersections, residential or central business district dense networks, linear or network signal systems, linear or corridor systems, or rural two-lane or multilane highways systems. Out of these components, this study is related to individual signalized intersections.

1.3   Study objective

The very first idea of this research was to compare the outcomes of traffic simulation software when used to model driver behavior in an American setting versus a driver in a Mexican one. The assumption was made that there must be an important or significant difference in simulation results, particularly derived from the fact of the apparent diversity within this specific location condition, as reflected or represented by the driver characteristics. TEXAS (Traffic Experimental Analytical Simulation) and CORSIM (CORridor SIMulation) models were selected as reference points.

TEXAS is a microscopic model developed by the Center for Highway Research (now Center for Transportation Research) of The University of Texas at Austin during the 1970's. It can be used as a tool by transportation engineers to evaluate traffic performance at isolated intersections operating under various types of intersection control. In this model, the driver characteristics are (1) the driver operational factor (85 for a slow driver, 100 for an average driver, and 110 for an aggressive driver) and (2) the perception-reaction time in seconds (1.5, 1.0, and 0.5 respectively in the same order).

CORSIM is an integrated set of two microscopic simulation models, NETSIM and FRESIM. The acronym NETSIM is derived from NETwork SIMulation representing surface-street, interrupted traffic, while FRESIM stands for FREeway SIMulation and represents freeway, uninterrupted traffic. There are 11 driver behavior parameters that define the driver type in NETSIM:

1. queue discharge headway and start-up lost time
2. distribution of free flow speed by driver type
3. mean duration of parking maneuvers
4. lane change parameters
5. maximum left and right turning speeds
6. probability of joining spillback
7. probability of left turn jumpers and laggers
8. gap acceptance at stop signs
9. gap acceptance for left and right turns
10. pedestrian delays, and
11. driver familiarity with their paths

From the previous brief description of each model, it is evident that driver type definition is more complete in CORSIM than in TEXAS. Therefore, at a certain point it was decided to work only with that model. Considering the parameters mentioned above, the first one which is typical of a signalized intersection operation, queue discharge headway and start-up lost time was chosen to be considered (they are actually two but the model refers to them as one record type).

As the first attempts were made to work with CORSIM, it was noticed that a small unsound amount of documented information on this driver parameter for a Mexican setting, specifically for the Monterrey metropolitan area, was available. Therefore, the objective of this study moved to try to determine and for some signalized intersections in the Monterrey metropolitan area.

1.4   Report organization

There are six chapters and one appendix in this dissertation: Introduction, Literature Review, Experimental Design, Field Data Collection, Analysis, and Conclusions and Recommendations. As a result of videotaped and subsequently processed field data from three signalized intersections, tables with time readings and individual headway (a concept to be defined in chapter two) values are included as Appendix A, at the end of this document.

Some pertinent definitions are presented at the beginning of the next chapter, then reviewing several of the references available on the subject, and ending with a brief comment on the importance of start-up lost time, the major component of total lost time (also to be defined in the first section of chapter two) in traffic signal timing.

CHAPTER 2

LITERATURE REVIEW

As mentioned in the introduction, the theme of comparing simulation results between USA driver characteristics and México driver characteristics (or, between a driver parameter for USA and México settings) was the initial goal of this research. Out of 11 driver parameters that define driver type within the CORSIM model, the queue discharge headway *(qdh)* and the start-up lost time *(sult)* were chosen to originally compare simulation results and to be studied.

At the very start of reviewing the literature, it became clear that in México little or no research had been conducted on the subject; therefore, the original motivation of the work would not in fact have been possible to be accomplished because of the lack of this input information. This is why the objective changed instead to try to determine such a parameter for Mexican conditions, and to assess how factors may affect its value.

There are three sections in this chapter. Section one offers a series of pertinent definitions so as to clarify and become familiar with the study objective. Then section two is specifically dedicated to review of literature on the subject, i.e., it touches on what has been done, to end in the third section dealing with the importance of *sult* in the process of traffic signal timing.

2.1   Some definitions

In regard to an isolated signalized intersection, i.e., one that is not part of a coordinated system along an arterial or within a network, cycle length (C) is defined as the time required for one complete sequence of signal indications green, plus yellow and red times (in some cases, all-red time as well); with phase being defined as the part of a cycle allocated to any combination of one or more non-conflictive traffic movements simultaneously receiving the right of way by means of the green indication, while conflictive movements are stopped by the red indication on the signal.

Thus, the subject parameter has to do with the process that takes place every time a line of stopped vehicles demanding exclusive time to go through the intersection, receives the change indication from red to green. Figure 2.1 presents vehicles waiting for the green light to enter in a signalized ntersection.
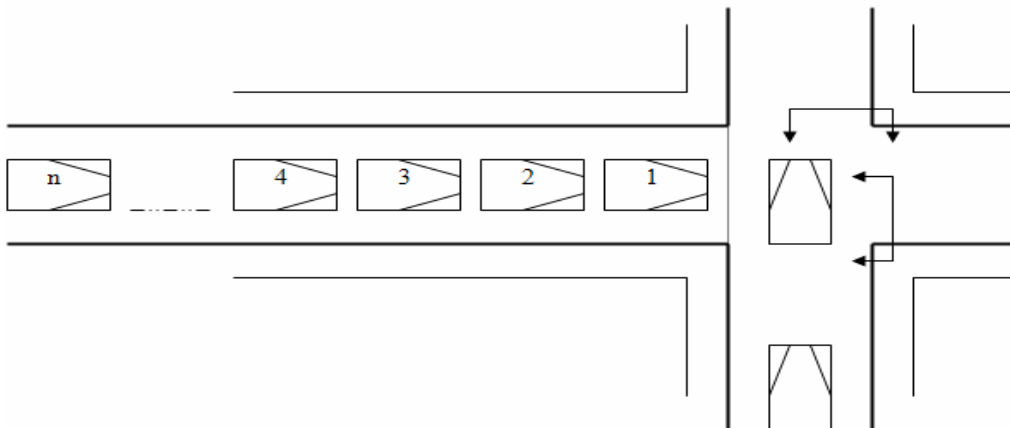
Figure 2.1 Queued vehicles at a signalized intersection.

The point is that this cycle-to-cycle process involves a stop and go condition for a line of vehicles arriving at the intersection. The first waiting driver will usually take some time to react to the red-to-green change before releasing the brake and starting acceleration. Subsequent drivers will usually also incur some reaction time, which will be shorter with every subsequent driver in the line, as it approaches zero.

Once there is no more reaction time implicit and provided that queue is long enough to fully use the allocated green time, a fairly constant or average headway technically known as the saturation headway (h), is reached. Saturation headway in turn leads to the term saturation flow rate (s), meaning the number of vehicles per hour that could enter the intersection, should every vehicle in a stable moving platoon consume h seconds and should its corresponding traffic signal indicate always green, that is to say $s = 3600/h$.

Sult refers to an unused time incurred by first drivers in the waiting line at light change from red to green. There is also a small portion of time not used at the end of the green light known as clearance lost time (*clt*), which is the time between the last vehicle entering the intersection in a fully-used green phase, and the initiation of the green on the next phase. Adding together *clt* and *sult* makes up the total lost time (*tlt*) that occurs at every signal phase, which along with the saturation flow rate represent critical inputs into the signal timing computation process, as discussed at the end of this chapter.

Previous research indicates that *h,* and consequently *s* vary widely depending on prevailing conditions for the movement being studied in the intersection. Lane widths, approach grades, presence of heavy vehicles, parking conditions, numbers of turning vehicles and conflicting pedestrians and vehicular flows significantly affect *h*. The summation of the extra time consumed by the first few drivers beyond *h* is known as *sult*; all of this is shown in Figure 2.2.
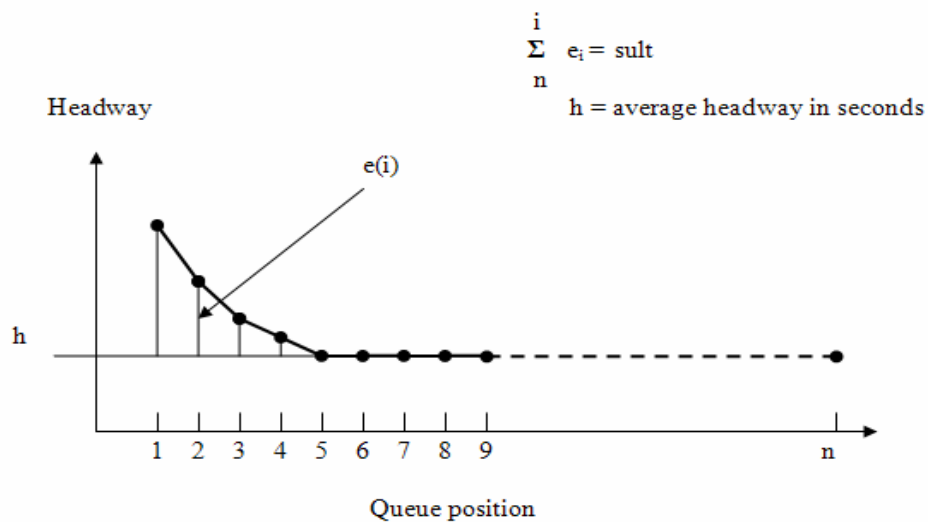
Figure 2.2 Queue discharge at a signalized intersection.

2.2   Literature review

The first work on measuring lost times and saturation headways was done by Bruce D. Greenshields in the mid-40's (Roess 1998). Available recording devices during that time demanded a several step process from field observation to final data transcription. Working with field data (2,359 observations from a

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signalized intersection in Hartford, Connecticut) that included left turn movements and heavy vehicles, and based on supplemental studies of equivalencies, his final findings refer to through movements and passenger cars only.

Mr. Greenshields demonstrated that headway between vehicles decreases at a lessening rate until after the fifth waiting vehicle enters the intersection, and that afterwards it tends to level out around 2.1 seconds; he also found that the extra time beyond these 2.1 seconds for the first 5 vehicles amounted to 3.7 seconds (Greenshields 1947).

In the process of preparing the 2nd edition of the Highway Capacity Manual, data from 1,100 signalized intersections collected during 1955 and 1956 it was found that a line of vehicles stopped by a traffic signal would rarely move away from the intersection at a rate greater than 1,500 vehicles per hour of green per lane, or vphgpl ($h = 2.4$ seconds). On the other hand, provided that the subject intersection is part of a perfectly-coordinated progressive signal system, then a capacity flow rate of 2,000 vphgpl ($h = 1.8$ seconds) might be achieved. Also, maximum capacity for a separate left-turn lane was found to be 1,200 vph ($h = 3.0$ seconds) of green (HRB 1965).

The main conclusions of a study conducted in the 1970's indicate that almost 30 years after Greenshields' findings, more aggressive driving habits and better acceleration performance of vehicles (standard vs. automatic transmissions)

had resulted in a start-up lost time considerably lower, 1.1 seconds instead of 3.7 seconds, nevertheless reporting the same value of 2.1 seconds for h (Kunzman 1978).

Left-turn and through lane capacity was another issue in this investigation, which found that by considering intersection width instead of number of lanes in the analysis procedure, the 1965 edition of the HCM underestimates capacities from 13 to 44% in those lanes. Finally, worth mentioning is the simplicity of the methodology used in this study, since it only required from the author to use a stop watch to collect data while merely traveling for about a month through various locations (Kunzman 1978).

As a result of the continuing evolution of both driving habits and vehicle performance, values for parameter h have shown a tendency to decrease, meaning higher saturation flow rates. While the 1985 edition of the HCM recommended an h value of 2.0 seconds or a saturation flow rate of 1,800 vphgpl, an update of the same manual in 1994 revised the h value to approximately 1.9 seconds or a saturation flow rate of 1,900 vphgpl; this is the recommended h value to use in 2000 edition of the HCM (TRB 2000).

The question of when a vehicle has either entered the intersection or been discharged from the queue was once investigated and documented at the beginning of the 1990 decade. This topic involves reference points both for the vehicle (front bumpers, front wheels, rear bumpers, rear wheels) and for the

roadway (stop bar, near-side curb line, far-side curb line), when reading times between vehicles. Consequently, depending on the reference point combination, a wide range of headways can result from region to region measurements, and so the recommendation is made to use methods commensurate with practices in the region where saturation flow measurement takes place (Teply 1991).

Until the 1994 HCM edition (TRB 1994), the most common practice in North America was to use rear wheels of vehicle and stop bar, but for consistency purposes with freeway headway measurements, the 2000 edition of the same manual suggests using front wheels of the vehicles instead (TRB 2000).

There are several other interesting, more recent research reports such as a PhD dissertation (Bonneson 1992), which reported that based on information of three single-point urban interchanges (SPUI[1]), calibrated models predict minimum discharge headways generally lower (1.81 seconds) and start-up lost times higher (3.67 seconds) than those calculated by traditional procedures. Bonneson also found the *h* value for the left turn lane varied with turn radius and was significantly lower than its through movement and lower than values traditionally used for protected left-turn movements under ideal conditions. Where ideal conditions include a 12-ft lane width, a level approach grade, no pedestrian, no curb parking, no illegal curb parking present, no near-side or far-side bus stop activity or signs designating the potential for buses to stop,

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[1] A new type of interchange first constructed about 30 years ago; there were around 40 of them operating in the United States, in the early 1990s.

intersection in an outlying area and less than two percent heavy vehicles. Figure 2.3shows a typical SPUI.
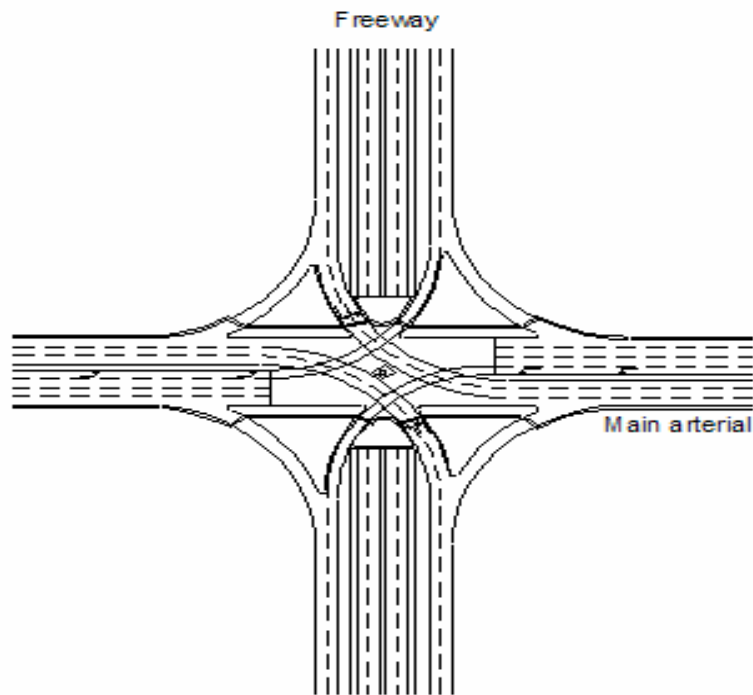
Figure 2.3 Single point urban intersection.

Bonneson also concluded that together with data from two other at-grade intersections, his discharge headway model indicated that h of a traffic movement was not reached until the 8th or higher queue positions, instead of the usually accepted 5th position.

A comprehensive study on saturation flow rates at intersections in five cities conducted in 1987 and 1988, demonstrated that saturation headways and

flow rates have a significant probabilistic component, making calibration of stable values difficult (McShane 1998).

Area type has also been addressed as a source of variation when studying capacity (saturation headways, in the end) of signalized intersections. Research undertaken in Florida to study the effects of four different area types on the capacity of signalized intersections, resulted in the recommendation of a new area-type adjustment factor of 0.92, for recreational settings (Le 2000).

The implied high variability in saturation headways was addressed in a more recent study, specifically in regards to long queues affecting stability of saturation headway, finding that traditional methods might underestimate saturation flow by almost 400 passenger cars per hour when compared to alternate method based on mean rate after the 23rd vehicle in a queue (Lin 2003).

A study at six signalized intersections in Santa Fe de Bogotá, Republic of Colombia was conducted as a part of an integral effort to develop a planning and design handbook for traffic and transit management applicable to that country. Field data was videotaped during February and March of 1998, and found that for all six intersections, the median value of h was 1.90 seconds for vehicle positions 5 to 11. The corresponding median value of h for first 4 vehicles was 2.29 seconds, and so start-up lost time resulted in [(2.29 – 1.9)x4] or 1.56 seconds (Cal y Mayor 1998).

A student in the Traffic Engineering graduate program at Universidad Autónoma de Nuevo León (López 1998), conducted a study in Monterrey, the capital city of the State of Nuevo León, and collected field information from 19 signalized intersections. Without stating some important details, such as queue length per cycle and treatment for the presence of vehicles other than passenger cars, time measurements were recorded by a two-man crew using a stopwatch for the 4th, 10th and last vehicle positions in each lane, at each intersection. Saturation flow values (*s)* were then determined by means of the following equation

$s = 3{,}600/[(T_1 - T_4)/(N_1 - 4)]$

where

s = saturation flow in vehicles per green hour per lane
T_1 = time of last vehicle crossing reference line, seconds
T_4 = time of 4th vehicle crossing reference line, seconds
N_1 = number of last vehicle crossing reference line

The mean saturation value from 19 intersections, resulted in 1,876 vphgpl or an *h* value of 1.92 seconds; the 85th percentile value of *s* was also determined as 2,056 vphgpl and then used instead of the default *s* value 1,900 vphgpl to show how this affects intersection performance assessment in terms of level of service classification.

However, it was not clear from the report why the 85th percentile was selected as the representative value for *s*. No reference or calculations in this report were made to determine start-up lost time (López 1998).

From the previous discussion, a number of questions or additional motivations for this work arise. Of course, the very first motivation continues to be the determination of h values for Monterrey, México, conditions. But also an attempt will be made to arrive to some conclusions when comparing h values occurring during weekday AM peak hour periods, to those during the weekday PM peak hour periods or weekend peak hour periods. The other likely interesting variable to look at is the type or use of lane, for example, through lanes versus left-turn lanes, and left-turn versus u-turn lanes as well.

Since *sult* estimation is dependent on the vehicle position at which h occurs, this may well be another aspect to be addressed. A comparison of h values to be determined in this work to those determined by López might be biased, because of the different field data collection method, that is to say López's stopwatch method versus the videotape method used for this research; bias might also come from the amount of field time measurements between vehicles, López's three per cycle versus as many as the number of vehicles in queue (variable from five to twenty) in this research.

2.3   Importance of *sult* in signal timing

To end this chapter, a comment on the importance of start-up lost time, as the main component of total lost time, to signal timing and in turn to fuel savings and emission reductions. Signal timing is a complex process by which cycle

length and cycle split or phasing are determined with consideration of pedestrian,

vehicle and driver characteristics as well as intersection geometry including lane

layout. Signal timing objectives including:

- Providing the orderly movement of traffic
- Minimizing average delay to vehicles and pedestrians
- Reducing the potential for accident-producing conflicts, and
- Maximizing the capacity of each intersection approach

are not compatible. That is to say, it is not possible to minimize delay by using as

few phases as possible and the shortest practical cycle length if, at the same time

multiple phases and longer cycles are indicated to produce fewer conflicts and in

consequence to reduce accident potential.

There are several procedures to calculate cycle length for a signalized

intersection that is not part of a coordinated system.   The importance of *tlt* in

signal timing is best understood when using Webster's equation:

$$C_o = \frac{1.5L + 5}{1.0 \; - \; \Sigma \, Y_i}$$

where

C_o = optimum cycle length that minimizes delay, in seconds,
L = lost time per cycle (*tlt* times the number of phases) in sec, and
Y_i = flow ratio = $\dfrac{\text{critical lane volume[2] for } i^{th} \text{ phase, vph}}{\text{saturation flow } s, \text{ in vph}}$

---

[2] During any given signal phase *i*, non-conflictive traffic flow on lane(s) along two or more approaches is allowed to move, with one or more of such lanes moving the highest flow. This is the critical lane volume for *i*[th] phase.

Setting aside the fact that cycle lengths in the range of 0.75 to 1.5 times C_o do not significantly increase delay (McShane 1998), it is clear from this equation that the higher *tlt* the longer C_o and consequently a higher delay will occur. Using a higher *tlt* than the actual one in the process of signal timing could then lead to poor traffic signal timing.

If we are dealing with only one intersection, at only one peak hour on a weekday, the aforementioned insignificant delay increase resulting from a longer cycle length, will not make much of an adverse impact. The problem is when we aggregate an increase to every peak hour, every week to the more than 260,000 signalized intersections in the United States.

This might be the type of reasoning, which estimates that poor traffic signal timing accounts for 5-10% of all traffic delay, or about 300 million vehicle-hours of delay, on major roadways alone (NTOC 2005).

The following are the positive impacts that a more actual *tlt* used as part of a more efficient, frequently checked signal timing process at signalized intersections, might have:

- Delay would decrease by 15 to 20%
- Travel time would reduce up to 25%
- Emissions would reduce up to 22%, and
- Fuel consumption would decrease up to 10%

Chapter 2 consisted of a review of relevant literature regarding the origin and evolution of studies addressing queue discharge headway at signalized intersections. The next chapter basically deals with the experimental design, identifying sample size and factors to be considered in the remaining chapters.

CHAPTER 3

EXPERIMENTAL DESIGN

In conducting studies for saturation flow determination, besides assuring the presence of specific traffic, control and intersection lane layout conditions when making the site selection, site conspicuousness for data collection activities is also important. Statistically based common practice states that sample size n, should consist of 30 valid queues. A valid queue means seven to ten passenger vehicles waiting for the intersection's traffic light change from red to green (ITE 2000). This light change, in a way, might well also be referred to as a cycle.

Nevertheless, since this traffic condition is related to an h determination procedure in the United States that assumes h is reached after the 4th vehicle in the queue, and considering that one of the additional objectives of this research is to determine h for Mexican conditions in Monterrey and to try to estimate queue position at which h is reached, some variations were regarded as valid. These variations included reducing the number of valid queues, or cycles from 30 to 20 cycles in one of the study sites, and a minimum of five instead of seven vehicles waiting for light change from red to green.

Despite the random nature of traffic arrivals at signalized intersections, assuming field data is normally distributed and keeping both standard deviation (σ) and estimated mean error (ε) of saturation flow rate (of h, in a way) within specified amounts common in the United States ($\sigma = 140$ vph and $\varepsilon = 50$ vph, respectively, ITE 2000), the number of cycles reduction from 30 cycles to 20 cycles is almost equivalent to reduce confidence level from 95% to 90%. The number of queued vehicles at the light change which is recommended when observing prevailing saturation flow rate, is at least five (Roess 1998).

As pointed out toward the end of chapter 2, the data collection equipment for this study consisted of videotaping. This particular technology provided a number of advantages over traditional stopwatch, especially because of its greater accuracy and creation of a permanent record that made it available either for double checking results or for conducting future studies.

Good light conditions and an inconspicuous clear vantage point are main site characteristics required for videotaped data collection . Even though an intersection may have the other necessary traffic, control and layout conditions, not having videotaping inconspicuousness most likely would eliminate that intersection, since drivers might well be affected in their reaction time once they were aware of being videotaped, thus resulting in deceptive h values.

Time of day, weather, and other events such as land use and traffic signal sequence may affect driver behavior. In turn, driver behavior has an effect on

saturation headway h, which produces the high variability already mentioned in chapter 2. Another factor likely affecting driver behavior is the maneuver taking place along each lane in the subject approach. For instance, if the left most lane is an exclusive lane for left turn rather than for through movement, the driver will slow down because of the turning maneuver thus increasing h. An attempt was made to include the number of lanes in the approach as a factor in the experimental design. However, after several journeys searching for sites with different numbers of lanes on their prospective approaches, those with only one or two lanes had to be discarded either because of not offering a clear vantage point for videotaping, due to low traffic volume conditions, or due to both reasons.

Bearing this in mind, the experimental design of this research addresses time of day (factor A with levels 1 and 2) and lane (factor B with levels 1, 2 and 3). The factor A levels were AM and other-than-AM , and the factor B levels represented the lanes for which data was collected, with lane 1 being the left most lane, and so forth. For that reason, data were collected from signalized intersection approaches both at AM peak periods and at other-than-AM peak periods (either at noon, afternoon or in the evening), and with separate left and through-movement lanes. Table 3.1 below shows this experimental design.

Table 3.1 Experimental design

		Factor B		
		1	2	3
Factor A	1	h_{11}	h_{12}	h_{13}
	2	h_{21}	h_{22}	h_{23}

This chapter briefly dealt with sample size and factors affecting headways. A mixed factorial design consisting of factor A (peak period, with levels AM and PM) and factor B (lane number, with levels 1, 2 and 3) has been presented. Finally three sites were selected for the study. They are described at the beginning of the next chapter, followed by an explanation of the field data collection method and continuing with one section for every site studied.

CHAPTER 4

FIELD DATA COLLECTION

There are five sections in this chapter. Three selected sites are introduced in the first section, and then the second section describes method for field data collection and processing. Appendix A presents the data used in the analysis only; additional data was collected in the field, but it was not included in the analysis. Each following section from third to fifth, presents information regarding to the three selected sites.

4.1   Selected sites

In the end, intersection approaches found with the necessary characteristics mentioned in the previous chapter, have five lanes at two signalized intersections, and three lanes at a third signalized intersection. The two signalized intersections with five lanes in the subject approach are along Eugenio Garza Sada Ave., a main 8-lane (on the average) two-way arterial connecting south of the Monterrey metropolitan area to the federal highway number 85, known as México – Laredo highway. The third signalized intersection is formed by Lázaro Cárdenas Ave. and Paseo de las Fuentes Ave. Here, Lázaro Cárdenas Ave. is also a two-way arterial serving as a main connection between the cities of

Monterrey and San Pedro Garza García, in the south part of the metropolitan area. Westbound Lázaro Cárdenas Ave. is the subject approach with three main lanes.

4.2   Field data collection and processing

Two AM and two PM peak periods, each on a different day were recorded at each of the three sites. After a number of trials that allowed for peak period identification and familiarity with the recording equipment, procedure and its details, each recorded period was afterwards converted to VHS format so that later reproduction in a video cassette player at thirty frames per second slow motion, would allow for reading individual vehicle times to an accuracy of hundredths of a second, finally rounded to tenths of a second.

Individual vehicle time readings were taken when the rear axle of vehicles crossed the reference line shown in every corresponding figure. An Excel spreadsheet was used to first build a table containing those readings, which in turn easily generated another table with individual headway values. This last table shows individual headways for each cycle number in rows and vehicle or queue position in columns, with average headway for every vehicle position at the end of columns.

Taking advantage of the spreadsheet capabilities, both mean and median headway values for every queue position are presented and later analyzed to see the difference between them. The potential importance of slightly different mean

and median headway values in capacity analyses is explained at the beginning of chapter 5, data analysis.

Where heavy vehicles were present, the average headway calculations were made both with and without incidence of that type of vehicles. That is to say, in order to see the effect of heavy vehicles, associated mean and median headways are first indicated just below the last cycle. After these average headways, it is stated which entire cycle(s) and or headway cells were discarded in newly calculated average headways, displayed at the very bottom of the table and later used in the analysis.

Cycle and timing for each of the three sites is operated through a traffic management center known as SINTRAM, handling up to six different cycle lengths depending on the time of day or night, i.e. depending on traffic demand.

4.3   Site 1

Garza Sada Ave. and Luis Elizondo St. signalized intersection is hereafter referred to as Site 1. For security reasons it was necessary to inform officials of the field data collection activities, particularly in this site due to a private university (Insitituto Tecnológico y de Estudios Superiores de Monterrey, ITESM) abutting to it. Figure 4.1 on the next page shows the site, with traffic signals regulating flows by means of three phases: A) Southbound, lead-protected left turn and through movement, B) Southbound and Northbound through

movements, and C) Eastbound through and turning movements. Lane numbering is from left to right, with lane one being left-turn exclusive and lanes two through five for through movement.
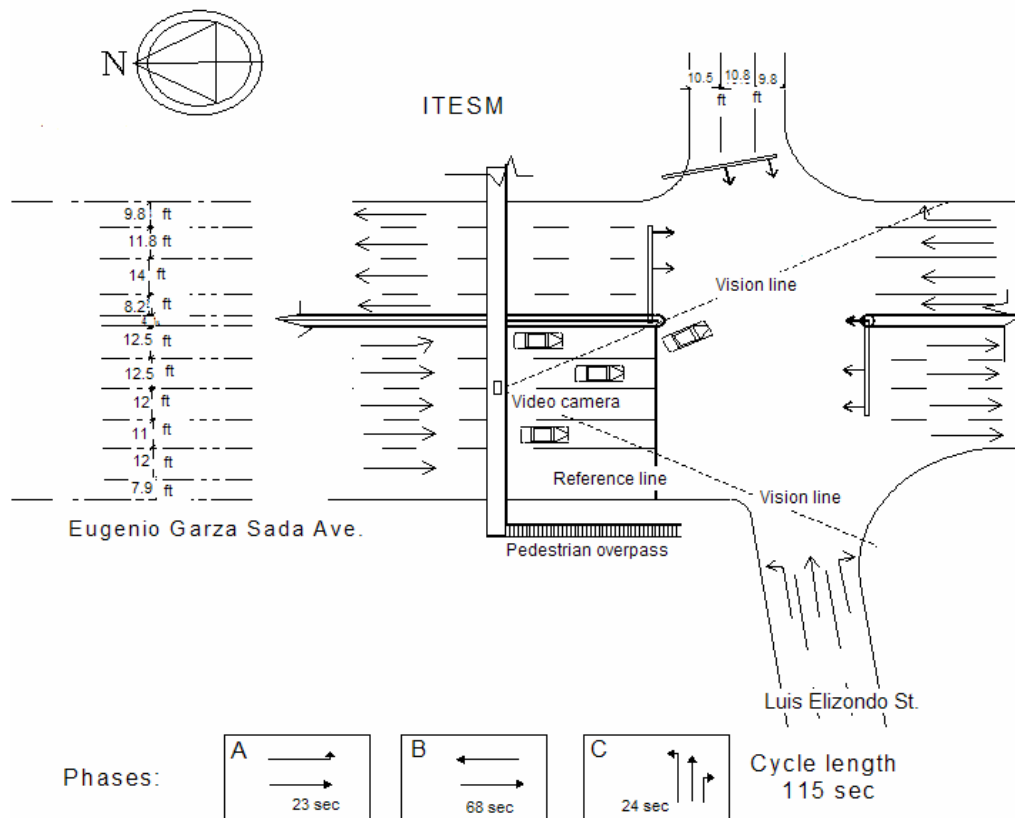

Figure 4.1 Signalized intersection Garza Sada Ave. -- Luis Elizondo St. (Site 1)

Southbound is the subject approach, with its cross-section changing from 5 lanes plus parking upstream of Luis Elizondo St., to only 4 lanes past the street. The best point for video taping the approach permitted the collection of

information from lanes 1, 2 and 3. As an inquisitive update, it is worth mentioning that construction of an overpass started in this intersection around mid August 2004, and it was completed by February 2005.

A number of bus routes run along Garza Sada Ave.; the buses mostly use lanes 4 and 5, therefore no data was collected from these lanes. Moreover, the video camera did not suitably cover those lanes.

A few buses or trucks were traveling in lane 3, which were noted when writing down their times so that corresponding headway determination may be properly assessed later on; the same goes for motorcycles. Vehicles not correctly using their traveling lane, such as those that travel on a through lane but make a left turn, were also identified.

The AM peak periods (approximately from 7:45 to 9:45) were recorded on Wednesday, February 25, and on Tuesday, April 20, 2004. For lane 1 (left turn only lane), Table A.1 in the appendix contains 525 readings from 45 consecutive cycles videotaped on that date; Table A.2 includes 480 individual headway calculations from readings in Table A.1.

All of the 45 cycles contained more than five queued vehicles when the signal changed from red to green, except for cycle 37 where only four vehicles were queued at that time; as a result, it was discarded in calculations of h along with cycles containing heavy vehicles, as indicated at the bottom of Table A.2.

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This table also shows the derived average headways for every vehicle or queue position and its corresponding number of cycles as well.

Five unusually low first-headway values ranging from 1.0 to 2.0 seconds occurred during this AM peak period recording, as either identifying drivers who anticipate the signal change from red to green, those stopping beyond the reference line thus reducing their headways, or both.

The second AM peak period recording for this same site was on Tuesday, April 20. Table A.3 includes 448 readings from 40 consecutive cycles videotaped on that date. Table A.4 records 480 individual headway calculations from readings in Table A.3. On this date, the minimum number of queued vehicles present when the signal changed from red to green was eight in 5 out of the 40 cycles, while 8 of them registered ten queued vehicles at signal change.

Unusually low first-headway values occurred in 7 out of 40 cycles, ranging from 0.7 second to 2.0 seconds; among the 408 individual headways, four corresponded to heavy vehicles and only one to a motorcycle. Since a heavy vehicle in the 2nd position of cycle 21 did not generate an unusually high headway for such a position (3.0 seconds) in the queue, and did not greatly influence the headway of next queue position (2.3 seconds), this particular value was not discarded in obtaining average headways.

On the other hand, cycle 32 was entirely discarded due to resultant heavy vehicle's headway of 8.0 seconds in the second position; in cycle 36 the last two

headway values were not considered because of the heavy vehicle's headway of 5.8 seconds in the second to the last position. Finally, the last cycle presented a heavy vehicle in its ending queue position number 8, also producing a high headway of 2.9 seconds requiring exclusion. Again, the bottom of Table A.4 allows for comparing average headways with and without cycles and headway cells involving heavy vehicles.

Tables A.1 through A.4 referred to lane 1 field data readings, individual headways and headway averages for two AM peak periods, each on a different date, Wednesday, February 25 and Tuesday, April 20. As mentioned earlier, the video camera location allowed for the collection of analogous information for lanes number 2 and 3 as well.

By following the same information organization pattern, Table A.5 contains lane 2 readings recorded on Wednesday, February 25, and Table A.6 includes headways from 45 consecutive cycles derived from Table A.5. Similarly, Tables A.7 and A.8 include respectively the readings and headways from recordings of Tuesday, April 20. In the same way, Table A.9 through Table A.12 display the information for lane 3, in the same sequence of contents and dates.

In order to avoid tedious wording repetition, the PM peak period (recordings made approximately from 12:00 to 14:00) readings and headways for Site 1 are contained in Tables A.13 through A.24, again in the same sequence of contents and dates.

4.4   Site 2

The Garza Sada Ave. and Vía Alcalá Ave. signalized intersection is henceforth identified as Site 2. Figure 4.2 shows the site, making noticeable the commercial land use along both sides of Garza Sada Ave. Southbound is also the subject approach and, as shown in Figure 4.2, lane 1 is exclusive for U-turn while lanes 2 through 4 are for through movements and the last lane, number 5, is a through-right turn shared lane. The figure also exhibits the video camera position on the pedestrian overpass, upstream of the reference line again capturing information from lanes 1 to 3, numbered from left to right. Moreover, Figure 4.2 also exhibits that cycle split here is a little more complicated than in site 1, since the traffic directions in Vía Alcalá Ave. are almost 100 feet apart and due to the protected lag-left turn portion of Garza Sada's northbound through phase.

Recordings were made on Wednesday, January 21, and on Thursday, February 19, approximately from 7:45 to 9:45 for the AM peak period and from 12:00 to 14:00 for the PM peak period, on each day. Tables A.25 through A.36 in the appendix contain the information for the two AM peak periods. Some cycles were left entirely blank because they did not accumulate the minimum number of vehicles, which is five.
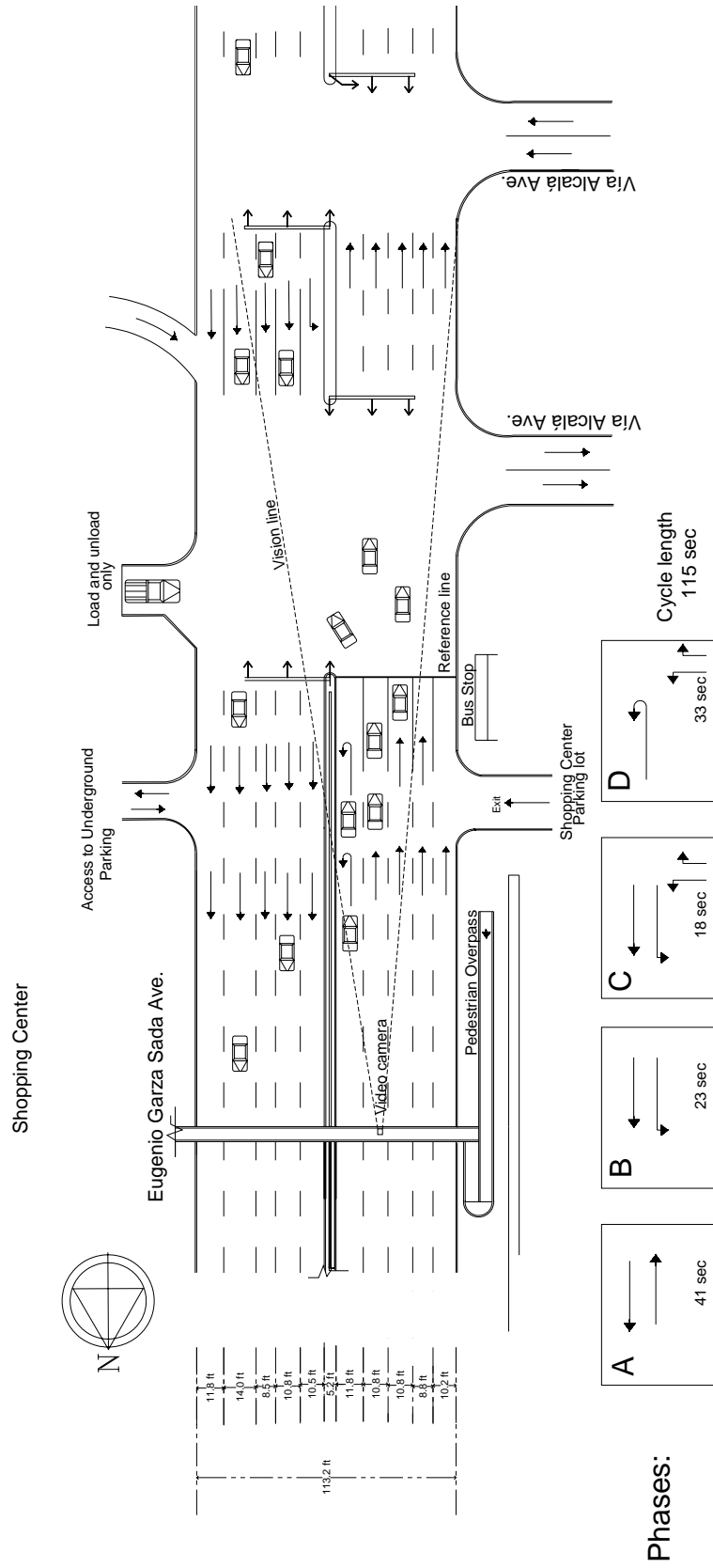
Figure 4.2. Signalized intersection Garza Sada Ave. and Vía Alcalá Ave. (Site 2)

The number of empty cycles varied from seven on Table A.25, to fourteen on Table A.33, nonetheless the sample size amounted to more than 30 cycles; no entire cycles nor headway cells were discarded for heavy vehicle or motorcycle presence in these two AM peak periods. Tables A.37 through A.48 show that something comparable happened with the two PM peak periods, with the empty cycles varying from two on Tables A.39 and A.40, to sixteen on Tables A.47 and A.48.

The previous criteria for discarding entire cycles and specific cell headway values relating to the presence of heavy vehicles or motorcycles was applied to Tables A.44 and A.48. Only lane 3 in the second PM peak period (Table A.48) was impacted, resulting 29 cycles with the minimum of five vehicles.

4.5   Site 3

From now on Site 3 will identify the signalized intersection of Lázaro Cárdenas Ave. at Paseo de las Fuentes Ave., here included as Figure 4.3. Following the same readings and headways organization as that for the two previous sites, Tables A.49 through A.60 in the appendix contain the information of the two AM peak periods while the information of the two PM peak periods is in Tables A.61 through A.72.
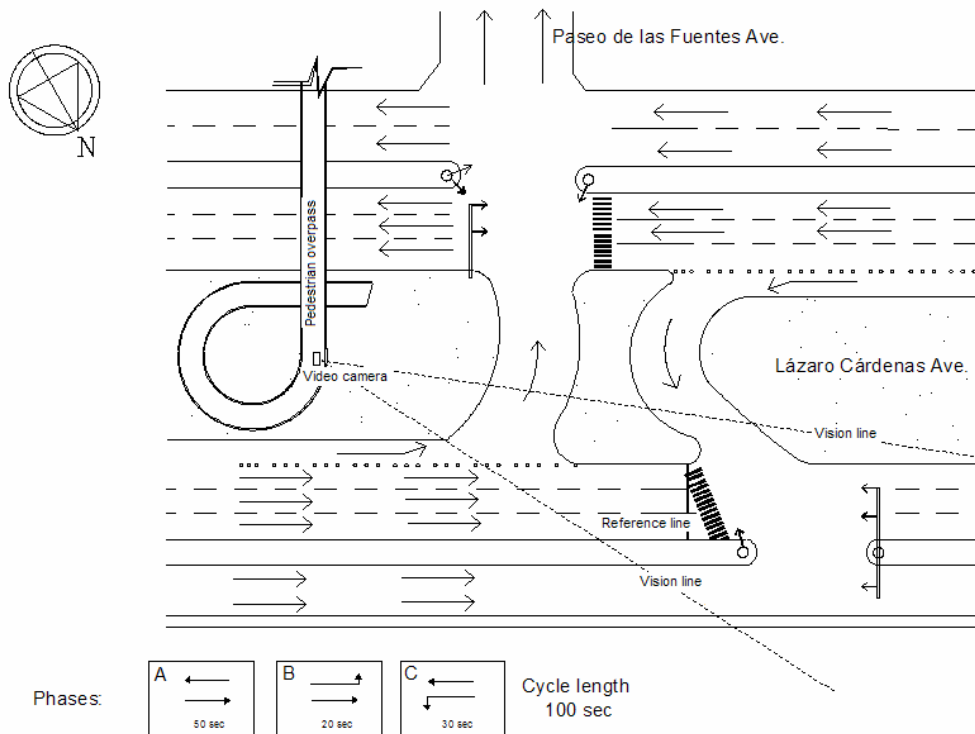
Figure 4.3 Signalized int. L. Cárdenas Ave. -- P. de las Fuentes Ave. (Site 3).

Site 3 recorded the highest number of cycles with the minimum of five queued vehicles. Furthermore, in dealing with lane 3, only 20 cycles had five vehicles on the second recording of PM peak period Table A.72). This sample size still produces a fairly accurate 90% confidence level.

Here, AM peak periods were recorded on Wednesday, April 21, and on Thursday, April 29. It was erroneously assumed that the site would also meet the traffic condition for the PM peak periods on a weekday, given that after trying a

number of PM peak periods Monday through Friday, in the end it became necessary to make the recordings on Sunday evenings.

It had to be on this specific day of the weekend, because westbound Lázaro Cárdenas Ave. has the particular characteristic of serving traffic flows that return from a nearby weekend recreational area. Recording dates were May 23 and July 4, approximately from 18:00 to 20:00 Hrs.

Unlike sites 1 and 2, here lane 1 is a through exclusive lane, along with lanes 2 and 3, though from lane 3 it is possible to exit the main lanes to secondary lanes. However, after retrieving the information from both AM and PM peak period recordings, it was noticed that none of the drivers traveling on lane 3 made the exiting maneuver. Table A.73 in the appendix relates the number of cycles to every queue position for each site.

Chapter 4 introduced the studied sites, described field data collection and processing, and presented figures and applicable information of every site. In the next chapter, analysis of the information determines an overall mean h value as representative for sites here studied, and then it compares two different h calculation procedures, to finally discuss how h values are affected by factors such as time of day and lane.

CHAPTER 5

DATA ANALYSIS

There is an introductory comment in this chapter about the use of either mean or median headway values, and then there are three sections. After the introductory comment, the analysis in first section is done using median headway values only, briefly stating one connotation for this choice.

Stopped delay, in seconds per vehicle, is the measure of effectiveness to determine Level Of Service (LOS) by lane or lane group (two or more lanes with common movements) on every approach to a signalized intersection under study.

In assessing signalized intersections' performance, levels of service go from A to F (with notation LOS_{A-F}), from very good free flow to very poor congested flow conditions. In other words, mean h is typically a little higher than median h, resulting in a higher estimate of control delay, which may result in an approach with lower LOS. That is to say, if a mean headway value is used, LOS_C may result instead of LOS_B should a median headway value be used. Section one is focused on the determination of an overall mean h value for sites studied under Mexican conditions in Monterrey, Nuevo León, then to learn about the vehicle position at which h is reached, with derived or implicit start-up lost time.

Section two is aimed to compare outcomes between h calculation procedures. Two general procedures are here understood, one that Greenshields used, and another that is mostly used nowadays. This study resembles what Greenshields did in regard to considering all of the headways and to using the median headways, contrary to the procedure mostly used nowadays that excludes first 5 headways and uses the mean headways.

Section three, addresses the effects of the two factors described in chapter 3, peak period and lane in a 2x3 mixed factorial model analyzed by means of the Statistical Package for Social Sciences (SPSS).

5.1   Overall mean h values

Greenshields basically determined median headway values for every queue position and by observation estimated queue position at which h levels off, followed by derivation of *sult*; conversely and most likely as a result of Greenshields' findings, nowadays it is commonly assumed that h leveling off occurs from 5th queue position on, discarding or not considering headway values for the first four queue positions, and thus averaging headways from the 5th to the last queued vehicle clearing the intersection at light change from green to yellow.

A quick inspection of tables containing headway averages shows that most of the times the mean headway resulted in higher values than the median headway, with the most frequent difference being of 0.1 or 0.2 second, a few

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cases with 0.3 second and seldom with 0.4 second difference. In some instances the values were the same for both mean and median, and in very few cases the median headway resulted in 0.1 second higher values than the mean headway.

Table A.74 summarizes for Site 1, previously determined median headway values for every queue position from number one to number eleven across lanes 1, 2 and 3. Then, peak period headway values are averaged for every queue position (average AM and average PM columns), as well as lanes 2 and 3 values (L2&3Avg in Table A.74). Finally with this table, the last row also presents the total number of cycles per lane, per peak period and per replicate, while the last column does the same for every queue position.

The mean of the eleven median headway values by queue position in seconds, for each of the peak periods and lanes (from Table A.74 as well), is included below.

peak	lane 1	lane 2	lane 3
AM	2.1	2.1	2.2
PM	2.2	2.0	2.1
dif.	0.1	0.1	0.1

Now, assuming these as h values, start-up lost time would be the extra time consumed by the first few vehicles. For example, using lane 1data from Table A.74 average AM column, median headway values starting with queue

position 1 are: 3.3, 2.6 and 2.2 seconds, consequently *sult* would be (3.3-2.1)+(2.6-2.1)+(2.2-2.1) or 1.8 seconds (the headway of the fourth vehicle matched the saturation headway). By repeating this simple process, the rest of *sult* values in seconds are:

peak	lane 1	lane 2	lane 3
AM	1.8	2.1	2.2
PM	1.7	1.8	1.8

Noting such a small difference between *h* and *sult* values across lane numbers and peak periods, final estimated overall mean *h* value for Site 1 is 2.1 seconds and overall mean start-up lost time value is 1.9 seconds; with number of observations amounting to 4,534 as noticed in Table A.74. It should be recalled that lane 1 at Site 1 is for left turn only, and at Site 2, for U-turn only. Using again data from Table A.74, Figure 5.1 compares AM vs. PM median headway values for lane 1. Except for the first queue position, Figure 5.1 makes evident that headway values are higher during PM peak period, thus being an indication time of day has an effect in headway values.
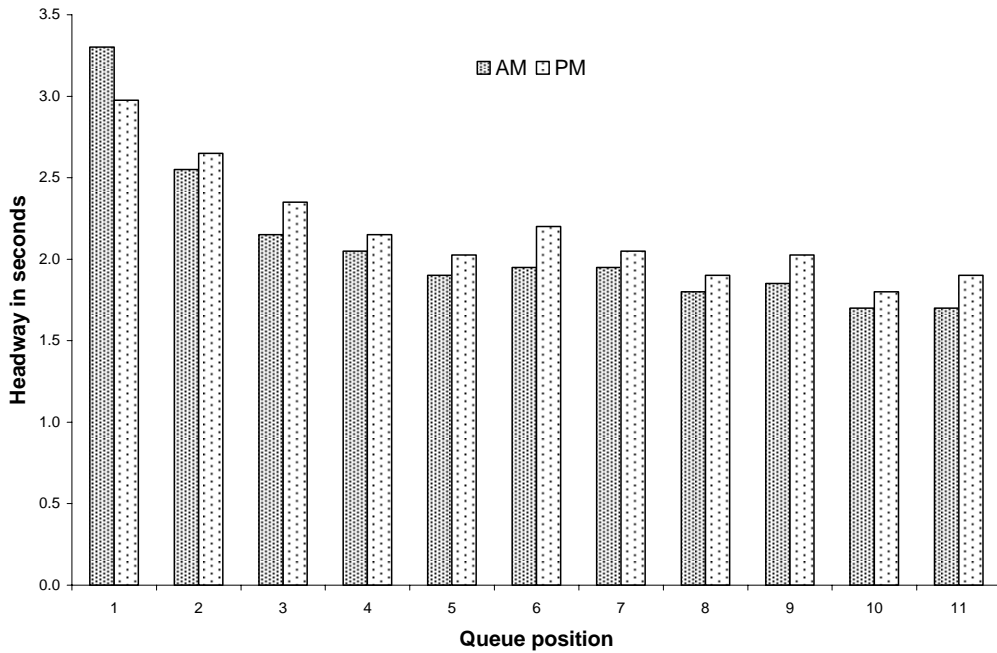
Figure 5.1. Lane 1 AM vs. PM median headway comparison.

Once more from data in Table A.74, Figure 5.2 on next page compares AM vs. PM median headway values for lanes 2 and 3 (L2&3Avg). Table A.75 shows the summary of median headway values for Site 2. The mean of the first eight values in seconds across lanes 1, 2 and 3 are next:

peak	lane 1	lane 2	lane 3
AM	2.4	2.1	2.1
PM	2.5	2.3	2.3
dif.	0.1	0.2	0.2
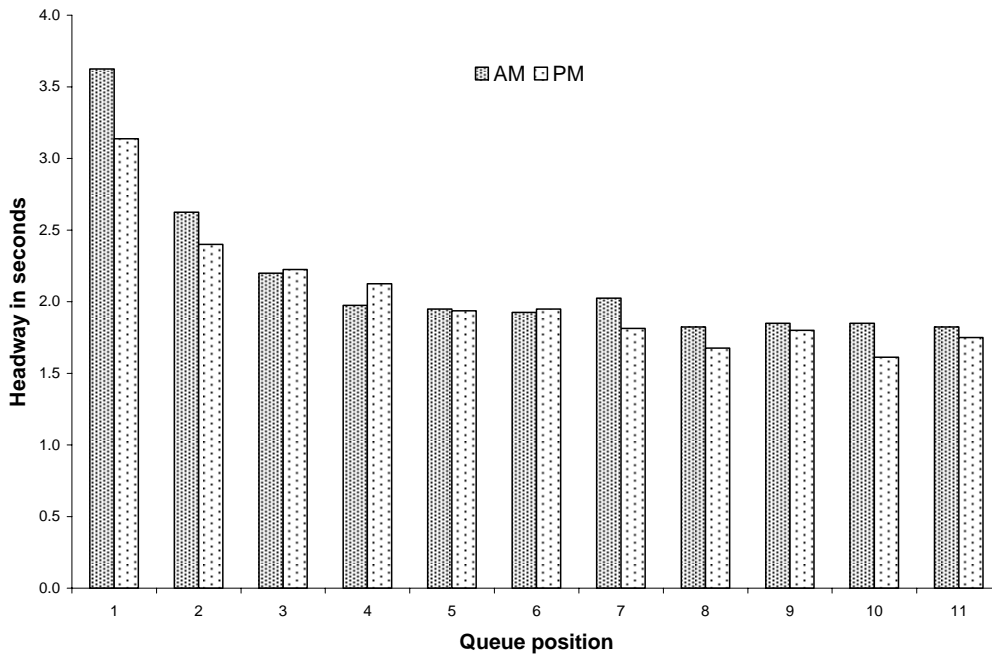
Figure 5.2 L2&3Avg. AM vs. PM median headway comparison.

and using these along with data from Table A.75, *sult* values in seconds result in:

peak	lane 1	lane 2	lane 3
AM	1.1	1.5	1.4
PM	1.1	1.7	1.7

For Site 2 then, estimated overall mean *h* value was rounded to 2.3 seconds and *sult* value resulted in 1.4 seconds.

Remembering that lane 1 use is different between Sites 1 and 2, the mean headway values in seconds presented in Table A.74 and Table A.75 respectively, seem to reflect this difference:

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peak	site 1	site 2
AM	2.1	2.4
PM	2.2	2.5

The likely cause for the difference of 0.3 second from Site 1 to Site 2, is the implied lower speed when making a U-turn at Site 2, as compared when making a left turn in Site 1. Although as little as 0.1 second, the above general difference between AM and PM peak periods headway values of these two sites, might as well reflect driver behavior, since driver motivations are usually quite different (in a hurry and so prone to higher speeds) early in the morning than at any other time of day. Figure 5.3, better illustrates the median headway value difference, between left turn in Site 1 and U-turn in Site 2 (for lane 1). In regards again to only Site 2, Figure 5.4 also compares AM vs. PM median headway values for lane 1 on a queue position basis. The same comparison for the average of lanes 2 and 3 (L2&3Avg) at Site 2 is shown in Figure 5.5.

Table A.76 summarizes the corresponding results for Site 3. In this particular site, the number of cycles with the minimum number of five queued vehicles went as low as twenty, and although the number of cycles was ten in one of the six recordings (Table A.73), data for the analysis included up to the sixth queue position.
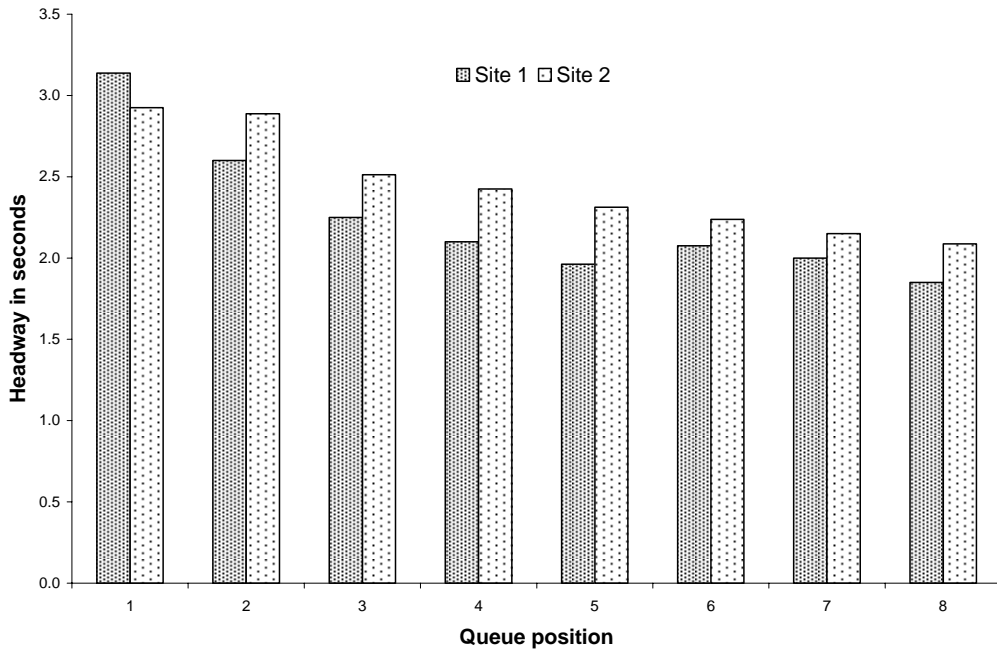
Figure 5.3 Lane 1 headway value comparison between Site 1 and Site 2.
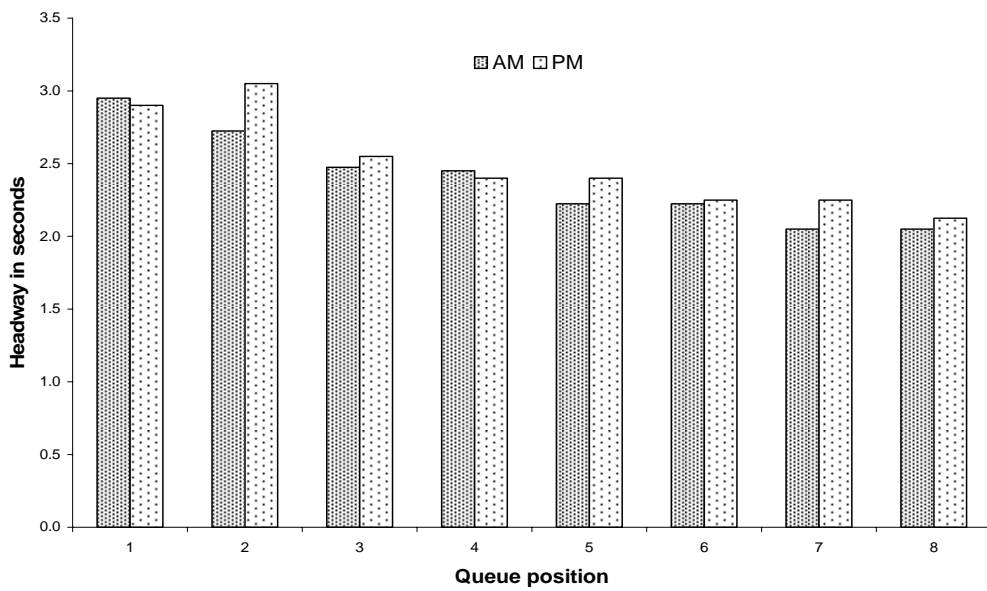
Figure 5.4 Site 2, lane 1 headway value comparison between AM and PM.
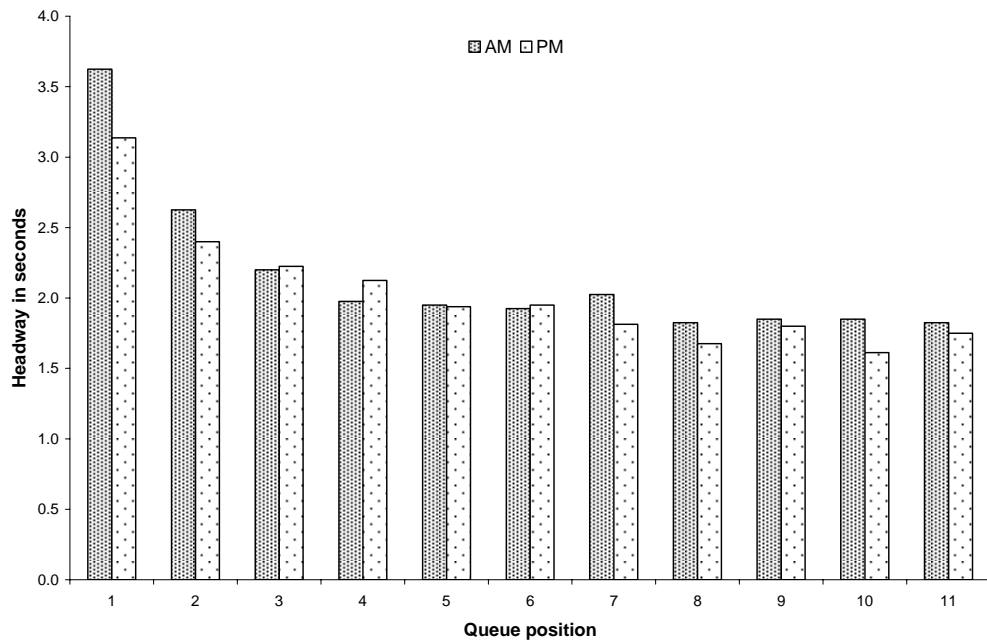
Figure 5.5 Site 2, L2&3Avg headway value comparison between AM and PM.

Following the same procedure with Site 3 as used for Sites 1 and 2 above,

the mean of the median headways resulted in:

peak	lane 1	lane 2	lane 3
AM	2.3	2.4	2.4
PM	2.4	2.2	2.4
dif.	0.1	0.2	0

with derived start-up lost times:

peak	lane 1	lane 2	lane 3
AM	1.3	1.4	1.6
PM	0.9	1.2	0.8

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and estimated overall mean *h* value for the site rounded to 2.3 seconds and *sult* at 1.2 seconds.

In this site, lane 3 offers the alternative to be used as thru-right diagonal (exit) shared, but none of the video taped vehicles traveling along it exited to secondary lanes. Because of this, here the average of the three lanes is compared between AM and PM in Figure 5.6, using values from Table A.76.
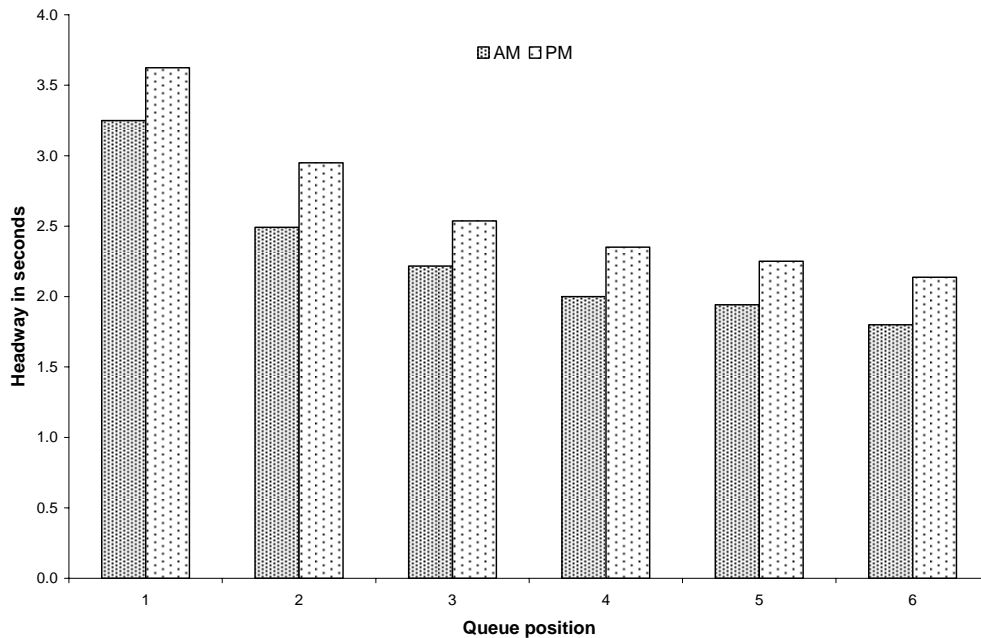
Figure 5.6 Site 3 headway comparison between AM and PM, avg of all three lanes

L2&3Avg. headway values of Site 1 and Site 2 are compared to headway values of Site 3 (all three lanes). Note that the comparison includes only the through lanes

peak	site 1	site 2	site 3
AM	2.2	2.1	2.4
PM	2.0	2.3	2.3

Note that each of the two peak periods reports a difference of up to 0.3 seconds in the headway value between sites, which is a reflection of h variability from site to site.

Based on the highest amount of field data (see total number of cycles in Tables A.74 through A.76), Site 1 could be regarded as the best representative of this work. It is interesting to note that its overall mean h value of 2.1 seconds corresponds to the one Greenshields determined around 60 years ago.

5.2   Comparison of two procedures for h calculation

The queue position at which h value levels off and subsequent *sult* value, resulted with some differences. Greenshields reported h leveling off at sixth queue position with derived start-up lost time value of 3.7 seconds, while in this research h leveled off typically after the third queue position with derived start-up lost time value of 1.9 seconds. Furthermore, the h calculation procedure mostly used nowadays (Roess, 1998), essentially makes an average of headway values for queue positions fifth to last in the queue.

Applying this last procedure to Site 1 median headway values of Table A.74, that is to say by averaging fifth to eleventh queue positions' median

headway values, we would have the following mean headway values (for a quick comparison, figures in parentheses are the previously presented mean headways) in seconds:

Peak	Lane 1	L2&3Avg
AM	1.8 (2.1)	1.9 (2.2)
PM	2.0 (2.2)	1.8 (2.0)

and, subtracting these from corresponding first median headway values on Table A.74, *sult* values in seconds would be (for a quick comparison, figure in parentheses are the previously presented start-up lost time values):

Peak	Lane 1	L2&3Avg
AM	3.5 (1.8)	2.9 (2.2)
PM	2.3 (1.7)	3.0 (1.8)

being apparent the big difference between discarding headways of first few queue positions and considering all of them, shown in parentheses, and calculated in Section 5.1.

It is very interesting to note that headway value of 1.9 seconds (AM, L2&3Avg, fifth to eleventh queue position) is almost the same as estimated from the first study of this kind in Monterrey, previously referred to in Chapter 2 (López 1998).

Table A.77 allows a comparison of similar lane uses, and median headway values by queue position, both at a single site itself and across sites. For instance,

about Site 1, AM peak, queue position number one first of all shows that opposed to what could be expected, left turn movement (lane 1) headway value of 3.4 seconds is lower than through movement (lane 3) headway value of 3.7 seconds, with the second replicate (second collection date) showing an even greater difference of 0.7 second.

Again regarding the 1st queue position, but for Site 2, an even higher headway value for lane 1 should be expected than for lanes 2 or 3, since a U-turn movement must be performed at a slower speed than a through movement; Table A.77 however reveals just the opposite, particularly in the 1st replicate of PM peak period where the headway value of lane 3 was 1.0 second higher than that of lane 1.

One possible explanation for this is the phase sequence of the traffic signal there (see Figure 4.2). There is a 3 second offset between the two sets of traffic lights that regulate southbound Garza Sada's Ave. through movement between phases D and A, so that when the green ball is displayed in the first set, the second set indicates red for another 3 seconds, time that eastbound Vía Alcalá Ave. drivers still use to cross west section Garza Sada Ave., thus impeding first southbound drivers' timely reaction. As expected, queue positions 2nd to 8th consistently reported higher headway values for lane 1 than for lanes 2 or 3.

Remembering that every lane of subject approach at Site 3 is for through movement, most headway values in Table A.77 precisely make evident the notion

of drivers reaching higher speeds on left lanes than on right ones, with lower headways in lane 1. For instance, headway values for lanes 1 and 3 in first replicate of same AM peak period, show the greatest difference of 0.8 second.

In a comparison of queue position number one across peak periods for Site 3, the highest difference was found to be 1.2 seconds:

peak	lane #, rep #	h in seconds
AM	1, 1	2.8
PM	2, 2	4.0

Of the three sites, this is the greatest peak periods' difference in headway values for queue position number one for the same lane use. The next highest headway difference for queue position one, 0.7 second results from Site 2 when comparing replicate one of either lane 2 or lane 3 AM peak period vs. corresponding replicate one of lane 2 or lane 3, PM peak period.

5.3 Effects of factors peak period and lane use

Now however, the other part of the ANOVA analysis concerns the 2x3 mixed factorial model, for which the Statistical Package for Social Sciences (SPSS) was used to assess the effect of peak period and lane use. Despite the capability of this package to perform with missing pieces of information, the number of queue positions analyzed for every site conforms to what is shown in Table A.73.

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That is to say, for Site 1 the number of queue positions analyzed is eleven, since for the twelfth queue position, four out of twelve data points were not available (not enough vehicles showed up).

Similarly, for Site 2 the program was run up to the ninth queue position, and up to the seventh queue position for Site 3. Both mean and median headways were used when running SPSS for each of the three sites.

As introduced in chapter 3, the peak period was identified as factor A with levels AM and PM (other-than-AM), whereas lane from which data was collected was identified as factor B with levels 1, 2 and 3; each factor combination had two replicates. Tables A.78 to A.80, summarize the SPSS input information respectively for Sites 1, 2 and 3. Each table includes both median ('med' column) and mean ('mean' column) headways previously estimated.

Regarding the output tables (shown in Tables A.81 through A.86), whenever a factor shows a significance less than or equal to 0.05 in last column (Sig.) of the table, it is interpreted to mean that that factor significantly affects the headway. Should this significance figure be less than or equal to 0.01, then the factor effect in terms of headway difference is highly significant; same criteria applies to factors' interaction. While headways values in SPSS outputs are reported to the thousands accuracy, following discussion uses only two significant digits.

With this in mind, Table A.81 contains output for Site 1 when the median headway was used. A quick inspection of Table A.81 reveals that only queue position 1 resulted with a highly significant effect from factor A (time of day), since its significance value was 0.006; AM mean headway reported by SPSS was 3.52 seconds while PM mean headway was 3.10 seconds, a little more than a 0.4 second difference. There was no interaction between factors A and B.

Table A.82, assessing mean headway at Site 1, shows that both factors A (time of day) and B (lane) affect queue position one, whereas queue position four was affected only by factor A. These were the only two queue positions significantly affected, and again, there was no AB interaction. For queue position one, with the significance values for factors A and B respectively, at 0.001 and 0.006. As such, each effect is said to be highly significant. The difference in mean headways, as reported by SPSS, across factor A levels was of 0.42 second, since AM mean headway 3.63 seconds and PM mean headway was 3.21 seconds. As for factor B, its highly significant effect is reflected when comparing lane 1 (left turn only lane) mean headway at 3.23 seconds, to lane 3 (through lane) mean headway at 3.63 seconds, making a difference of 0.40 second. No factor B effect had significance between lanes 1 and 2, nor between lanes 2 and 3.

As mentioned above, fourth queue position was affected by factor A (0.038 significance value), with AM mean headway at 2.16 seconds versus PM mean headway at 2.24 seconds, which means an almost 0.1 second difference.

Then for Site 2, Table A.83 shows the results of the ANOVA analysis when median headway is used, revealing that one or both of the factors significantly affected eight (bold indicated) out of the nine queue positions. Table A.84 on page 231 in the appendix contains the results of the ANOVA analysis when mean headway is used, showing that factor B (lane) highly affected six out of the nine queue positions, while factor A (time of day) effect was significant only on queue position five, as clearly indicated.

So for Site 2, average headways are shown for each condition of factors and queue positions that showed a significant impact (significance less than 0.05) for both median and mean headway. When taking differences shown in Table 5.1, AM mean headway values were subtracted from PM's since the assumption was made that AM's would result lower than PM's. Similarly for factor B, either lane 2 or lane 3 headway values were subtracted from lane 1 values as it was supposed that headway values for lane 1 would be higher than those for lane 2 or lane 3.

The three headway differences due to factor A were: 0.31 second for queue position 1, .28 second for queue position 2, and .13 second for queue position 5; the headway differences due to factor B varied from -0.41 second for queue position 1, to 0.46 second for queue position 5. SPSS output of the ANOVA analysis for Site 3 is presented in Tables A.85 and A.86, for median and

Table 5.1-- Site 1, A and B effects on headways

Queue position	Headways in sec		Headways in sec			Factor difference in sec	
median type of average							
	am	pm	lane1	lane2	lane3	A	B
1	3.01	3.32	2.93		3.34	0.31	-0.41
2	2.55	2.83				0.28	
3			2.51		2.18		0.34
4			2.43	2.00			0.43
5	1.98	2.11	2.33		1.86	0.13	0.46
6			2.25	1.90			0.35
7			2.15		1.74		0.41
8			2.09	1.75			0.34
mean type of average							
	am	pm	lane1	lane2	lane3	A	B
2			2.98	2.63			0.35
3			2.63		2.30		0.33
4			2.53	2.06			0.47
5	2.08	2.21	2.37	1.98		0.13	0.40
6			2.32	2.01			0.31
8			2.23	1.91			0.32

mean headways. As noted earlier, input data for this site consisted of queue positions one to seven, with four of them being affected by factor A (time of day) when working with median headway. Similar results were found for mean headway, but factor B also affecting one of the four queue positions. Again, all of this is indicated in both tables. Table 5.2 shows these effects in terms of headway differences, in seconds, both for median and mean headways, where only the significant differences are shown. As for the median headway, the effect of factor

A on queue position four was highly significant, with a headway difference of 0.38 second; other three queue positions with significant effect from factor A were: number two with a 0.48 second difference, number three with a 0.28 second difference, and number six with a 0.28 second difference as well.

Table 5.2-- Site 3, A and B effects on headways

	median type of average						
Queue	A effect, headways in sec		B effect, headways in sec			Factor diff, sec	
position	am	pm	lane1	lane2	lane3	A	B
2	2.48	2.97				0.48	
3	2.21	2.49				0.28	
4	1.97	2.35				0.38	
6	1.85	2.13				0.28	
	mean type of average						
Queue	Headways in sec		Headways in sec			Factor diff, sec	
position	am	pm	lane1	lane2	lane3	A	B
1	3.26	3.62	3.16		3.65	0.36	0.49
3	2.31	2.67				0.37	
4	2.11	2.44				0.33	
5	2.16	2.38				0.23	

Examining mean headways in Table 5.2, factor A affected mean headway of four positions, three of them with a highly significant effect: position three with a difference of 0.37 second, position four with a difference of 0.33 second, and position five with a difference of 0.23 second; factor A also affected position one with a difference of 0.36 second. The lane use factor, or factor B, had a significant effect only in queue position number one, with a headway difference

between lane 1 and lane 3 of 0.49 second. Table 5.3 gathers for the three sites the grand mean headway values by queue position that resulted from the ANOVA analysis.

Table 5.3-- Grand mean headway values for every site, in seconds, from the ANOVA analysis

		Site 1		Site 2		Site 3	
		median	mean	median	mean	median	mean
	1	3.3	3.4	3.2	3.3	3.4	3.4
	2	2.6	2.7	2.7	2.8	2.7	2.8
	3	2.2	2.3	2.3	2.4	2.3	2.5
	4	2.1	2.2	2.2	2.3	2.2	2.3
Queue position	5	2.0	2.0	2.0	2.1	2.1	2.2
	6	2.0	2.0	2.0	2.1	2.0	2.1
	7	2.0	2.0	1.9	2.0	2.1	2.2
	8	1.8	1.9	1.9	2.0		
	9	1.9	2.0	1.9	2.0		
	10	1.7	1.9				
	11	1.7	1.8				

The grand mean includes the effects of the factors, peak period and lane. Two columns are under every site, one for the mean headway, and the other for the median headway. Using the data in Table 5.3, Figure 5.7 compares the grand mean results for Site 1; using mean headways produces higher grand mean headway values than those produced when median headways are used.
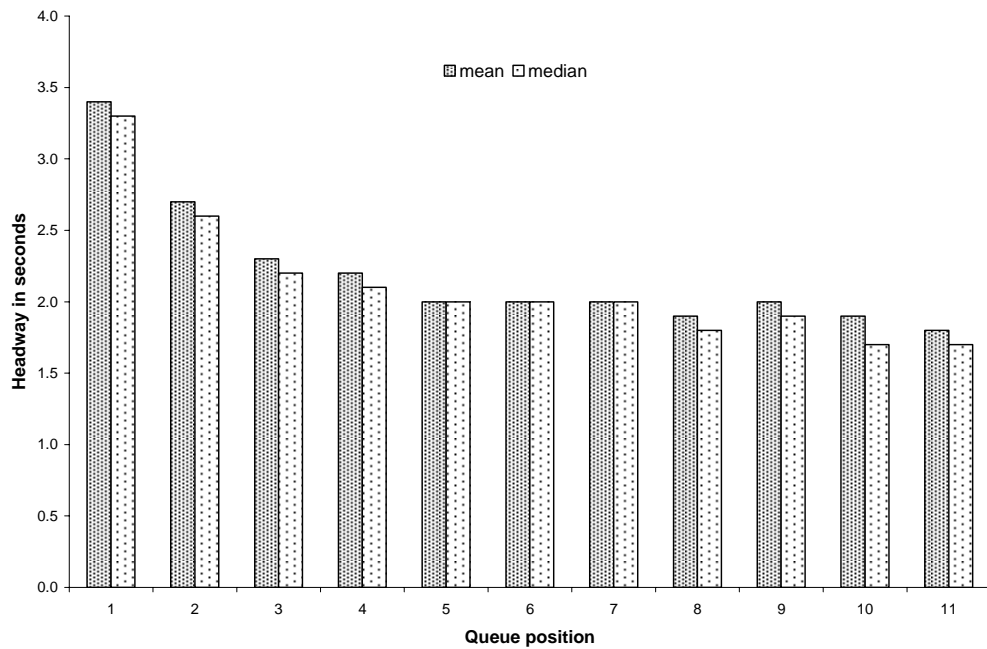
Figure 5.7. Site 1 grand mean comparison

Figures 5.8 and 5.9 on the next page illustrate the same information respectively for Sites 2 and 3. At Site 2, every queue position shows that using mean headways in the input data results in grand mean headway values higher to those that result from using median headways. For site 3 mean headways were higher than median headways for every queue position.
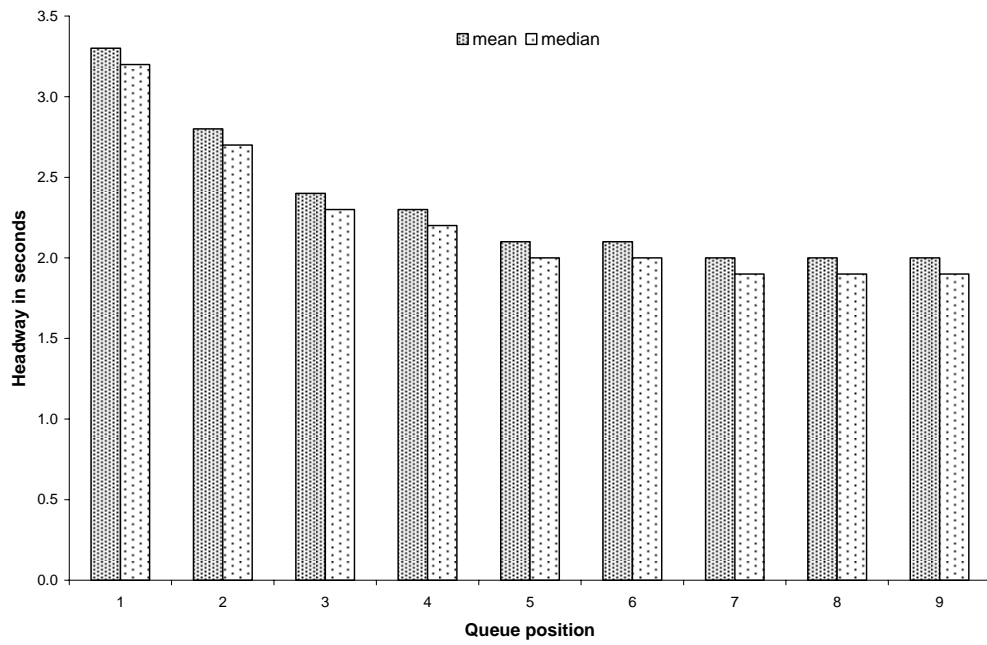
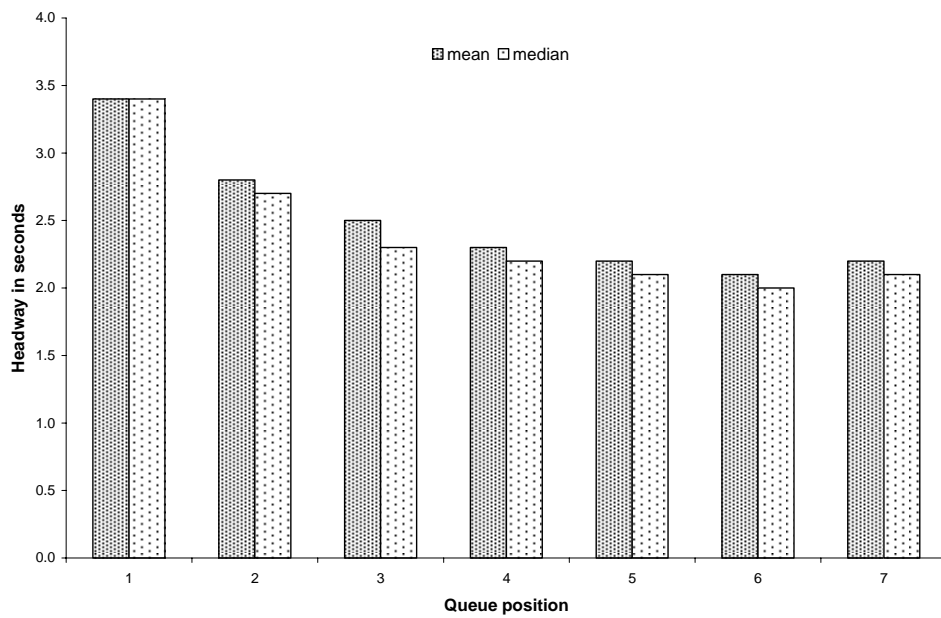Figure 5.8. Site 2 grand mean comparison.

Figure 5.9. Site 3 grand mean comparison.

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This chapter analyzed the general results of the study, compared two methods for headway calcualtion and assessed the effect of time of day (factor A) and lane number (factor B). The conclusions and recommendations of this study are presented in the next chapter.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

This last chapter has four sections. Section 1 presents the general results of the study. Section two presents two procedures to estimate the headway values. Section three recommends other possible statistical analysis to the data set obtained from this study, and finally section four briefly touches on a few mixed aspects of the study.

6.1   General results

Based on its sample size, the intersection of Garza Sada Ave. and Luis Elizondo St. (Site 1) is the most reliable, representative site of this research for estimated h, with an overall mean value of 2.1 seconds. Given its marked central tendency, this estimated overall mean h was derived from individual headway values throughout every cycle, for every queue position; using the median average rather than the mean average. Lastly an arithmetic mean was computed for the median headways found for each condition. While the 2.1 seconds coincides with the value estimated by Greenshields in the late 1940's for h; from this study, leveling off of h taking place at the fourth queue position, in contrast to the sixth queue

position also found by Greenshields almost 60 years ago. Such a difference leads in turn to a difference in start-up lost time value, 1.9 seconds vs. 3.7 seconds. Variability is another aspect of headway that proves to be consistent in this study. Overall mean h varies up to 0.2 seconds between Site 1 and Site 3. As can be seen in Tables A.74 and A.76, the difference increased to 0.7 seconds when comparing PM peak, lane 2 mean h of 2.0 seconds for Site 1, and mean h of 2.7 seconds for Site 3, for the same peak period and lane number. A quick inspection of Table A.77 mentioned in Chapter 5, also supports the h variability perception.

The lane used by the drivers was shown to affect headway as well, since from Site 1 the headway for the left turn only lane, the PM peak period resulted 0.2 seconds higher than that of the adjacent through only lane (Table A.74). This difference increased to 0.3 seconds in Site 2 (Table A.75), since headway for the U-turn only lane, at the AM peak period was 2.4 seconds compared to the headway of the adjacent through only lane of 2.1 seconds.

The peak hour effect was smaller, since within every site, the greatest mean headway difference between AM and PM peak periods was only 0.3 second (Tables A.74 through A.76).

In summary, since overall the mean h varied from 2.1 to 2.3 seconds depending on the site, no particular h value could be recommended as representative of Mexican, Monterrey conditions.

6.2 Methods to estimate *h* values

It is also important to accentuate that the method nowadays commonly used to calculate *h* tends to underestimate its value. Such a method excludes the headway of the first 5 queue positions and uses the mean average as headway for the remaining queue positions. For instance, when the method was applied to data from Site 1, the results were a low estimate of *h* value, and a high derived *sult* value. There was up to a 0.3 second difference of the *h* value, and for the *sult* value the highest difference was 1.7 seconds, as presented in Chapter 5. It is clear that the source for such a difference comes from the dissimilar queue position at which *h* levels off.

Although it might only be a coincidence, the almost same *h* value estimated by the commonly method applied to field data from Site 1 (1.9 seconds, from Chapter 5), as compared to the one López estimated in 1998 (1.92 seconds, referred on Chapter 2) might be an indication of uniformity of driver behavior in Monterrey. However, this does not mean to make a conclusion that the common method of discarding first 5 headways should be applied to Monterrey or other Mexican cities. The recommendation is to follow a procedure that allows for a clear determination of the queue position at which *h* levels off and of the derived start-up lost time, and to use the median headway instead of the mean headway of all the individual headway values in every valid cycle (i.e. a cycle with a minimum of five queued vehicles). The need of a defined procedure for study

method is evident in the Lopez's study. The use of the median headway instead of the mean headway is to eliminate the influence of extreme values.Also, given the *h* and *sult* differences just mentioned, the method most commonly used to estimate *h* does not seem to apply to Mexican conditions in Monterrey. It is rather recommended, whenever feasible to conduct the study using a method that allows for collection of headways from the greatest possible amount of valid queues (a minimum of 5 queued vehicles), thus being feasible to estimate the queue position at which *h* levels off, and the derived *sult*.

6.3   Factors' effects

Some conclusions can be drawn from the ANOVA results, with the effect of factors in terms of difference in headway values. In Site 1, when the median average was used as part of the input data, the peak period (or factor A) showed a highly significant effect only on queue position one. In Site 2, the effect was also highly significant on queue positions one, two and five. Finally for Site 3, the effect of factor A was highly significant for queue position four, and significant for queue positions two, three and six.

Still referring to the median average headway as part of the input data, lane from which data was collected (or factor B) did not affect headways at Site 1, but did affect headways at seven and four queue positions at Sites 2 and 3, respectively. In Site 2, the lane use effect was highly significant on two out of

seven queue positions, and on one out of four queue positions in Site 3. No AB interaction resulted at any of these sites with median type of average as the input statistic.

When mean headway was part of the input data to run SPSS, at Site 1, factor A affected queue position one again, in addition of also affecting queue position 4, causing a significant effect. At site 2, factor A affected only queue position four with a significant effect. And finally for Site 3, the peak period factor caused a significant effect on queue position one, only.

Regarding lane use factor mean headways, at Site 1, this factor affected queue position one only, causing a significant effect. This factor had a highly significant effect on queue positions four, five and six at Site 2, and a significant effect for queue positions two, three and eight. At Site 3, lane factor had a significant effect only on queue position one, between lanes 1 and 3. No AB interaction resulted at any of the sites with mean headway used as part of the input data.

One conclusion from the above discussion refers to the marked central tendency of the headway. At Site 1, for median headway, the ANOVA analysis reported the peak period having a highly significant effect on queue position one only, and no lane use effect at all. For the same site, however for the mean headway, the ANOVA analysis reported that factor A affected queue position one, as did factor B, but factor A also affected queue position four, thus maybe

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reflecting an extreme headway value present in the mean statistic. Nevertheless, it is necessary to remember that lane use is different at every site. At Site 1 for example, lane 1 is left turn only, while lanes 2 and 3 are through lanes, and therefore a headway difference between lanes 1 and 2, or between lanes 1 and 3 could reasonably be expected.

Again referring only to Site 1, why did no lane use effect result when the median headway was used? Maybe the answer lies on the land use abutting this intersection. As earlier mentioned, a large private university occupies a huge block in the northeast corner, with drivers in lane 1 making a left turn to enter this site.

Video tapings were made during school season, and a fairly high percentage of lane 1 drivers were students and teachers alike hurrying to class, thus causing a more aggressive reaction (a reduced reaction time) to a signal change from red to green, and reaching similar or higher speeds than those of drivers on lanes 2 and 3. All the same, lane use factor not only affected queue position one when mean headway was part of the input data to run SPSS, but also the effect was counter intuitive, contrary to what could be expected, i.e., lane 1 headway was lower than lane 3 headway; furthermore, peak period factor also affected headway of queue position four, very probably due to an extreme value.

The previous paragraphs dealt with lane use for Site 1 as related to either using median or mean headway. Now, in regards to Site 2, the signalized

intersection Garza Sada with Vía Alcalá Ave. is located in a pure shopping area, with lane 1 being used for U-turn only, and no motivation for drivers to be on time to school. This site did not have any special motivations for the headways between lanes 1 and 2, or between lanes 1 and 3 to be the same. As mentioned in chapter 5, the likely impeding effect of phase sequence on other than lane 1 drivers at first queue position, when using median headway as part of the input data to run SPSS, as an explanation for the lane 3 headway values resulting higher than the lane 1 headway values.

Interestingly enough, when using the mean headway it happened quite the opposite. Lane 2 headway was significantly lower than lane 1 headway. Did headway variability or the marked central tendency of headway caused this? Either one, or simultaneously both were the reason for this to happen.

The signalized intersection of Lázaro Cárdenas Ave. at Paseo de las Fuentes Ave. is the only site with three through lanes, even though there is a possibility of using lane 3 to exit the main lanes. Here again, using mean instead of median headway, resulted in a significant difference between lanes 1 and 3, for queue position one.

Out of six queue positions, factor A (peak hour) showed significant effect in three queue positions, and a highly significant effect in one of them. This effect was present either using median or mean headways. Using mean headways, lane number significantly affected the headway of queue position one,

stressing once again the recommendation of using the median instead of the mean headway.

6.4   Miscellaneous

The method used for field data collection is important. Videotaping proved to have a number of advantages versus traditional stopwatch, one of them being the creation of a permanent file available to review particular pieces of information, including the accuracy of the readings. In fact, videos will be made available to future students in the graduate program offered by the UANL in Monterrey, México and to students from other universities as well.

The attempt was made to take advantage of permanent video cameras that collect information for the Monterrey metropolitan area traffic management center, known as SINTRAM. A number of technical and coordination problems prevented doing this. Should such problems be resolved, this method would allow for a more extensive data set to be collected; it could even be possible to collect information under saturated conditions, which might result in headways rather different to those of non-saturated conditions as reported by Lin (Lin, 2000).

The conspicuousness of the data collection team is likely an important factor. It seems reasonable that a cleary visible team will have an impact on

drivers reaction to the traffic signal. Therefore, inconspicuousness is an important element which should be considered for all studies.

Finally, given the crucial role of this parameter in the assessing of signalized intersections and in traffic signal timing design, it is clear that more research is necessary for local conditions (Monterrey, México), since the spectrum of variable combinations is very wide.

APPENDIX

A.   TABLES WITH READINGS, HEADWAYS AND OTHER

INFORMATION

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BIOGRAPHICAL INFORMATION

Luis Francisco Chapa González received his Bachelor of Science in Civil Engineering from Universidad Autónoma de Nuevo León (UANL), Monterrey, México in 1976. In 1978 he completed studies of a graduate program in Traffic Engineering, in the same University. He has been a member of the faculty at UANL since 1982, and has held public positions at the municipal, state and federal levels from 1979 to 1995. He received a Master of Science degree in Civil Engineering from The University of Texas at Austin in May 1992. He worked as a consultant from 1996 to 1999, and in fall of 2000 he started his PhD program at The Univeristy of Texas at Arlington. He was the acting president of the student chapter at UTA of the Texas Section of the Institute of Transportation Engineers (TexITE), and received the TexITE Outstanding Student Award in 2001. Luis Francisco Chapa González received his Doctor of Philosophy degree in Civil Engineering from The University of Texas at Arlington in May 2006.