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COMPARATIVE ANALYSIS OF LEADING AND LAGGING LEFT TURNS

Final Report

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16. Abstract

This project was undertaken to compare operational and safety characteristics of leading verses lagging protected left turn operation. The measures of effectiveness included field measured intersection delay with leading and with lagging left turns. Intersections in Pima County, Glendale, Tempe and Mesa, Arizona were studied with both leading and lagging operation. Intersection delay studies were also done with first car verses third car actuation of leading protected left turn phases. Signal progression was studied with leading, lagging and combination of leading and lagging left turns which provided the best progression. This evaluation was accomplished with an instrumented vehicle and travel time runs throughout a grid.

Accident studies were conducted in Tucson, Pima County and Scottsdale, Arizona. These studies compared accident frequency before and after the conversion from leading to lagging left turns.

A public opinion survey was conducted to obtain motorists' preference of leading or lagging left turns.

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COMPARATIVE ANALYSIS OF LEADING AND LAGGING LEFT TURNS

INTRODUCTION

In 1985, the City of Tucson initiated an effort to convert the protected left turn signal phases from a leading to lagging operation. It was believed that the use of lagging left turns would improve intersection operations and network flows. The limited studies following the conversion from leading to lagging left turn operations suggested that operational and safety advantages were realized.

Pima County has a number of signalized intersections in proximity to the City of Tucson. In order to provide uniformity in the area, Pima County converted from leading to lagging left turn operations in 1987.

Based on the reported Tucson experience, other jurisdictions in Arizona began to consider changing to the lagging left turn phasing. Scottsdale, which is in the Phoenix metropolitan area, converted their protected left turn phasing to a lagging operation in early 1989.

The evaluation studies by the City of Tucson were rather limited, and there were a number of questions about the operational and safety aspects of the protected left turn phasing alternatives. In an effort to address these questions, a research project was initiated in 1989 by the Arizona Department of Transportation for the purpose of comparing the leading and lagging left turn operations. More specifically, the study was to investigate the following four basic research questions:

1) Is there a difference in intersection delay at isolated intersections between the leading and lagging left turn operation?

2) Is there a difference in signal progression among leading only, lagging only, and mixed left turn operations?

3) Is there a difference in accident experience between leading and lagging left turn operations?

4) Is there a motorist preference between leading and lagging operation?

FIELD STUDIES AND ANALYSIS

In order to address the research questions, field studies were undertaken in the Phoenix and Tucson metropolitan areas. The Phoenix area portion of the research focused primarily on the effects of the various left turn phasing patterns on signal coordination and system behavior. Also, the Phoenix area research evaluated the effects of left turn phasing on intersection delay and accidents. Finally, a public opinion poll was performed to obtain information regarding possible motorist preferences for the leading or lagging left turn phasing.

The Tucson area portion of the research project examined the accident experience resulting from the conversion of left turn operations at intersections in the City of Tucson and Pima County. In addition, the study compared traffic operations before and after the left turn conversion at selected isolated signalized intersections in Pima County.

Accident Studies

The City of Scottsdale, in 1988, undertook a six month trial period of lagging left turn operation. Five intersections were converted from leading to lagging operations in June 1988. Based on the trial period experience, the City of Scottsdale converted an additional 45 signals to a lagging operation in the early part of 1989. Due to the brief history of the use of lagging left turns, it was necessary to compare a one year accident experience with the lagging left turns with three years of accident experience with the leading left turns. It is recognized that a multiple year after period would be more desirable due to the random nature of accidents and the multitude of factors which may influence accident patterns. For this reason, the statistical test which was used in the analysis makes use of a control group that serves to discount the influence of extraneous factors and helps to identify general trends in accidents. Of the 50 intersections in the City of Scottsdale that were converted from leading to lagging left turns, nine intersections were selected for the accident study. The accident experience during the before and after periods were compared with a control group consisting of two phase signalized intersections in the City of Scottsdale.

The conversion from leading to lagging left turn operation at signalized intersections under the control of Pima County occurred in 1987. While 37 intersections were involved in the conversion program, some of the intersections were not suitable for accident analysis due to other changes; thus only 21 of the intersections were included. Because some of the approaches at these intersections did not have protected left turn movements, the accident analysis was accomplished by approach. In this way, only the approaches that were affected by the conversion from leading to lagging left turns were analyzed. The analysis period consisted of two years prior to the conversion of the signal operations and approximately two years following the change over. At a few intersections, the duration of the before period was less than two years due to the date of signal installation. A "before and after" analysis technique was used where the accident experience at each intersection approach was compared.

In the City of Tucson, the conversion from leading to lagging left turns was accomplished in 1985. Again, a "before and after" comparison of the accident experience at individual intersections was undertaken. Data for a before period from 1982 to 1984 and an after period from 1986 to 1987 were provided by the City from computerized accident reports. A total of 62 intersections were included in the analysis of which 50 were the intersection of major arterial streets. The remaining 12 intersections involved the intersection of major arterial streets with collector streets.

Travel Time Studies

In an effort to assess the effect of the left turn phasing alternatives on a system of signalized intersections, travel time studies were conducted in three cities in the Phoenix area. Alternative left turn phasing patterns were tested using travel time data along five routes in Glendale and four routes in Tempe. Four Glendale intersections and two Tempe intersections were changed from leading to lagging operation. The patterns tested were a) all leading left turns, b) all lagging left turns, and c) a combination of leading or lagging left turns depending on which best fit the progression along the route. In addition, a combination phasing was tested along one route in Mesa.

In order to obtain a true comparison between leading and lagging left turns, it was necessary to use signal timing patterns developed by a common optimization program. Because of the ease of operation and the numerous runs that would be required as part of the combination portion of the study, the computer program known as FORCAST was utilized to determine the optimal signal timings.

Intersection Studies

Intersection stopped time delay studies were conducted at six Phoenix area locations to perform a comparative analysis of leading and lagging left turn operations. Manual stopped time delay studies were conducted at each intersection prior to changing from leading to lagging left turns. Five of the six intersections operated with protected/permissive left turn phasing on all approaches. The sixth intersection operated with protected only left turns on the northbound and southbound approaches and protected/permissive left turn phasing on the other two approaches. In addition, studies comparing leading left turns with a combination leading/lagging operation were conducted at one intersection in Mesa.

At some intersections in the Phoenix area, third car actuation is used on approaches with protected/permitted left turns. With this operation, the protected left turn phase will not occur unless three or more vehicles are queued in the left turn lane. As part of the research project, delay studies were undertaken for the purpose of comparing the third car verses the first car actuation. This particular part of the research evaluated only the leading left turn condition.

Prior to the conversion from the leading to lagging left turn operation, a limited number of intersections under the control of Pima County were selected for a "before and after" comparison of the effect of the change. Nine intersections were filmed with two time-lapse cameras from approximately 3:00 p.m. to 6:00 p.m. Following the change to lagging left turns, the intersections were again filmed during the same time periods. Due to difficulties at two intersections, only seven were included in the final detailed analysis. Using the film record of the intersections, data which represented pertinent operational parameters were extracted. For example, the measures included cycle length, stopped delay, and volumes.

In contrast to the intersections in the Phoenix area, all of the Pima County intersections were isolated and operating with actuated control. Because the intersections were in the outlying areas of Tucson, the traffic volumes were not equally distributed in terms of the opposing movements. There were very few cycles in which the approaching traffic failed to clear the intersection; thus the intersections were generally not operating at saturated conditions.

RESULTS AND CONCLUSIONS

Based on the field studies, it was found that intersection delay is significantly greater with the lagging left turn operation. No significant change in total delay was found with third car actuation of leading protected left turns. In addition, no significant differences were found in progression between the leading, lagging, and mixed operations. In terms of the accident experience, no significant differences were found between the leading and lagging left turns. Finally, there was a mixed response from the motorist preference survey. Glendale drivers felt that leading left turns were better while Tempe drivers preferred the lagging left turns.

More specifically, the following results were found with respect to each of the questions posed for the research project:

1. *Is there a difference in intersection delay at isolated intersections between the leading and lagging left turn operation?*

   The results of this study indicate significantly greater delay per approach vehicle occurs with lagging operation than with leading operation for the intersections and time period tested. It is important to note that the time periods tested were generally PM peak hour conditions. These would not be as likely to have sufficiently low left turn and through volumes to eliminate many protected left turn phases in the lagging condition. It is conceivable that in off peak conditions more of the left turns could be made in a permissive manner therefore skipping the protected left turn phase. Eliminating the protected phases would likely reduce intersection delay.

   Intersection delay was also collected for test intersections with both first car and third car actuation. Although there was no significant difference between the two, this test also was only done in the PM peak hour condition. The probable benefit of third car actuation on intersection delay is most likely in off peak conditions.

2. *Is there a difference in signal progression among leading only, lagging only, and mixed left turn operations?*

   There were no statistically significant differences in stops, delay or travel time with the different operating conditions. Additionally, the large number of signal timing optimization runs required to evaluate all combinations of leading and lagging operation makes for a cumbersome, time consuming process. The requirement that the Glendale and Tempe "mixed" operation was limited to either both leading or both lagging on the same street in order to avoid the "trap" restricted potential progression benefit. An additional limitation was that only four of eight multi-phase Glendale intersections and two of four multi-phase Tempe intersections were considered for change to lagging.

   The most promise for benefit from lagging or mixed operation was found in the Mesa study where leading left turn operation was utilized for eastbound traffic and lagging for westbound traffic in the after condition. This was the operation which provided the best east-west progression. This mixed operation was possible without the trap condition because of the use of protected only left turns.

3. *Is there a difference in accident experience between leading and lagging left turn operations?*

In all three accident studies - Tucson, Pima County and Scottsdale, there was no significant difference in left turn accident history between leading and lagging operation.

4. *Is there a motorist preference between leading and lagging operation?*

Lagging left turns seem to be more favorably received in Tempe than in Glendale. This could possibly be due to the close proximity of Tempe to Scottsdale, where lagging left turns are utilized.

Public Perception Results

More Green Lights With:

	Glendale	Tempe
Leading	38%	30%
Lagging	16%	21%
Combination	24%	27%
No Difference/No Response	22%	22%

Left Turns Better With:

	Glendale	Tempe
Leading	49%	30%
Lagging	42%	61%
No Difference/No Response	9%	10%

The field studies by the research team provided valuable insight to the understanding of the many variables which influence left turn operations. Within the scope of this study and the conditions at the study sites, it was not possible to collect data for all combinations of the pertinent variables. The results of the field studies together with a somewhat theoretical analysis or understanding yield a comprehensive assessment of leading and lagging left turn operations. In essence, a number of variables have an impact on the effectiveness of left turn alternatives at a specific site. The variables that should be considered fall into the general categories of a) signal control, b) network considerations, c) traffic characteristics, and d) driver perception.

PART I

INTRODUCTION

PART I presents an introduction to the research report, documents the literature search and discusses the research problem statement.

CHAPTER 1 gives a background to the consideration and use of a leading or lagging left turn operation. A brief summary of the research results is presented. The overall organization of the research report is also discussed.

CHAPTER 2 documents the findings of the literature search, summarizes the theoretical basis for using a leading or lagging operation and reviews actual experience with both operations. The research problem statement is presented which was used as the framework for carrying out the individual research studies. The following four research questions were addressed in the individual studies:

- *Is there a difference in intersection delay at isolated intersections between leading and lagging operation?*
- *Is there a difference in signal progression among leading only, lagging only and mixed operation?*
- *Is there a difference in accident experience between leading and lagging operation?*
- *Is there a motorist preference between leading and lagging operation?*

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CHAPTER 1

INTRODUCTION

This research project entitled "Comparative Analysis of Leading and Lagging Left Turns" was conducted by Lee Engineering, Inc. in association with Dr. Robert H. Wortman. The rationale for this association in undertaking the research is based on the ability of Lee Engineering to conduct field studies in the Phoenix area while Dr. Wortman had information and data related to conditions in the Tucson area. The combined efforts of the two groups permitted a comprehensive investigation of the problem.

Lee Engineering was responsible for data collection and analyses of intersections in the Phoenix area, and Robert Wortman conducted the work related to analyzing intersections in the Tucson area. Following the accomplishment of these studies, the research team jointly evaluated the collective findings of the research and prepared this final project report.

Probably the most controversial item in the Arizona traffic engineering community for the last few years relates to leading versus lagging left turns. In 1985, the City of Tucson, Arizona initiated an effort to convert the protected left turn signal phases from a leading to lagging operation. It was believed that the use of lagging left turns would improve intersection operations and network flows. The limited studies following the conversion from leading to lagging left turn operations suggested that operational and safety advantages were realized.

Pima County has a number of signalized intersections in proximity to the City of Tucson. In order to provide uniformity in the area, Pima County converted from leading to lagging left turn operations in 1987. Based on the Tucson experience, other jurisdictions in Arizona began to consider changing to the lagging left turn phasing. Scottsdale, which is in the Phoenix metropolitan area, in early 1989 converted their protected left turn phasing to a lagging operation. There are widespread opinions as to the benefits of each of the two methods. A tremendous amount of misinformation and misunderstood information exists. This is particularly critical when one realizes the significant actions which are being taken and considered by cities and counties within the state based on this suspect information.

It should be noted that most intersections in Arizona with a protected left turn phase also have a permitted phase which allows motorists to turn left through gaps in opposing traffic. At intersections with permitted/protected phasing, simultaneous lagging left turn arrows are used to avoid trapping motorists who have pulled into the intersection while waiting to turn.

If more complete information regarding leading and lagging left turns is available and traffic operations decisions are based on that information, the opportunity exists for reduction in automobile delay and number of stops as well as increased safety. Certainly these are worthwhile goals. The need is further enhanced when one considers the reduction in auto emissions and fuel consumption associated with such operational improvements. This research project is intended to provide additional information in this area.

One of the perceived advantages of lagging left turns in a permissive/protected operation is the possibility of eliminating some of the protected left turn phases. This would occur when the left turning vehicles find sufficient gaps during the permissive period to reduce the protected green time or skip the phase.

Third car detection has been utilized by some Arizona cities to attempt to accomplish the same omission of the protected phase in a protected/permissive (leading) operator. This technique require a vehicle actuation of a detector placed a distance back from the stop line where the third left turning vehicle would be stopped. The protected left turn phase only is called when this "third car detector' is actuated.

It is appropriate that a study be undertaken to provide a factual basis for making the determination of the type of left turn phasing needed at individual intersections within Arizona cities. Some of the questions addressed in this study include the following:

1. *Does lagging left turn operation reduce intersection delay?*

   In other experiments documented in the literature based both on measured and simulated experiments, there generally has been found no significant difference in intersection delay between the two phase set options at isolated intersections.

2. *Does lagging left turn operation provide better signal progression?*

   This study investigates the difference in number of stops and travel time along arterial streets with both leading and lagging operation.

3. *Is it necessary or desirable to have consistent phasing (either leading or lagging left turns) within any given city, urbanized area, and throughout the state of Arizona?*

   There has been a concern among Arizona traffic engineers, elected officials and citizens that the mixture of leading and lagging operation among jurisdictions created safety or operational problems for motorists.

4. *What is the effect on accidents of leading versus lagging left turns?*

   Although the City of Tucson has reported a reduction in accident rate with lagging left turn operation, it was based on only six months of after period. The Federal Highway Administration (5, p.17) reports a higher accident rate for lagging than leading left turn operation. This may be due to the previously discussed safety problem of phase overlap on permissive/protected operation.

5. *What is the motorists' perception of the leading versus lagging left turns?*

   Based on the experience reported by Tucson (2) and Scottsdale (6) there may be a motorist preference of lagging over leading operation.

RESEARCH APPROACH

This research project was divided into several subareas for analysis as follows:

- Effect of leading vs. lagging left turns on intersection delay.
- Effect of leading vs. lagging left turns on signal system progression.
- Effect of leading vs. lagging left turns on accident experience.
- Effect of third car detection actuation on intersection delay.
- Motorists preference of leading and lagging left turns.

Intersection Delay

The intersection delay study was conducted both in the Phoenix area and in Pima County. At seven intersections in Glendale, Tempe and Mesa, the intersection delay with leading left turns was compared to that with lagging left turns. At the one Mesa intersection the only after condition involved a leading left turn in one direction and a lagging left turn in the opposing direction. The delay in the Phoenix area was obtained by counting the queued vehicles at 15 second increments. The Pima County intersection delay was obtained using time lapse photography with both leading and lagging left turns at 9 locations.

Signal System Progression

The signal system progression was evaluated in Glendale, Tempe and Mesa by driving an instrumented test vehicle down each street to be evaluated for six runs in each direction. The runs were conducted for each of four conditions in Glendale and Tempe:

- Existing timing (all leading).
- Optimized all leading timing.
- Optimized all lagging timing.
- Optimized combination.

It should be mentioned that all-lagging phasing was implemented only at the four Glendale intersections and the two Tempe intersections being changed although there were more intersections being timed and evaluated which were held to leading operation.

The combination timing consisted of the best combination (from a system signal progression standpoint) of leading and lagging left turns at the four Glendale and the two Tempe intersections being changed. The theory behind this test is that one can establish the best two-way progression on three streets of a grid by fitting the east-west green into the already established north-south red. This can also be done for the third street, however when attempting to "close" the grid, frequently the bands don't properly fit. It was hypothesized that having the flexibility on one of the streets would better permit a good grid closure.

The signal progression evaluation in Mesa consisted of the following:

- Existing leading operation.
- Combination of leading left eastbound and lagging left westbound.

This combination of leading and lagging left turns had been determined to provide better progression than all leading.

Accident Experience

A before (leading) - after (lagging) accident study was conducted in Tucson, Pima County, and Scottsdale. Although there were varying periods in both conditions among the three jurisdictions, Scottsdale generally had a shorter after period than the other two. This was because lagging left turn operation had been more recently implemented in Scottsdale.

Third Car Detection Evaluation

A study of intersection delay with first car versus third car detection was conducted at 3 intersections in Phoenix and 2 intersections in Tempe. The purpose of the evaluation was based on the premise that if only one or two vehicles were desiring to make a left turn, they could do so during the permissive green period or during the clearance interval. It was hypothesized that eliminating some of the protected left turn phases should provide more green time for through vehicles thereby reducing intersection delay.

It should be noted that one of the possible advantages of third car actuation is the ability to continue to have the phase overlap capabilities associated with protected/permissive (leading) operation.

Motorists' Preference

In an attempt to determine if there was a drivers' preference for either leading or lagging left turns, a questionnaire was sent out to owners of vehicles which had been observed driving on streets being tested in Glendale and Tempe. Approximately 4500 questionnaires were mailed with about half going to drivers in each of the two cities.

ADVISORY COMMITTEE

An important part of this study involved the formation and use of an advisory committee of municipal, county and state traffic engineers within the state. The philosophy was that these are the individuals who have to operate the signal systems, therefore they should be directly involved in planning, conducting and evaluating the research operation. Advisory Committee Members are as follows:

Representative	Agency
Roger Hatton	ADOT Traffic Engineering Section
Robert Pike	ADOT Research
Al Letzkus	Maricopa County
Paul Basha	City of Scottsdale
Kenneth Shackman	Pima County DOT
Hugo Malanga	City of Glendale
James Matteson	City of Phoenix
Richard Nassi	City of Tucson
Ron Krosting	City of Mesa
Harvey Friedson	City of Tempe

ORGANIZATION OF REPORT

This research report is organized into five parts. Part I presents an introduction to the research, a summary of the literature search and discussion of the research problem statement. Part II presents the accident studies which were carried out in the City of Scottsdale, Pima County and the City of Tucson. Part III presents the results of traffic operations studies which were conducted in the Phoenix area and Pima County. Part IV presents the results of the public awareness and perception analysis conducted for the cities of Glendale and Tempe. Part V discusses the study results, theoretical analysis of leading and lagging left turns, and presents recommendations for future research work.

STUDY RESULTS

This research study yielded the following results:

- Lagging operation resulted in greater delay per approach vehicle than leading for the intersections and time periods tested.

- There was no significant difference in signal progression between all leading left turns, all lagging left turns and some streets with leading and some streets with lagging.

- Although not significant due to limited sample size, there was a notable reduction in delay and travel time on the one street tested with leading left one direction and lagging left in the opposing direction when compared to the leading condition in both directions.

- There was no significant difference in accident experience between leading and lagging operations.

- Motorists in Glendale and Tempe felt that they experienced more green lights with leading than lagging. Glendale motorists felt that left turns were better with leading operation while Tempe motorists felt left turns were better with lagging operation.

It should be noted that delay studies for leading/lagging and 3rd/1st car actuation were primarily conducted during the PM peak hour. It is possible that there might be a greater chance to skip protected left turn phases for both lagging and 3rd car actuation in an off-peak period.

CHAPTER 2

LITERATURE REVIEW

The concept of a lagging green left turn interval is not new. Neither is the question as to whether leading or lagging left turns is preferable as evidenced by the following excerpt from the 1965 version of the *Traffic Engineering Handbook* (1, p.403).

> While not exactly a third phase, the use of a leading (advance) or lagging (delayed) green may be helpful in special situations.
>
> At an intersection having a fairly heavy left-turn movement, say on Phase A eastbound, the holding of the westbound Phase A green for 5 to 10 sec after eastbound traffic receives its green could be helpful. This is known as giving a "leading" green to Phase A eastbound.
>
> On the other hand, if near the end of Phase A, an additional 5 to 10 sec is allotted to Phase A eastbound, it would be receiving a "lagging" green.
>
> The use of either one of these should be approached with extreme caution because a motorist who is receiving the shorter green might not realize it since he sees opposing traffic flowing freely. He may continue to proceed into the intersection against a red signal and may collide with a motorist making a left turn who expected that the opposing motorist would stop.
>
> Some authorities feel that the leading green is probably less hazardous than the lagging green because motorists in opposing directions would generally be starting from a stopped position.
>
> On the other hand, some authorities favor the lagging green because of a tendency for traffic standing at the Stop line in the opposing direction to start when they see the traffic having the leading green begin to move. They feel that the left-turn capacity is increased because the front left-turning vehicles have moved into the intersection during the regular green period and a greater number of vehicles are able to clear the intersection than when they are starting from the Stop line at the beginning of a leading green interval.

Potential for the lagging left turn being more hazardous as mentioned in the previous excerpt refers to what is sometimes called the "trap" of lagging left turns. When lagging left turns have been used with phase overlap (one left turn becomes lagging protected while the opposing permissive left turn terminates) an increase in accidents has been observed in some locations. The driver who is waiting in the intersection to turn left and sees all the traffic lights on the approach change to red and adjacent through traffic stop expects the opposing traffic to also be stopping. The driver waiting in the intersection to turn left either turns unknowingly into the path of opposing traffic which still has the green or gets trapped in the middle of the intersection. The phasing diagram on Figure 2-1 demonstrates how this situation could occur.

The potential problem occurs in transition from the 2-6 phase to the 2-5 phase. As the driver of the left turn permissive movement associated with phase 6 sees the yellow, he might erroneously assume phase 2 is also ending and pull out in front of phase 2 traffic still viewing a green indication.

One way to alleviate this problem where permissive lagging left turn operation is used, is to require the left turns in opposing directions to be operated simultaneously in a protected manner. This is undesirable where there is a definite imbalance in directional flow. Consider the example of considerably more left turn and through traffic in a northbound direction during a particular period of the day. If the northbound lefts cannot be accommodated during the permissive operation they

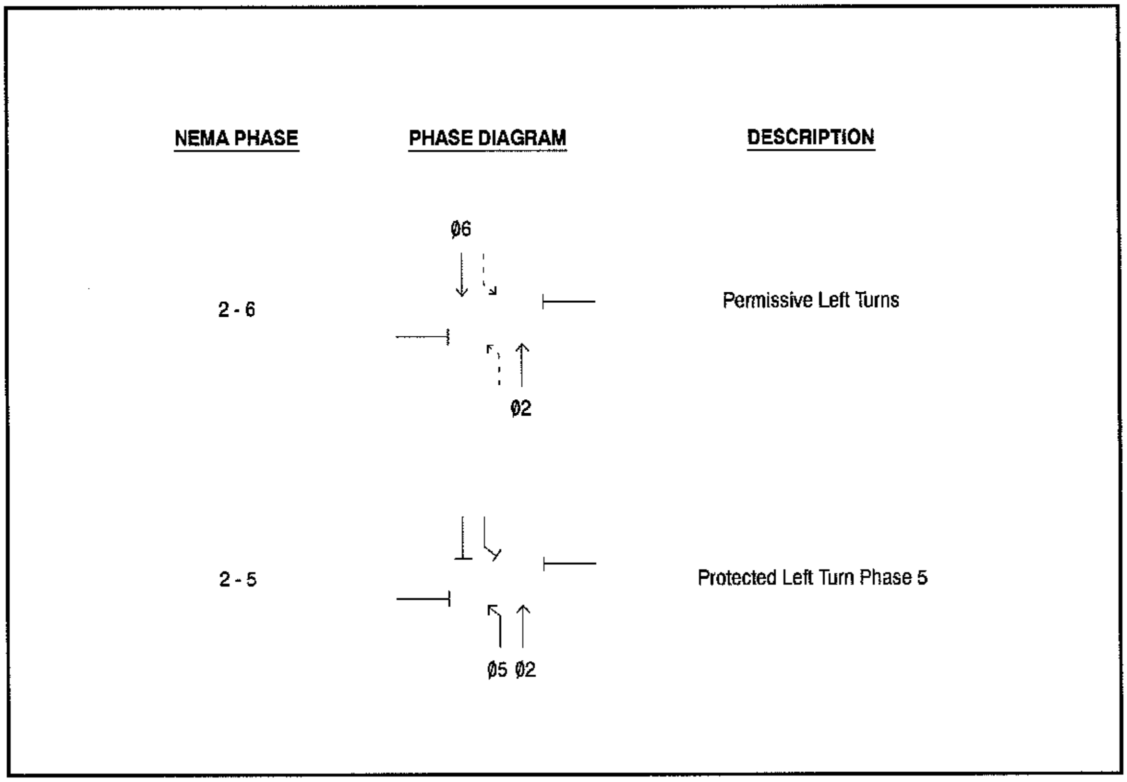

Figure 2-1. Potential Safety Problem: Lagging Left with Phase Overlap

will have to be accommodated in the protected phase. If there is little southbound left turning traffic, the heavy northbound through movement is unnecessarily delayed.

Another possibility is the use of protected only lagging left as shown in Figure 2-2. Although this looses the advantage of permissive operation, it gains the advantage of phase overlap.

In this situation there are no permissive left turning vehicles which would be expecting an opposing through termination. In this manner, where there is unbalanced flow and protected only operation, phase overlap can apparently be utilized without increased accident potential.

The simultaneous dual lag operation is utilized by the City of Tucson, which has the most experience within the state with operating lagging left turn operations. In 1984, Tucson conducted a experiment on 22nd Street from Tucson Blvd. to Kolb Road. In this study, by converting from leading to lagging operation they reported the following (2):

DELAY	-	45% reduction in off peak hours 40% reduction in peak hours
FUEL CONSUMPTION	-	reduction of 61 gallons per day per intersection
AUTO EMISSIONS	-	30% reduction in off peak hours 40% reduction in peak hours
ACCIDENT RATE	-	40% reduction

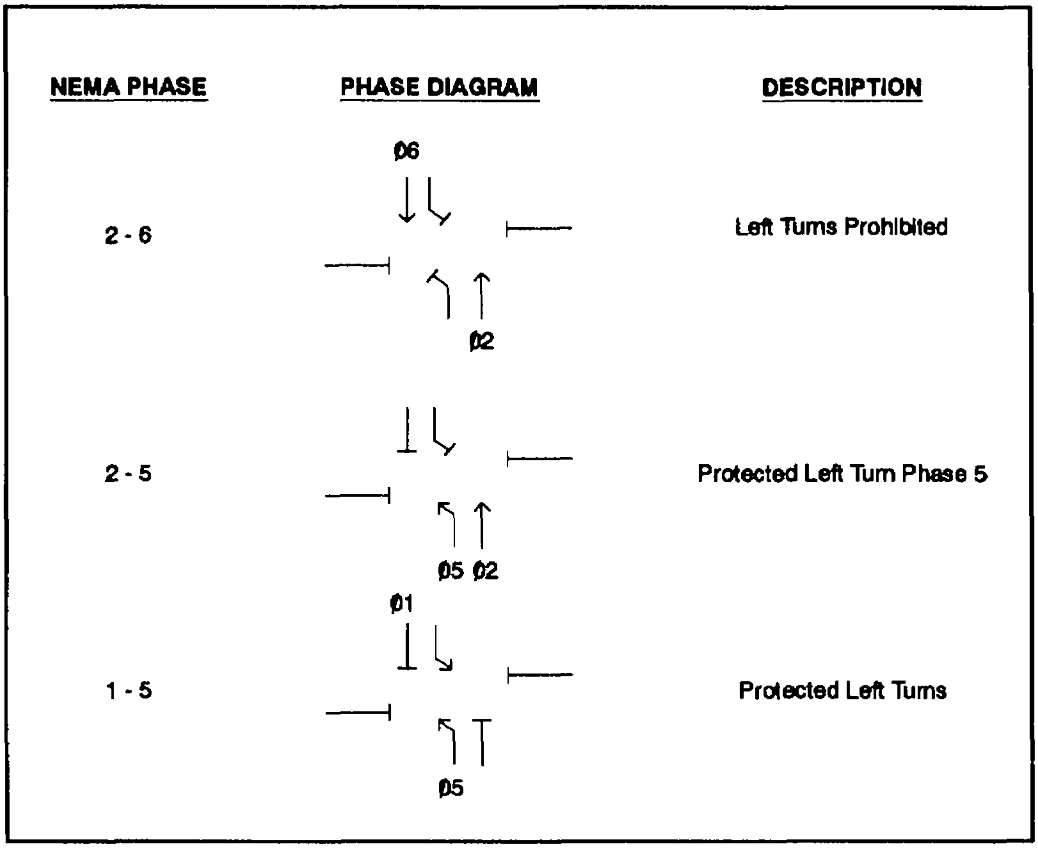

Figure 2-2. Phase Overlap with Protected Only Lagging Left

The data on delay, fuel consumption and auto emissions was based on a simulation model used by the Pima Association of Governments (3). Certainly these impressive figures merit further consideration of lagging left turn operation. The fact that this information is based on simulated data rather than field-measured data may reduce the impact of the findings, however. Additionally, there was a reported reduction in volume during the study which was apparently represented in the simulation model. This volume reduction could account for part of the delay reduction. The simulation run also considered a 10% lower cycle length with the lagging operation than the leading operation. This factor also would result in reduced delay being simulated. It should be noted that this study was based on limited data and only a six month after period.

A recent study prepared for the City of Scottsdale presented guidelines for the implementation of leading and lagging left turn phasing. It also considered the potential of third car actuated left turn phasing for protected/permissive operation. These guidelines are presented here (4, p.21):

> Guidelines for Third Car Actuated - Leading Left Turn Phasing and Permitted/Protected -Lagging Left Turn Phasing:
>
> When a traffic engineering study indicates that protected/permitted (leading) - permitted/protected (lagging) phasing is appropriate, a second decision is needed regarding lead versus lag. A basic conclusion of this study and premise for the following guidelines is that all leading-protected/

permitted operations should be third car actuated. This will reduce delay by enhancing intersection capacity and arterial travel speed. Recommended guidelines are:

1. Use Leading Protected/Permitted left turn phasing when:

   - intersection is isolated and fully actuated

   - intersection is in a coordinated system and leading left turns will enhance progressive movements, or leading left turns will provide for more efficient traffic flow than lagging.

2. Use Lagging-Permitted/Protected Left Turn Phasing when:

   - intersection is in a coordinated system

   - a capacity study and system operational analysis indicates that simultaneous lagging phasing offers operational benefits when compared to leading left turn phasing. (In some systems a combination of lead-lag phasing may be appropriate). There are several signal system simulation programs that can be used to generate key traffic operations "data" for use in the decision process. Specific data which should be considered are intersection capacity, level of service, delay, average speed, fuel consumption and emissions. Care should be taken to evaluate the left turn lane length requirements so as to avoid left turn traffic backing up into the through lanes. Such a backup can offset the other benefits of lagging left phasing.

It should be noted that at locations with protected left turns, lagging left phasing may offer operational benefits in comparison with leading left turn phasing. The guidelines discussed above should be followed in evaluating lead versus lag turns in a protected system.

Last, but not least, the City should develop a public information program including media notifications and temporary signing in conjunction with the implementation of lagging left turn-permitted/protected phasing.

The City of Scottsdale (5) reported a reduction in delay after going to lagging operation in 5 of 6 tests. This was based on AM peak, noon peak, and PM peak for both eastbound and westbound direction and is summarized in Table 2-1 (6, p.3). The city's study was based on five intersections along Thomas Road. Four of these five intersections were protected/permissive in the leading condition and were permissive/protected in the lagging condition. One intersection was protected only in the leading condition and permissive/protected in the lagging. All five intersections had phase overlap for eastbound and westbound approaches in the leading condition and simultaneous lagging arrows in the after condition. This study was based on travel time and delay data. The lagging operation

Table 2-1. City of Scottsdale Delay Study

Period and Direction	Delay Time, Seconds: With Leading Arrows	Delay Time, Seconds: With Lagging Arrows	Difference
AM Peak - Eastbound	46	26	-20
AM Peak - Westbound	14	8	-6
Mid Peak - Eastbound	15	70	55
Mid Peak - Westbound	44	11	-33
PM Peak - Eastbound	88	59	-29
PM Peak - Westbound	53	30	-23

was a best fit offset selection into the previously developed signal plans. The change in delay could possibly be due to the random nature of the lagging left turn fitting into the time space diagram. The system progression was primarily determined by other streets with a higher priority (e.g. Scottsdale Road and Hayden Road). The research team on this current project performed the signal optimization study for the City of Scottsdale and assisted in the hand fitting of offsets on Thomas Road to optimize the operation of signals there within the previously determined signal timing patterns.

The number of intersection accidents in June and July of 1987 (with leading left turn operation) was compared with the same period in 1988 after lagging operation was implemented. The results are shown in Table 2-2 (6, p.2). Caution should be used in drawing conclusions from this data because of the short time period.

Table 2-2. City of Scottsdale Accident Data

Accident Type	With Leading Arrows	With Lagging Arrows
Left Turn	2	8
Rear End	7	6
Angle	2	2
Other	2	2
Total	13	18

The City of Mesa tested a leading/lagging operation on Alma School Road in 1988. The intersection of Grove and Holmes was converted from leading left both northbound and southbound to leading left in one direction and lagging in the other based on the time-space diagram.

> Dramatic improvement was seen in the noon and evening periods after implementation of the new timing plan. During the noon period, northbound travel time decreased by 106 seconds (a 52% improvement), and southbound travel time decreased by 54 seconds (a 66% improvement). During the evening period, northbound travel time decreased by 61 seconds (a 65% improvement), and the southbound travel time decreased by 85 seconds (a 54% improvement). (7)

Research by Fambro and Woods (8) which was included in the Federal Highway Administration publication *Guidelines for Signalized Left Turn Treatments* indicates lagging left operation has an accident rate of twice that of leading. This information is presented in Table 2-3 (9, p.17).

Table 2-3. Relative Left-Turn Accident Rates for Various Left-Turn Signalization Schemes

Type of Left-Turn Phasing	Relative Accident Rate
Unprotected (Permissive)	1.00
Permissive/Protected - Lagging	0.73
Protected-Leading/Permissive	0.35
Protected	0.10

It is possible this is due to the previously discussed "trap" of lagging operation where the through movements do not terminate at the same time. This document states (9, p.13):

> When selecting the type of left-turn signal phasing to install at an intersection, standardization often enters the picture. Some agencies recommend that only leading left-turn phasing be installed, while others recommend lagging left-turn phasing. In fact, there is one large city that uses two-phase (unprotected left-turn) signals exclusively. Uniformity and consistency in the type of signal phasing that is employed has inherent advantages in the area of driver expectancy. The motorists know the phasing arrangement to anticipate and can react accordingly; however, as demonstrated at actuated signals, this same group of drivers has proved adaptable to changes in signalization. Uniformity in left-turn phasing offers no proven safety benefit and in some situations does not result in the most efficient operation.

Of particular significance is the last sentence relating to the potential safety benefit of uniformity. This is particularly true when one considers the apparent perceived importance of uniformity demonstrated by the City of Tucson, Pima County, and the City of Scottsdale.

The FHWA publication also gives recommendations on phase selection. It states that there are system considerations of left turn phase selection (9, p.27):

> Signals placed in a system configuration require consideration of the effects of the left-turn phasing on the system operation.
>
> Dual Left-Turn Phasing. If the time space diagram indicates that traffic on each approach to the intersection arrives at the same time, dual left-turn phasing should be implemented.
>
> Lead-Lag Left-Turn Phasing. If the time space diagram indicates that traffic on each approach to the intersection arrives at an appreciable difference in time (10 seconds or more) lead-lag phasing should be implemented.

A copy of Table 7 of the FHWA report is included in Appendix A which summarizes the phase selection guideline consideration for left turn phases. Because of the previously discussed potential safety problem of lagging permissive, it was determined that this research project consider these overlap phases and split phases in a protected only lagging operation.

A study by Machemehl which was based on a simulation model called TEXAS investigated various left turn sequence patterns at an isolated intersection. Machemehl reported (10, p.39):

> In cases where split left-turn sequences are selected under actuated control, the question of which left-turn movement should lead a through movement green may arise. To determine whether the leading left-turn movement performs differently than the lagging movement in a split left-turn phase arrangement, 20 traffic approach demand combinations were compared for each of the two situations.
>
> The results indicate that there is no significant difference in delay to left-turning or to through vehicles when a lagging phase is used instead of a leading phase, even though the required phase lengths are very different. This is because the left-turn queue discharges more efficiently with a leading phase minimizing delay to individual vehicles, but it requires a longer phase to do so, causing a longer cycle duration and more delay at the intersection. On the other hand, because the lagging phase is shorter, the main street green signal must be longer to process the through vehicles that would be processed with the left-turn vehicles with a leading phase. Thus, there is no significant difference between leading and lagging phases with split left turns and actuated control.

The literature search does not support the current phasing practices within the state, particularly the apparent need for standardization of either leading or lagging operation within the various governmental jurisdictions. Conversely, the literature generally recommends that the decision for leading versus lagging operation be based on conditions at the specific intersection and the opportunity to provide the best progression.

Even though the literature refers to the potential safety problem when terminating one through movement but not the other when going to a lag operation, it is not recognized to be as significant a problem as the local perception. The apparent source of the local importance is the Tucson experience of several lawsuits immediately after implementation of lagging left.

PART II

ACCIDENT STUDIES

PART II documents accident studies which were performed for three Arizona jurisdictions which have converted from leading to lagging operations.

CHAPTER 3 examines the accident experience in the City of Scottsdale. The Scottsdale accident analysis is based on a before and after comparison of the number of left turn accidents at nine intersections. The analysis compares three years of leading left arrow operation with one year of lagging operation. The Scottsdale accident analysis indicated no significant difference in the number of left turn accidents between a leading and a lagging operation.

CHAPTER 4 examines the accident experience in the Tucson area. The Pima County analysis is based on a before and after comparison of 21 intersections. The analysis, in most cases, compares two years of leading operation with two years of lagging operation. The City of Tucson analysis is based on a before and after comparison of 50 major arterial intersections and 12 intersections of major arterials with collector streets. The analysis compares three years of leading operation with two years of lagging operation. The analysis of left turn accidents in Pima County and the City of Tucson indicated that there were no significant differences resulting from the conversion from leading to lagging left turn operations.

CHAPTER 3

SCOTTSDALE ACCIDENT ANALYSIS

INTRODUCTION

The City of Scottsdale, in 1988, undertook a 10 week trial period of lagging left turn operation. Five intersections were converted from leading to lagging operations in June 1988. Based on the trial period experience, the City of Scottsdale converted an additional 45 signals to a lagging operation during the early part of 1989. Due to the brief history of lagging left turn operation in the City of Scottsdale, a one year lagging experience was compared with a three year leading experience. It is recognized that a multiple year after period would be more desirable due to the random nature of accidents and the multitude of factors which may influence roadway safety. For this reason, the statistical test which was selected for the analysis makes use of a control group which serves to discount the influence of extraneous factors and helps to identify general trends in accidents apart from the changes which may be attributed to the implementation of lagging left turn operation.

SELECTED INTERSECTIONS

The intersection selection process involved the development of an appropriate list for both the test and control intersections. The goal of the analysis is to assess the change in the number of accidents strictly as a function of the conversion from leading to lagging operation apart from any changes in protected or protected/permissive left turn phasing. Therefore, the test group was selected on the basis of similar operating conditions in the before and after period. If the intersection was operated in a permissive/protected phasing during the lagging operation then the intersection must have operated in the protected/permissive mode during the three year leading operation time period in order to be included in the test group. This constraint severely restricted the number of intersections which could be used in the analysis. Of the 50 intersections in the City of Scottsdale which were converted from a leading to a lagging operation only nine met this constraint. The test intersections used in the analysis are shown in Table 3-1. Also shown is the date of conversion from leading to lagging left turn phasing and the mode of operation before and after the conversion. One test intersection (Hayden Road/McDowell) was converted to dual left turn lanes in the last half of 1987. However, the intersection was retained in the analysis because it met the primary criteria of comparable signal operation in the before and after conditions.

Similar criteria were used to develop an appropriate set of control intersections. The primary reason for using a set of control intersections is to discount extraneous factors such as changes in traffic volumes, unusually inclement weather over some period of time, changes in accident reporting, etc. The secondary reason for using a set of control intersections is to identify unusual changes in the number of accidents as the result of purely random occurrences which may not be representative of long term trends. A set of 37 two phase intersections in the City of Scottsdale were used as the control group. Intersections which had undergone major reconstruction during the study period were not included in the set of control intersections. A list of the control intersections used in the analysis is presented in Table 3-2.

Table 3-1. Accident Analysis Test Intersections, City of Scottsdale

Intersection	Before Condition	Date of Conversion	After Condition
61st Pl./Thomas Rd.	Protected/Permissive	06/01/88	Permissive/Protected
Scottsdale Rd./Thomas Rd.	Protected/Permissive	06/01/88	Permissive/Protected
Hayden Rd./McDowell Rd.	Leading Protected	01/27/89	Lagging Protected
Miller Rd./Indian School Rd.	Protected/Permissive	02/02/89	Permissive/Protected
68th St./Indian School Rd.	Leading Protected	02/07/89	Lagging Protected
Hayden Rd./Chaparral	N/S Leading Protected	02/08/89	N/S Lagging Protected
Scottsdale Rd./Camelback Rd.	E/W Leading Protected	02/09/89	E/W Lagging Protected
Hayden Rd./McDonald Rd.	Leading Protected	02/14/89	Lagging Protected
Pima Rd./Shea Blvd.	Leading Protected	02/22/89	Lagging Protected

Table 3-2. Accident Analysis Control Intersections, City of Scottsdale

Intersection	Intersection
60th Street / Thomas	Scottsdale / Pinnacle Peak
64th Street / Camelback	Civic Center / Osborn
64th Street / Cactus	74th Street / McDowell
68th Street / Oak	75th Street / Indian School
68th Street / Osborn	Miller / Mckellips
70th Street / McDowell	Miller / Chaparral
70th Street / Osborn	Miller / McDonald
70th Place / Camelback	Miller / Shea
71st Street / Camelback	77th Street / McDowell
71st Place / Shea	Hayden / Oak
Scottsdale / Roosevelt	Hayden / Jackrabbit
Scottsdale / Oak	Hayden / Indian Bend
Scottsdale / Earll	82nd Street / Indian School
Scottsdale / Fifth Avenue	Granite Reef / Thomas
Scottsdale / Fashion Square	Granite Reef / Camelback
Scottsdale / Jackrabbit	Granite Reef / Chaparral
Scottsdale / Mercer	Granite Reef / McDonald
Scottsdale / Cholla	Pima / Mountain View
Scottsdale / Sweetwater	

DATA COLLECTION

Accident data and information about each intersection was obtained from the Arizona Department of Transportation (ADOT) and the City of Scottsdale's Traffic Engineering Department. The primary source for accident data was the ADOT Accident Location Identification and Surveillance System (ALISS) data base. The summary reports generated by ALISS were manually reviewed to identify the accidents which were of interest to this study. Police accident reports were also used to supplement the data base.

The number of total intersection related accidents and the number of intersection related left turn accidents at each test intersection were recorded for each month from June 1985 to February 1990. For the purposes of this study, a left turn accident was defined as those accidents which were classified as "left turn" manner of collision or where either vehicle action was classified as "making left turn". Only those left turn accidents on the east and west approaches at the intersection of Scottsdale Road/ Camelback Road and on the north and south approaches at the intersection of Hayden Road/Chaparral were recorded for the left turn accident analysis. The other approaches at these two intersections were not considered due to incomparable conditions in the before and after periods. The recorded accident data for the test intersections is shown in Table B-1 and Table B-2 in Appendix B.

The total number of intersection related accidents at each control intersection were also recorded for each month from June 1985 to February 1990. The recorded accident data for the control intersections is shown in Table B-3 in Appendix B.

ANALYSIS

The Scottsdale accident analysis is based on a before and after comparison of total intersection accidents and left turn accidents at nine test intersections. Due to the low number of accidents at most intersections it was necessary to develop a pool of intersections rather than testing each intersection individually. Two test intersections were converted to a lagging operation as part of the six month trial period starting in June 1988. The remaining seven test intersections were converted at various times in January and February 1989. Two sets of test intersections were developed to distinguish between protected/permissive operations and protected-only operations.

The first group of test intersections are those which have converted from a protected/permissive to a permissive/protected operation and includes the two intersections which were converted as part of the 10 week trial period. The before conversion time period for Group No. 1 extends from 1 June 1985 to 31 May 1988. The after conversion time period for Group No. 1 extends from 1 March 1989 to 28 February 1990.

The second group of test intersections are those which were converted in early 1989 from a leading protected-only to a lagging protected-only operation. The before conversion time period for Group No. 2 extends from 1 January 1986 to 31 December 1988. The after conversion time period for Group No. 2 extends from 1 March 1989 to 28 February 1990. Accident data was available only through the end of February 1990 at the time of the analysis.

The 37 control intersections were pooled for comparisons with the two test intersection groups. A

Table 3-3. Accident Analysis Summary Count, City of Scottsdale

Left Turn Accidents

Group No. 1 Test Intersections	Year B3	B2	B1	A1	Key:
61st Place / Thomas	0	1	0	0	B3 = Jun 85 - May 86
Scottsdale / Thomas	12	16	16	6	B2 = Jun 86 - May 87
Miller / Indian School	7	6	5	6	B1 = Jun 87 - May 88
Total	19	23	21	12	A1 = Mar 89 - Feb 90
Control Intersections Total Accidents	230	286	269	219	

Group No. 2 Test Intersections	Year B3	B2	B1	A1	Key:
Hayden / McDowell	1	4	5	4	B3 = Jan 86 - Dec 86
68th Street / Indian School Rd.	3	1	1	2	B2 = Jan 87 - Dec 87
Scottsdale / Camelback	0	0	2	2	B1 = Jan 88 - Dec 88
Hayden / Chaparral	1	1	1	1	A1 = Mar 89 - Feb 90
Hayden / McDonald	2	0	2	2	
Pima / Shea	0	0	1	0	
Total	7	6	12	11	
Total minus Hayden / McDowell	6	2	7	7	
Control Intersections Total Accidents	260	284	237	219	

Total Intersection Accidents

Group No. 1 Test Intersections	B3	B2	B1	A1	Key:
61st Place / Thomas	0	3	1	1	B3 = Jun 85 - May 86
Scottsdale / Thomas	16	34	27	20	B2 = Jun 86 - May 87
Miller / Indian School Rd	23	16	14	12	B1 = Jun 87 - May 88
Total	39	53	42	33	A1 = Mar 89 - Feb 90
Control Intersections Total Accidents	230	286	269	219	

Group No. 2 Test Intersections	Year B3	B2	B1	A1	Key:
Hayden / McDowell	17	22	21	26	B3 = Jan 86 - Dec 86
68th Street / Indian School Rd	17	16	14	10	B2 = Jan 87 - Jan 87
Hayden / Chaparral	9	10	10	7	B1 = Jan 88 - Dec 88
Scottsdale / Camelback	14	10	16	16	A1 = Mar 89 - Feb 90
Hayden / McDonald	9	16	20	13	
Pima / Shea	5	5	13	8	
Total	71	79	94	80	
Total Minus Hayden / McDowell	54	57	73	54	
Control Intersections Total Accidents	260	284	237	219	

summary of the number of accidents during the study period for both the test and the control intersections is shown in Table 3-3.

The statistical test used in the analysis is a chi-square goodness-of-fit test. The anlysis involves two checks:

(1) a test for comparability between the control intersections and the test intersections in the before period, and

(2) a test of the effect of changing from leading to lagging left turns at the test intersections.

A cross product ratio was also calculated to measure the apparent effect of the conversion from leading to lagging left turn phasing relative to the control intersections. The chi-square test and cross product analyses are taken from a Texas Transportation Institute report: *Three Procedures for Evaluating Highway Safety Improvement Programs* by Lindsay I. Griffin, III. (11).

The test for comparability compares the test and control intersections using three years of before data. A test for comparability in the after period could not be performed due to the fact only one year of after data was available. The results of these tests are presented in Table 3-4. The calculated chi-square for Group 1 left turn accidents was 0.03 ($p=0.98$) which indicates a strong comparability with the control intersections. The calculated chi-square for Group 2 left turn accidents was 3.48 ($p=0.18$) which indicates Group 2 is not very comparable to the control group. Therefore, the use of the control group to measure the effect of the lead to lag conversion should be viewed with caution. The comparability of Group 2 minus the Hayden/McDowell intersection (Group 2A) was also evaluated. The results indicate an even weaker level of comparability ($G^2=4.15$, df=2, $p=0.13$). The calculated chi-square for Group 1 total intersection accidents was 0.59 ($p=0.75$) which again indicates good comparability between Group 1 and the control intersections. The calculated chi-square for Group 2 ($G^2=5.59$, $p=0.064$) and Group 2A ($G^2=5.81$, $p=0.056$) indicates that Group 2 total intersection accident results are not very comparable to the control intersections. Therefore, the estimate of the effect of the Group 2 lead to lag conversion relative to the control intersections should be viewed with extreme caution.

The test of treatment compares the before and after changes at the test intersections relative to the changes at the control intersections. The results of the test are presented in Table 3-5. Group 1 left turn accidents decreased 32% relative to the control intersections ($G^2=1.49$, df=1, $p=0.23$) with the conversion from leading (protected/permissive) to lagging (permissive/protected) phasing. Group 2 left turn accidents showed an increase of 57% ($G^2=1.40$, $p=0.24$) relative to the control

Table 3-4. Accident Analysis Test for Comparability, City of Scottsdale

	Left Turn Accidents		Total Accidents	
Group	Comparability Test Statistic	Level of Significance (p)	Comparability Test Statistic	Level of Significance (p)
1	0.03	0.98	0.59	0.75
2	3.48	0.18	5.59	0.064
2A	4.15	0.13	5.81	0.056

Table 3-5. Accident Analysis Test of Treatment, City of Scottsdale

Group	Left Turn Accidents Treatment Test Statistic	Left Turn Accidents Level of Significance (p)	Left Turn Accidents Change (%)	Total Accidents Treatment Test Statistic	Total Accidents Level of Significance (p)	Total Accidents Change (%)
1	1.49	0.23	-32%	0.36	0.56	-12%
2	1.40	0.24	57%	1.08	0.30	17%
2A	1.13	0.29	66%	0.07	0.79	5%

intersections. With the conversion from leading to lagging phasing the results were similar for Group 2 minus Hayden/McDowell (Group 2A) with an apparent increase of 66% (G^2=1.13, p=0.29). Group 1 total intersection accidents decreased 12% relative to the control intersections (G^2=0.36, p=0.56) with the conversion. Group 2 total intersection accidents showed an increase of 17% (G^2=1.08, p=0.30) relative to the control intersections and Group 2A a slight increase of 5% (G^2=0.07, p=0.79) with the change for leading protected to lagging protected phasing.

The significance of these Group 2 changes is questionable due to the lack of comparability between the Group 2 intersections and the control intersections. Therefore, a simple before/after test was also performed to evaluate the absolute change in the number of accidents independent of the control intersections. The results are presented in Table 3-6. The reduction in the number of accidents at the Group 1 intersections become more pronounced when evaluated independent of the control intersections. Group 1 left accidents declined 43% (p=0.12) and total intersection accidents declined 26% (p=0.18). The increase in the number of accidents at the Group 2 intersections becomes less pronounced when evaluated independent of the control intersections. Group 2 left turn accidents increased 32% (p=0.54) and total accidents show a slight increase of 7% to (p=0.66). Group 2A provided similar results except the total intersection accidents show a slight decrease of 12% (p=0.49) when the Hayden/McDowell intersection is removed from the group. The high p-values indicate a low probability of any statistically significant difference in the number of accidents at the Group 2 intersections in the before and after periods.

DISCUSSION OF RESULTS

The Scottsdale accident analysis indicates no statistically significant change at the 90% confidence level in the number of left turn accidents or in the number of total intersection related accidents, with the conversion from leading to lagging left turns. The analysis which was applied utilized a comparison group and a test for comparability to evaluate the change in number of accidents. Also, extreme care was exercised in the selection of both the test and control intersections to ensure the analysis isolated strictly on the effects of the conversion from leading to lagging without allowing any extraneous factors to either into the evaluation. The test is admittedly rigorous and the number of intersections which could be analyzed was limited. The small sample size contributed to the finding of no statistically significant change.

However, the apparent decrease for Group 1 coupled with the apparent increase in Group 2 seems to indicate that the impact of lagging operation on left turn accidents may be different for protected/ permitted and protected only intersections. The apparent decrease in accidents with the conversion from protected/permitted leading operation to a permitted/protected lagging operation indicates people may be less likely to turn across a gap in the permissive phase given the knowledge that a

Table 3-6. Accident Analysis Results, City of Scottsdale

Left Turn Accidents

Group	Leading (acc/yr)	Lagging (acc/yr)	Absolute Change (%)	Test Statistic (Z)	Level of Significance (p)	Relative Change* (%)
1	21	12	-42.9%	-1.57	0.12	-31.7%
2	8.3	11	32.0%	0.61	0.54	56.9%
2A	5	7	40.0%	0.58	0.56	66.4%

Total Intersection Accidents

Group	Leading (acc/yr)	Lagging (acc/yr)	Absolute Change (%)	Test Statistic (Z)	Level of Significance (p)	Relative Change* (%)
1	44.7	33	-26.1%	-1.32	0.18	-11.7%
2	74.7	80	7.1%	0.43	0.66	16.9%
2A	61.3	54	-12.0%	0.68	0.49	4.7%

* Change relative to the control intersections.

Group 2A is Group 2 minus the Hayden / McDowell Intersection.

protected left turn can be executed at the end of the permitted phase. This same safety advantage would not be realized with the conversion from a protected-only leading to protected-only lagging operation.

The availability of accident data did not allow for a three month driver adjustment period at all intersections. However, the lagging left turn was not completely new to City of Scottsdale drivers at the time these intersections were converted. They had just participated in a 10 week trial period of lagging operation at five locations in the City of Scottsdale. Nevertheless, further analysis should be performed as this data becomes available.

Finally, it should be noted that the number of reported accidents in the City of Scottsdale has generally declined over the past few years. This is evidenced by the general decline in the number of accidents recorded at the control intersections for the last year before conversion and the first year after conversion. The City of Scottsdale's Traffic Engineering Department is not aware of any particular changes which may have brought about this welcomed event. However, in terms of the analysis performed, this put added pressure on the limited number of test intersections analyzed to show commensurate declines.

CHAPTER 4

PIMA COUNTY/TUCSON ACCIDENT STUDIES

As part of this research project, an analysis of the accident experience in the Tucson area was undertaken. This analysis involved an examination of accidents at signalized intersections before and after the conversion from leading to lagging turns. Accident data from the City of Tucson as well as Pima County were utilized in the study; however the experience of each jurisdiction was analyzed separately. The purpose of this chapter is to document the data collection, analysis, and results of the study.

PIMA COUNTY ANALYSIS

Introduction

The conversion from leading to lagging left turn operation at signalized intersections under the control of Pima County occurred in 1987. At that time, Pima County had a total of 37 intersections which were converted. This constituted virtually all of signalized intersections under the control of Pima County. In addition to the conversion from leading to lagging left turn operation, other operational changes were made at a number of intersections. These operational changes, lack of data, or annexation by the City of Tucson necessitated the elimination of some of the intersections from the accident study. As a result, the analysis included a total of 21 intersections which are listed in Table 4-1. The type of left turn operation by approach is also shown in the table. As may be noted from the information in Table 4-1, most of the study approaches utilized protected/permitted left turn operations. A limited number of the approaches had protected only left turn movements. Two of the approaches included in the study had protected/permitted left turn operations in the before period. Changes in intersection signal operations at these two intersections resulted in protected only movements in the after period.

Data Collection

For the analysis of the Pima County signalized intersections, data and information about each of the intersections were obtained from the records of the Pima County Department of Transportation. For each of the intersections, the following data and information were obtained:

- date of conversion from leading to lagging left turn operation
- accident data for the before and after periods
- signal timing plans
- estimated traffic volumes
- Other relevant information relative to changes in design operation of the intersection.

The records maintained by the Pima County Department of Transportation yielded detailed information about the intersections. For example, it was possible to obtain collision diagrams for each intersection as well as accident summaries. In addition, the accidents could be analyzed by type and intersection approach.

Table 4-1. Number of Left-Turn Accidents, Pima County

Intersection	Direction	Number of Accidents Before	Number of Accidents After
Ajo Way / Palo Verde Rd.	Northbound	0	1
	Southbound	2	0
Alvernon Way / Irvington Rd.	Southbound	0	0
	Eastbound	0	7
Alvernon Way / Valencia Rd.	Eastbound	1	7
	Westbound	4	0
Campbell Ave. / River Rd.	Northbound (P)	1	8
	Westbound	5	6
Craycroft Ave. / River Rd.	Southbound	3	4
	Westbound	0	0
Craycroft Ave. / Sunrise Dr.	Northbound	0	0
Dodge Blvd. / River Rd.	Westbound	1	1
Dos Hombres / Tanque Verde Rd.	Eastbound	2	1
	Westbound	2	1
First Ave. / Ina Rd.	Northbound (*)	1	0
First Ave. / Orange Grove Rd.	Northbound	3	1
	Southbound	1	0
First Ave. / River Rd.	Northbound	1	3
	Southbound	3	5
Ina Rd. / La Canada Dr.	Eastbound	3	10
	Westbound	1	2
Ina Rd. / La Cholla Blvd.	Eastbound	2	2
	Westbound	4	5
Ina Rd. / Oldfather Rd.	Eastbound	0	3
Ina Rd. / Thornydale Rd.	Northbound	2	9
	Southbound	1	5
	Eastbound	2	1
	Westbound	4	0
Kolb Rd. / Valencia Rd.	Northbound (P)	0	0
	Southbound (P)	0	0
	Eastbound (P)	2	0
	Westbound (P)	0	0
La Cholla Blvd. / Orange Grove Rd.	Northbound (P)	0	0
	Southbound (P)	0	1
	Eastbound (P)	0	1
	Westbound (P)	1	0
Mission Rd. / Valencia Rd.	Eastbound	0	2
	Westbound	3	3
Orange Grove Rd. / Skyline Dr.	Northbound (P)	8	13
River Rd. / Swan Rd.	Northbound	3	6
	Westbound	0	0
Sunrise Dr. / Swan Rd.	Southbound (*)	2	0

(P) - Protected only left turns.

(*) - Protected / permitted left turns in the before period and protected left turns in the after period.

Because the actual dates of conversion from leading to lagging left turn operation were known, the before period was defined as two years preceding the conversion in 1987. In some cases, it was not possible to obtain a full two years of accident data at an intersection due to other factors such as the date of signal installation. The actual number of days in the analysis periods for each of the intersections are indicated in a later section of this report.

For the analysis, a three month driver adjustment was used; thus the after period commenced three months following the conversion at an intersection. The after period then included all accidents from three months after the conversion of the signal operation until November 30, 1989. This cut-off date was necessitated by the data collection process; however it resulted in an after period of over two years at each of the intersections.

Analysis

During the process of converting from leading to lagging left turn operation, Pima County re-evaluated the need for protected turn movements at the intersections. This resulted in left turn phases being added or deleted at many of the intersections listed in Table 4-1. In order to maintain a reasonable data base, the intersections were analyzed on the basis of the individual approaches. For example, the evaluation focused on left turn accidents on the intersection approaches where the conversion from leading to lagging operation had occurred. Intersection approaches where other changes had been made (such as the addition or deletion of a left turn phase) were not included in the final analysis. In this way, accidents that were directly associated with the leading and lagging left turn operations were isolated for comparison.

Table 4-1 indicates the number of left turn accidents that were reported during the before and after periods at each intersection. The accidents are shown by approach for each of the intersections. For this study, an accident was considered to be a left turn accident if a left turning vehicle on a given approach was involved.

As indicated previously, the initial intent was to obtain the accident records for a two year period prior to the signal operation conversion. In some cases, this was not possible. The after period for all of the intersections was greater than two years with the exception of one which had an after period of a few days less than two years. The number of days in the before and after periods of each intersection is shown in Table C-1 in Appendix C.

Using the number of reported accidents and the durations of the before and after periods, the equivalent number of accidents per year was calculated for each study approach. This information is presented in Table 4-2.

It should be noted that accidents involving bicycles were not included in the analysis. At all of the intersections, there were only two bicycle related accidents in the before period and one in the after period.

The average daily volumes for each of the intersection approaches were obtained from the Pima County Department of Transportation. The estimated average daily approach volumes for the before and after periods are given in Table 4-3. These values reflect the total volume for an intersection

Table 4-2. Equivalent Number of Accidents per Year, Pima County

Intersection	Direction	Number of Accidents Per Year Before	After	Difference
Ajo Way / Palo Verde Rd.	Northbound	0.00	0.41	0.41
	Southbound	1.00	0.00	-1.00
Alvernon Way / Irvington Rd.	Southbound	0.00	0.00	0.00
	Eastbound	0.00	2.90	2.90
Alvernon Way / Valencia Rd.	Eastbound	0.50	2.89	2.39
	Westbound	2.00	0.00	-2.00
Campbell Ave. / River Rd.	Northbound (P)	0.50	3.39	2.89
	Westbound	2.50	2.54	0.04
Craycroft Ave. / River Rd.	Southbound	1.50	1.69	0.19
	Westbound	0.00	0.00	0.00
Craycroft Ave. / Sunrise Dr.	Northbound	0.00	0.00	0.00
Dodge Blvd. / River Rd.	Westbound	0.88	0.42	-0.46
Dos Hombres / Tanque Verde Rd.	Eastbound	1.00	0.43	-0.57
	Westbound	1.00	0.43	-0.57
First Ave. / Ina Rd.	Northbound (*)	0.50	0.00	-0.50
First Ave. / Orange Grove Rd.	Northbound	1.50	0.42	-1.08
	Southbound	0.50	0.00	-0.50
First Ave. / River Rd.	Northbound	1.59	1.25	-0.34
	Southbound	4.76	2.08	-2.68
Ina Rd. / La Canada Dr.	Eastbound	1.50	4.31	2.81
	Westbound	0.50	0.86	0.36
Ina Rd. / La Cholla Blvd.	Eastbound	1.00	0.86	-0.14
	Westbound	2.00	2.16	0.16
Ina Rd. / Oldfather Rd.	Eastbound	0.00	1.33	1.33
Ina Rd. / Thornydale Rd.	Northbound	1.00	3.96	2.96
	Southbound	0.50	2.20	1.70
	Eastbound	1.00	0.44	-0.56
	Westbound	2.00	0.00	-2.00
Kolb Rd. / Valencia Rd.	Northbound (P)	0.00	0.00	0.00
	Southbound (P)	0.00	0.00	0.00
	Eastbound (P)	2.25	0.00	-2.25
	Westbound (P)	0.00	0.00	0.00
La Cholla Blvd. / Orange Grove Rd.	Northbound (P)	0.00	0.00	0.00
	Southbound (P)	0.00	0.42	0.42
	Eastbound (P)	0.00	0.42	0.42
	Westbound (P)	1.10	0.00	-1.10
Mission Rd. / Valencia Rd.	Eastbound	0.00	0.83	0.83
	Westbound	1.50	1.24	-0.26
Orange Grove Rd. / Skyline Dr.	Northbound (P)	4.00	5.51	1.51
River Rd. / Swan Rd.	Northbound	1.95	3.08	1.13
	Westbound	0.00	0.00	0.00
Sunrise Dr. / Swan Rd.	Southbound (*)	0.80	0.00	0.80

(P) - Protected only left turns

(*) - Protected / permitted left turns in the before period and protected left turns in the after period

Table 4-3. Estimated Approach Volumes, Pima County

Intersection	Direction	Approach Volumes Before (vpd)	After (vpd)
Ajo Way / Palo Verde Rd.	Northbound	14,709	12,127
	Southbound	15,210	11,376
Alvernon Way / Irvington Rd.	Southbound	7,490	13,788
	Eastbound	6,104	6,256
Alvernon Way / Valencia Rd.	Eastbound	9,624	10,703
	Westbound	10,203	7,854
Campbell Ave. / River Rd.	Northbound	12,062	13,568
	Westbound	8,078	10,040
Craycroft Ave. / River Rd.	Southbound	7,456	7,607
	Westbound	4,231	6,415
Craycroft Ave. / Sunrise Dr.	Northbound	6,450	7,856
Dodge Blvd. / River Rd.	Westbound	5,169	8,004
Dos Hombres / Tanque Verde Rd.	Eastbound	16,010	18,959
	Westbound	16,610	17,384
First Ave. / Ina Rd.	Northbound	4,763	5,547
First Ave. / Orange Grove Rd.	Northbound	8,683	9,059
	Southbound	5,231	5,268
First Ave. / River Rd.	Northbound	11,428	11,914
	Southbound	11,002	11,710
Ina Rd. / La Canada Dr.	Eastbound	12,664	15,054
	Westbound	12,460	14,488
Ina Rd. / La Cholla Blvd.	Eastbound	11,612	15,202
	Westbound	13,527	15,860
Ina Rd. / Oldfather Rd.	Eastbound	12,942	14,808
Ina Rd. / Thornydale Rd.	Northbound	6,740	8,383
	Southbound	5,804	8,414
	Eastbound	11,782	14,242
	Westbound	10,525	19,253
Kolb Rd. / Valencia Rd.	Northbound	3,972	3,970
	Southbound	9,863	12,418
	Eastbound	8,429	8,098
	Westbound	1,105	1,205
La Cholla Blvd. / Orange Grove Rd.	Northbound	8,058	7,610
	Southbound	4,219	5,815
	Eastbound	6,113	6,244
	Westbound	7,871	8,119
Mission Rd. / Valencia Rd.	Eastbound	8,695	10,475
	Westbound	11,625	12,878
Orange Grove Rd. / Skyline Dr.	Northbound	10,571	12,417
River Rd. / Swan Rd.	Northbound	9,644	11,655
	Westbound	2,372	3,548
Sunrise Dr. / Swan Rd.	Southbound	4,704	4,231

approach. While it would have been desirable to have only the left turn approach volume, this level of detail was not available.

It is recognized that the true left turn accident rate should be based on the volume of left turn vehicles entering the intersection. Because this information was not available, the accident rate was based on the total approach volume. In this way, the influence of exposure and the durations of the study periods were considered. In view of the fact that the before and after periods were separated only by a three month driver adjustment period, it would be expected that the proportion of left turning vehicles in the approach volumes would remain about the same. The accident rate (based on the total approach volume) for each of the intersection approaches is shown in Table 4-4.

A number of different statistical tests have historically been applied to accident analyses. Typically, these analyses evaluate differences in accidents or the average accident rate. In some cases, differences in accident rates for the before and after periods are compared with the experience at other intersections which were not subjected to a given treatment.

After considering the nature of the data set, it was decided to apply the Wilcoxen Signed-Ranks Test. Basically, the test examines the direction of the difference within a sample pair as well as the relative magnitude of the difference. It provides a means of analyzing the experience for each of the intersection approaches in addition to examining the collective results of the total sample.

An analysis of the total intersection accidents was also undertaken for the purpose of examining any possible effect of the change in left turn operation on the total intersection safety. In order to eliminate possible impacts of other changes at the intersections, only the accidents associated with the approaches included in the left turn study were evaluated. Appendix C contains tables which summarize the data for all accidents on the approaches considered.

Discussion of Results

The summary results of the Wilcoxen test based on accident rates are presented in Table C-2 in Appendix C. In reviewing Table C-2, it may be noted that some of the intersection approaches are not listed in the table. This is due to the fact that samples with no difference in the before and after periods are dropped from the statistical test. Only the intersection approaches for which there was a difference in the accident rate are shown in Table C-2. Similar information for the analysis of the equivalent number of accidents is given in Table C-3 in Appendix C. Also, information related to the analysis of total intersection accidents is in Appendix C.

For the analyses, the null hypothesis was that there is no difference in the accident experience for the before and after periods. At the 95 percent confidence level, the analyses indicate that the hypothesis should be accepted in both cases. In essence, the use of accident rates, the use of the equivalent number of left turn accidents, or the use of total accident data yielded the same results. The conclusion, therefore, is that there was no difference in the accident experience.

While the Wilcoxen test did not indicate statistical significance, the actual change in number of accidents was calculated for the left turn accidents and the total accidents. Based on the number of accidents per year, there was a 13.8 percent increase in left turn accidents while the total accidents decreased by 1.5 percent.

Table 4-4. Left-Turn Accident Rate, Pima County

Intersection	Direction	Accident Rate (Accidents / Million Entering Vehicles) Before	After	Difference
Ajo Way / Palo Verde Rd.	Northbound	0.000	0.094	- 0.094
	Southbound	0.180	0.000	+ 0.180
Alvernon Way / Irvington Rd.	Southbound	0.000	0.000	
	Eastbound	0.000	1.274	- 1.274
Alvernon Way / Valencia Rd.	Eastbound	0.142	0.740	- 0.598
	Westbound	0.537	0.000	+ 0.537
Campbell Ave. / River Rd.	Northbound (P)	0.114	0.683	- 0.569
	Westbound	0.848	0.693	+ 0.155
Craycroft Ave. / River Rd.	Southbound	0.551	0.611	- 0.060
	Westbound	0.000	0.000	
Craycroft Ave. / Sunrise Dr.	Northbound	0.000	0.000	
Dodge Blvd. / River Rd.	Westbound	0.464	0.145	+ 0.319
Dos Hombres / Tanque Verde Rd.	Eastbound	0.171	0.062	+ 0.109
	Westbound	0.165	0.067	+ 0.098
First Ave. / Ina Rd.	Northbound (*)	0.288	0.000	+ 0.288
First Ave. / Orange Grove Rd.	Northbound	0.473	0.629	- 0.156
	Southbound	0.262	0.000	+ 0.262
First Ave. / River Rd.	Northbound	0.379	0.287	+ 0.092
	Southbound	1.180	0.487	+ 0.693
Ina Rd. / La Canada Dr.	Eastbound	0.325	0.783	- 0.458
	Westbound	0.110	0.163	- 0.053
Ina Rd. / La Cholla Blvd.	Eastbound	0.236	0.155	+ 0.081
	Westbound	0.405	0.372	+ 0.033
Ina Rd. / Oldfather Rd.	Eastbound	0.000	0.245	- 0.245
Ina Rd. / Thornydale Rd.	Northbound	0.407	1.298	- 0.891
	Southbound	0.236	0.719	- 0.483
	Eastbound	0.233	0.085	+ 0.148
	Westbound	0.521	0.000	+ 0.521
Kolb Rd. / Valencia Rd.	Northbound (P)	0.000	0.000	
	Southbound (P)	0.000	0.000	
	Eastbound (P)	0.730	0.000	+ 0.730
	Westbound (P)	0.000	0.000	
La Cholla Blvd. / Orange Grove Rd.	Northbound (P)	0.000	0.000	
	Southbound (P)	0.000	0.197	- 0.197
	Eastbound (P)	0.000	0.184	- 0.184
	Westbound (P)	0.384	0.000	+ 0.384
Mission Rd. / Valencia Rd.	Eastbound	0.000	0.216	- 0.216
	Westbound	0.354	0.264	+ 0.090
Orange Grove Rd. / Skyline Dr.	Northbound (P)	1.037	1.213	- 0.176
River Rd. / Swan Rd.	Northbound	0.554	0.724	- 0.170
	Westbound	0.000	0.000	
Sunrise Dr. / Swan Rd.	Southbound (*)	0.466	0.000	+ 0.466

(P) - Protected only left turns

(*) - Protected / permitted left turns in the before period and protected left turns in the after period

Because the Wilcoxen test examines the experience at each intersection, the analysis was undertaken by including all of the approaches into a single group. In this way, the collective result of the conversion to lagging left turns was analyzed. It can be argued that the analysis should be accomplished by considering the approaches with protected/permitted operations separate from the approaches with protected only operations. Subsequent evaluation of the approaches separated by type of operation also indicated variation in the accident experience in both group with no statistical difference at the 95 percent confidence level.

CITY OF TUCSON ANALYSIS

Introduction

The conversion from leading to lagging left turn operation in the City of Tucson was accomplished in 1985. At that time, virtually all traffic signals in the City with protected left turn phases were converted to the lagging left turn operation. In the City of Tucson, the practice is to use permitted left turns with the protected movement.

For the evaluation of the accident experience in the City of Tucson, a "before and after" type of analysis was again used. Some of the detailed information about accidents as well as the intersections was not readily available; thus a slightly different approach was taken for the analysis.

Data Collection

The City of Tucson furnished computer summaries of intersection accident data. Because the conversion in signal operation occurred in 1985, that year was eliminated from the analysis. Data for a before period from 1982 to 1984 and an after period of 1986 to 1987 were compiled by the City from the computerized accident records. The information indicated the total accident rate for an intersection as well as the number of accidents by general types. Again, an accident was considered to be a left turn accident if a left turner was involved.

The City also sorts the intersections by type. For example, the data was compiled for the intersection of major arterial streets and for the intersection of major arterials with collector streets. Generally, signals at the intersection of major arterials will have protected left turn phases on all approaches; and signals at intersections with collector streets will have left turn phases on the major arterial approaches.

As was the case with the Pima County situation, it was necessary to screen the list of intersections for the purpose of eliminating those where other obvious changes had been made. This resulted in 50 intersections involving major arterials and 12 intersections of major arterials with collector streets being included in the study.

Analysis

For the analysis, the initial problem was to find a method for determining the left turn accident rates. The total approach volumes were available for each intersection; however the left turn volumes were unknown. Thus, it was not possible to directly determine the left turn accident rate at each of the intersections. As a surrogate measure, the left turn accident rate was calculated by multiplying the total intersection accident rate by the ratio of left turn accidents to all accidents. In essence, this resulted in a value that is based on the total left turn accidents within the intersection, the average total entering intersection volumes, and the time period over which the data were gathered. The summaries of information used for both intersection groups are shown in Tables 4-5 through 4-10.

The Wilcoxen test was also utilized to statistically evaluate the experience in the before and after periods. This analysis was accomplished for each of the two groups of intersections using accident rates as well as the number of accidents per year. The results of the statistical tests are in Tables C-4 through C-7. As was the case with the Pima County intersections, it was concluded that there was no significant difference in the before and after accident experience.

For the two categories of City of Tucson intersections, the total intersection accidents were also compiled. This compilation included an examination of all reported accidents at each of the intersections. The summary tables indicating the total intersection accident experience are included in Appendix C. Again, the analysis of all accidents did not indicate any statistical difference in total accident experience.

Discussion of Results

The analysis of left turn accidents in Pima County and the City of Tucson indicated that there were no significant differences resulting from the conversion from leading to lagging left turn operations. This finding is somewhat contrary to the comments in the literature that indicate that the lagging left turn operation results in a more hazardous condition.

The examination of the change in the number of accidents per year yielded the following results for the before and after periods:

- At the intersection of major arterial streets, the number of left turn accidents decreased by 2.8 percent while all accidents decreased by 6.1 percent.
- For the group of intersections involving major arterials and collector streets, the left turn accidents decrease by 11.3 percent and all accidents decreased by 17.8 percent.

Certainly, a review of the accident experience at the individual intersections reveals considerable variation in the results. At some intersections, there were large increases or decreases in accidents. There was nothing in the available information about the intersections that would explain these variations other than the random nature of accidents.

It should be noted that in Pima County as well as the City of Tucson lagging protected left turn phase overlaps are not used in conjunction with permitted left turn operations. This means that the relatively hazardous "left turn trap" condition does not occur. For this reason, it is not surprising that a difference was not found between the leading and lagging conditions.

Some traffic engineers will suggest that the lagging left turn operation is safer because the motorist knows that the left turn can be made at the end of the permitted phase. In this case, the driver is not as pressured to make the permitted left turn. Generally, the City of Tucson uses permitted/protected left turn operations at intersections with protected left turn phases. Again, the data did not reveal any significant differences.

Table 4-5. Left-Turn Accidents, City of Tucson Arterial / Arterial Intersections

	Number of Left Turn Accidents	
Intersection	**1982 - 1984**	**1986 - 1987**
Ajo Way / Mission Rd.	18	13
Ajo Way / Interstate 19	0	0
Ajo Way / 12th Ave.	24	10
Alvernon Way / Broadway Blvd.	23	9
Alvernon Way / 22nd St.	111	41
Broadway Blvd. / Campbell Ave.	14	26
Broadway Blvd. / Country Club Rd.	17	14
Broadway Blvd. / Craycroft Rd.	16	8
Broadway Blvd. / Kolb Rd.	16	11
Broadway Blvd. / Swan Rd.	16	9
Broadway Blvd. / Wilmot Rd.	24	7
Campbell Ave. / Fort Lowell Rd.	35	16
Campbell Ave. / Grant Rd.	28	26
Campbell Ave. / Speedway Blvd.	31	21
Congress St. / Granada Ave.	3	3
Congress St. / Interstate 10	20	9
Country Club Rd. / Grant Rd.	11	13
Country Club Rd. / Speedway Blvd.	10	6
Country Club Rd. / Valencia Rd.	4	6
Craycroft Rd. / Golf Links Rd.	9	21
Craycroft Rd. / 22nd St.	46	32
Fort Lowell Rd. / Oracle Rd.	3	16
Golf Links Rd. / Kolb Rd.	16	16
Golf Links Rd. / Wilmot Rd.	24	18
Grant Rd. / Oracle Rd.	24	18
Grant Rd. / Stone Ave.	21	13
Grant Rd. / Swan Rd.	30	15
Grant (Kolb) Rd. / Tanque Verde Rd.	36	27
Grant Rd. / First Ave.	21	7
Grant Rd. / Interstate 10	2	13
Kolb Rd. / Speedway Blvd.	28	14
Kolb Rd. / 22nd St.	55	27
Main Ave. / Speedway Blvd.	13	5
Miracle Mile / Oracle Rd.	2	14
Nogales Highway / Valencia Rd.	28	13
Oracle Rd. / Prince Rd.	34	26
Oracle Rd. / River Rd.	5	8
Oracle Rd. / Wetmore Rd.	3	11
Speedway Blvd. / Stone Ave.	20	10
Speedway Blvd. / Swan Rd.	5	3
Speedway Blvd. / Wilmot Rd.	12	12
Speedway Blvd. / Interstate 10	9	3
St. Mary's Rd. / Interstate 10	14	3
Swan Rd. / 22nd St.	43	5
Valencia Rd. / 12th Ave.	20	12
Wetmore Rd. / First Ave.	7	5
Wilmot Rd. / 5th St.	12	9
Wilmot Rd. / 22nd St.	39	29
Interstate 10 / 22nd St.	12	5
5th Ave. / Interstate 10	5	2

Table 4-6. Average Left-Turn Accidents per Year, City of Tucson Arterial / Arterial Intersections

	Left Turn Accidents Per Year		
Intersection	**Before**	**After**	**Difference**
Ajo Way / Mission Rd.	6.00	6.50	0.50
Ajo Way / Interstate 19	0.00	0.00	0.00
Ajo Way / 12th Ave.	8.00	5.00	-3.00
Alvernon Way / Broadway Blvd.	7.67	4.50	-3.17
Alvernon Way / 22nd St.	37.00	20.50	-16.50
Broadway Blvd. / Campbell Ave.	4.67	13.00	8.33
Broadway Blvd. / Country Club Rd.	5.67	7.00	1.33
Broadway Blvd. / Craycroft Rd.	5.33	4.00	-1.33
Broadway Blvd. / Kolb Rd.	5.33	5.50	0.17
Broadway Blvd. / Swan Rd.	5.33	4.50	-0.83
Broadway Blvd. / Wilmot Rd.	8.00	3.50	-4.50
Campbell Ave. / Fort Lowell Rd.	11.67	8.00	-3.67
Campbell Ave. / Grant Rd.	9.33	13.00	3.67
Campbell Ave. / Speedway Blvd.	10.33	10.50	0.17
Congress St. / Granada Ave.	1.00	1.50	0.50
Congress St. / Interstate 10	6.67	4.50	-2.17
Country Club Rd. / Grant Rd.	3.67	6.50	2.83
Country Club Rd. / Speedway Blvd.	3.33	3.00	-0.33
Country Club Rd. / Valencia Rd.	1.33	3.00	1.67
Craycroft Rd. / Golf Links Rd.	3.00	10.50	7.50
Craycroft Rd. / 22nd St.	15.33	16.00	0.67
Fort Lowell Rd. / Oracle Rd.	1.00	8.00	7.00
Golf Links Rd. / Kolb Rd.	5.33	8.00	2.67
Golf Links Rd. / Wilmot Rd.	8.00	9.00	1.00
Grant Rd. / Oracle Rd.	8.00	9.00	1.00
Grant Rd. / Stone Ave.	7.00	6.50	-0.50
Grant Rd. / Swan Rd.	10.00	7.50	-2.50
Grant (Kolb) Rd. / Tanque Verde Rd.	12.00	13.50	1.50
Grant Rd. / First Ave.	7.00	3.50	-3.50
Grant Rd. / Interstate 10	0.67	6.50	5.83
Kolb Rd. / Speedway Blvd.	9.33	7.00	-2.33
Kolb Rd. / 22nd St.	18.33	13.50	-4.83
Main Ave. / Speedway Blvd.	4.33	2.50	-1.83
Miracle Mile / Oracle Rd.	0.67	7.00	6.33
Nogales Highway / Valencia Rd.	9.33	6.50	-2.83
Oracle Rd. / Prince Rd.	11.33	13.00	1.67
Oracle Rd. / River Rd.	1.67	4.00	2.33
Oracle Rd. / Wetmore Rd.	1.00	5.50	4.50
Speedway Blvd. / Stone Ave.	6.67	5.00	-1.67
Speedway Blvd. / Swan Rd.	1.67	1.50	-0.17
Speedway Blvd. / Wilmot Rd.	4.00	6.00	2.00
Speedway Blvd. / Interstate 10	3.00	1.50	-1.50
St. Mary's Rd. / Interstate 10	4.67	1.50	-3.17
Swan Rd. / 22nd St.	14.33	2.50	-11.83
Valencia Rd. / 12th Ave.	6.67	6.00	-0.67
Wetmore Rd. / First Ave.	2.33	2.50	0.17
Wilmot Rd. / 5th St.	4.00	4.50	0.50
Wilmot Rd. / 22nd St.	13.00	14.50	1.50
Interstate 10 / 22nd St.	4.00	2.50	-1.50
5th Ave. / Interstate 10	1.67	1.00	-0.67
Total	339.66	330.00	-9.66
Percent Change			-2.84%

Table 4-7. Left-Turn Accident Rate, City of Tucson Arterial / Arterial Intersections

Intersection	Left Turn Accident Rate * 1982 - 1984	Left Turn Accident Rate * 1986 - 1987
Ajo Way / Mission Rd.	0.453	0.485
Ajo Way / Interstate 19	0.000	0.000
Ajo Way / 12th Ave.	0.593	0.344
Alvernon Way / Broadway Blvd.	0.337	0.186
Alvernon Way / 22nd St.	1.444	0.748
Broadway Blvd. / Campbell Ave.	0.292	0.836
Broadway Blvd. / Country Club Rd.	0.330	0.382
Broadway Blvd. / Craycroft Rd.	0.258	0.203
Broadway Blvd. / Kolb Rd.	0.236	0.198
Broadway Blvd. / Swan Rd.	0.266	0.222
Broadway Blvd. / Wilmot Rd.	0.376	0.145
Campbell Ave. / Fort Lowell Rd.	0.704	0.528
Campbell Ave. / Grant Rd.	0.320	0.373
Campbell Ave. / Speedway Blvd.	0.467	0.429
Congress St. / Granada Ave.	0.098	0.134
Congress St. / Interstate 10	0.921	0.704
Country Club Rd. / Grant Rd.	0.198	0.339
Country Club Rd. / Speedway Blvd.	0.184	0.191
Country Club Rd. / Valencia Rd.	0.158	0.311
Craycroft Rd. / Golf Links Rd.	0.158	0.508
Craycroft Rd. / 22nd St.	0.660	0.712
Fort Lowell Rd. / Oracle Rd.	0.056	0.461
Golf Links Rd. / Kolb Rd.	0.294	0.439
Golf Links Rd. / Wilmot Rd.	0.557	0.629
Grant Rd. / Oracle Rd.	0.456	0.502
Grant Rd. / Stone Ave.	0.442	0.421
Grant Rd. / Swan Rd.	0.546	0.380
Grant (Kolb) Rd. / Tanque Verde Rd.	0.659	0.736
Grant Rd. / First Ave.	0.333	0.172
Grant Rd. / Interstate 10	0.047	0.464
Kolb Rd. / Speedway Blvd.	0.463	0.358
Kolb Rd. / 22nd St.	0.767	0.590
Main Ave. / Speedway Blvd.	0.332	0.227
Miracle Mile / Oracle Rd.	0.036	0.372
Nogales Highway / Valencia Rd.	0.867	0.629
Oracle Rd. / Prince Rd.	0.594	0.671
Oracle Rd. / River Rd.	0.083	0.210
Oracle Rd. / Wetmore Rd.	0.056	0.281
Speedway Blvd. / Stone Ave.	0.657	0.498
Speedway Blvd. / Swan Rd.	0.078	0.095
Speedway Blvd. / Wilmot Rd.	0.185	0.293
Speedway Blvd. / Interstate 10	0.325	0.148
St. Mary's Rd. / Interstate 10	0.422	0.157
Swan Rd. / 22nd St.	0.725	0.129
Valencia Rd. / 12th Ave.	0.461	0.463
Wetmore Rd. / First Ave.	0.305	0.233
Wilmot Rd. / 5th St.	0.256	0.340
Wilmot Rd. / 22nd St.	0.556	0.625
Interstate 10 / 22nd St.	0.316	0.182
5th Ave. / Interstate 10	0.179	0.116

* Accidents per million entering vehicles

Table 4-8. Left-Turn Accidents, City of Tucson Arterial / Collector Intersections

Intersection	Number of Left Turn Accidents 1982 - 1984	1986 - 1987
Alvernon Way / 29th St.	29	14
Auto Mall Dr. / Oracle Rd.	1	3
Broadway Blvd. / Columbus Blvd.	13	3
Broadway Blvd. / Randolf Way	11	7
Broadway Blvd. / Rosemont Blvd.	5	3
Cherry Ave. / 22nd St.	11	11
Columbus Blvd. / 22nd St.	26	8
Grant Rd. / Wilmot Rd.	7	11
Limberlost Rd. / First Ave.	5	4
Oracle Rd. / Roger Rd.	6	1
Santa Clara Ave. / Valencia Rd.	7	2
Tucson Blvd. / Valencia Rd.	16	14

Table 4-9. Left-Turn Accidents per Year, City of Tucson Arterial / Collector Intersections

Intersection	Left Turn Accidents Per Year Before	After	Difference
Alvernon Way / 29th St.	9.67	7.00	-2.67
Auto Mall Dr. / Oracle Rd.	0.33	1.50	1.17
Broadway Blvd. / Columbus Blvd.	4.33	1.50	-2.83
Broadway Blvd. / Randolf Way	3.67	3.50	-0.17
Broadway Blvd. / Rosemont Blvd.	1.67	1.50	-0.17
Cherry Ave. / 22nd St.	3.67	5.50	1.83
Columbus Blvd. / 22nd St.	8.67	4.00	-4.67
Grant Rd. / Wilmot Rd.	2.33	5.50	3.17
Limberlost Rd. / First Ave.	1.67	2.00	0.33
Oracle Rd. / Roger Rd.	2.00	0.50	-1.50
Santa Clara Ave. / Valencia Rd.	2.33	1.00	-1.33
Tucson Blvd. / Valencia Rd.	5.33	7.00	1.67
Total	45.67	40.50	-5.17
Percent Change			-11.32%

Table 4-10. Left-Turn Accident Rate, City of Tucson Arterial / Collector Intersections

Intersection	Left Turn Accident Rate* 1982 - 1984	1986 - 1987
Alvernon Way / 29th St.	0.502	0.351
Auto Mall Dr. / Oracle Rd.	0.017	0.073
Broadway Blvd. / Columbus Blvd.	0.159	0.058
Broadway Blvd. / Randolf Way	0.273	0.236
Broadway Blvd. / Rosemont Blvd.	0.118	0.12
Cherry Ave. / 22nd St.	0.211	0.347
Columbus Blvd. / 22nd St.	0.489	0.233
Grant Rd. / Wilmot Rd.	0.126	0.263
Limberlost Rd. / First Ave.	0.161	0.175
Oracle Rd. / Roger Rd.	0.183	0.042
Santa Clara Ave. / Valencia Rd.	0.216	0.079
Tucson Blvd. / Valencia Rd.	0.474	0.652

* Accidents per million entering vehicles

PART III

TRAFFIC OPERATIONS STUDIES

PART III of this report discusses the effort undertaken to determine the differences in traffic operation between leading left turn phasing and lagging left turn phasing.

CHAPTER 5 discusses the various intersection delay studies that were conducted in the Phoenix area. The first comparison made is that between leading and lagging operation operating at six intersections in the Phoenix area. From the analysis it is shown that total intersection delay is significantly greater with lagging left turns. The second comparison is between leading and combination operation at one intersection in Mesa. Due to the small sample size, a statistical test cannot be performed here. The third comparison is between Third Car and First car actuation operating at five intersections in the Phoenix area. These comparison showed no significant difference in total intersection delay, but a significant increase in left turn delay for the 3rd car actuated condition.

CHAPTER 6 discusses the signal operation analysis performed in the Pima County area. Leading left turn operation was compared to lagging left turn operation at actuated-isolated-unsaturated signals in the Pima County area. In a comparison in percent stopped vehicles, there is no change between leading and lagging operation. Vehicle delay increased at all intersections.

CHAPTER 7 discusses the travel time and delay studies that were performed in the Phoenix area. In Glendale and Tempe, nine routes were studied with four different timing plans - Existing all-leading, optimized all-leading, optimized all-lagging, and optimized combination. Leading and lagging were not permitted in opposing directions due to the trap situation. The results show no consistent result in the operation of these various patterns. In Mesa, however, leading left turn phasing was compared to leading eastbound, lagging westbound phasing. This phasing is different in that these phases are protected only and leading and lagging were permitted in opposing directions. The result of the Mesa study showed that delay, travel time and stops were all reduced with the combination phasing, although not significantly.

CHAPTER 5

PHOENIX AREA INTERSECTION ANALYSIS

INTRODUCTION

Intersection stopped time delay studies were conducted to evaluate the difference in performance between leading and lagging left turn arrow operation. One intersection was studied to evaluate the difference between leading and combination leading and lagging operation. Delay studies were also conducted, as part of this research, to evaluate the difference between 3rd car and 1st car actuation. The 3rd car/1st car comparison was performed strictly for the leading operation.

DATA COLLECTION

The study of comparisons took the form of a before and after analysis, therefore special care was taken to insure similar conditions existed for each study performed at a particular intersection. The duration of each study was one hour during the PM peak. Each study was conducted in good weather under normal traffic conditions. Measurement of intersection delay was performed by direct observation of stopped vehicles counted at fifteen second intervals. One observer was assigned to each approach. A turning movement volume count was performed for each study. Vehicles were counted as they entered the intersection. Volume count summaries were generated for each 15 minute interval.

The average stopped time delay was calculated using the equation:

$$\text{DELAY} = (\Sigma V_s * 15)/V$$

where:

DELAY	= average delay, in seconds/vehicle;
ΣV_s	= sum of stopped vehicle counts;
15	= interval between stopped vehicle counts, in seconds; and
V	= total volume observed during the study period.

Average stopped time delay values were calculated for left turn vehicles, through/right turn vehicles, and total intersection approach vehicles. Summary worksheets for each delay study are presented in Appendix D.

The following is a discussion on each of the comparisons that were made.

ANALYSIS

Leading Versus Lagging Operation

A paired comparison was made between the average delay per vehicle in the leading condition and the average delay per vehicle in the lagging condition. Six intersections were used in the analysis:

- 51st Avenue/Glendale,
- 51st Avenue/Northern,

- 51st Avenue/Olive,
- 51st Avenue/Peoria,
- 48th Street/Southern, and
- 48th Street/Broadway.

Manual stopped time delay studies were conducted at each intersection prior to any signal timing changes associated with this research. In the before condition, each of the six intersections operated with leading left turns. Five of the six intersections operated with protected/permissive left turn phasing and third car actuation on all approaches. The 48th St./Broadway intersection operated with protected only left turns and first car actuation on the northbound and southbound approaches and protected/permissive left turn phasing with third car actuation on the eastbound and westbound approaches.

Manual stopped time delay studies were conducted at each intersection with lagging operation. All approaches which were protected/permissive in the leading condition were permissive/protected in the lagging condition. The two protected only approaches remained protected only in the lagging operation.

Results

A before and after difference in the average stopped time delay per approach vehicle was calculated for each intersection. A difference was calculated for left turn vehicles, through/right turn vehicles, and total intersection approach vehicles. The percent change in delay from the before to the after condition was also calculated. The results of the Phoenix area intersection analysis of leading versus lagging left turn operation are presented in Table 5-1 and Figures 5-1, 5-2, and 5-3.

Average stopped time delay per left turn approach vehicle increased in the after condition at four of the six intersections studied. The largest change occurred at 51st Ave./Northern, where delay increased by 139% for left turn vehicles. The 48th St./Southern intersection measured essentially no change for left turn vehicle delay with the conversion to lagging left turns, while the intersection of 48th Street/Broadway registered a 5% decrease in delay for left turn vehicles in the after condition.

Average delay per through/right turn approach vehicle increased at five of the six intersections studied. The largest increase occurred at 48th St./Southern, with 129% more delay for through/right turn vehicles in the after condition. The 51st Ave./Northern intersection was the only intersection which registered a decrease in delay for through/right turn vehicles in the after condition. Delay decreased approximately 16% at this location.

Average delay per total approach vehicle also showed increases in the after condition at the same five intersections, though the changes were not as drastic when total intersection approach vehicles were considered. The large increase in through/right turn delay at 48th St./Southern was partially offset by no change in left turn delay. However, this intersection still registered the largest increase (85%) in total intersection delay with the conversion to a lagging operation. The 51st Ave./Northern intersection was the only location which registered an overall improvement in the after condition with a decrease in total intersection delay of approximately 4%.

Table 5-1. Leading vs. Lagging Intersection Delay, Phoenix Area

Intersection		Delay per Approach Vehicle (sec/veh) Left Turn	Thru/Right	Total
1. 51st Ave/Glendale	Before	25.70	22.55	22.95
	After	57.79	34.34	37.66
	Difference	32.09	11.79	14.71
	Change	125%	52%	64%
2. 51st Ave/Northern	Before	23.51	44.57	41.57
	After	56.24	37.32	39.80
	Difference	32.73	-7.25	-1.77
	Change	139%	-16%	-4%
3. 51st Ave/Olive	Before	27.50	21.58	22.41
	After	45.30	27.65	30.19
	Difference	17.80	6.07	7.78
	Change	65%	28%	35%
4. 51st Ave/Peoria	Before	42.03	20.07	22.88
	After	65.64	33.83	38.00
	Difference	23.61	13.76	15.12
	Change	56%	69%	66%
5. 48th St/Southern	Before	54.95	21.56	27.23
	After	54.92	49.28	50.30
	Difference	-0.03	27.72	23.07
	Change	-0%	129%	85%
6. 48th St/Broadway	Before	63.39	39.27	44.51
	After	60.14	43.97	47.91
	Difference	-3.25	4.70	3.40
	Change	-5%	12%	8%
Analysis				
	Sample Size	6	6	6
	Mean of Difference	17.16	9.47	10.38
	Overall Change	63.30%	45.54%	42.17%
	Sample Standard Deviation	15.62	11.58	9.00
	Test Statistic (t)	2.691	2.002	2.825
	Significant @ 95%?	yes (p=.04)	no (p=.10)	yes (p=.04)

Before Condition: Leading Operation
After Condition: Lagging Operation

Statistical Analysis

Three statistical tests were performed:

- a difference by intersection left turn movements,
- a difference by intersection through/right turn movements, and
- a difference by total intersection delay.

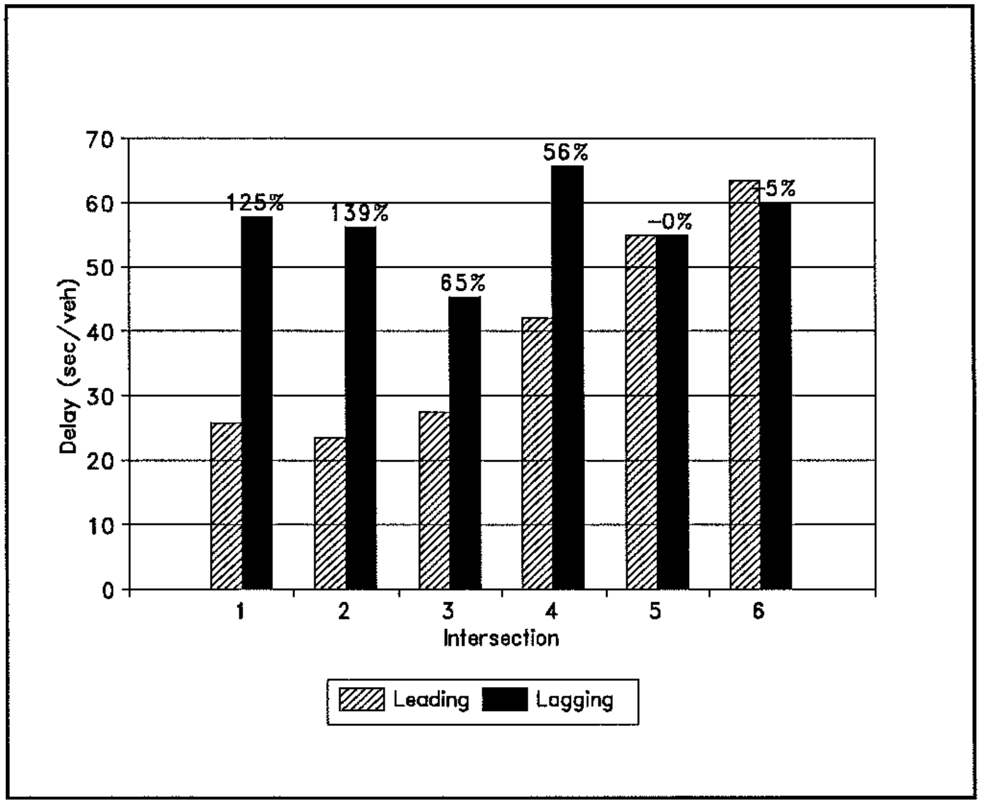

Figure 5-1. Leading vs. Lagging Left Turn Delay, Phoenix Area

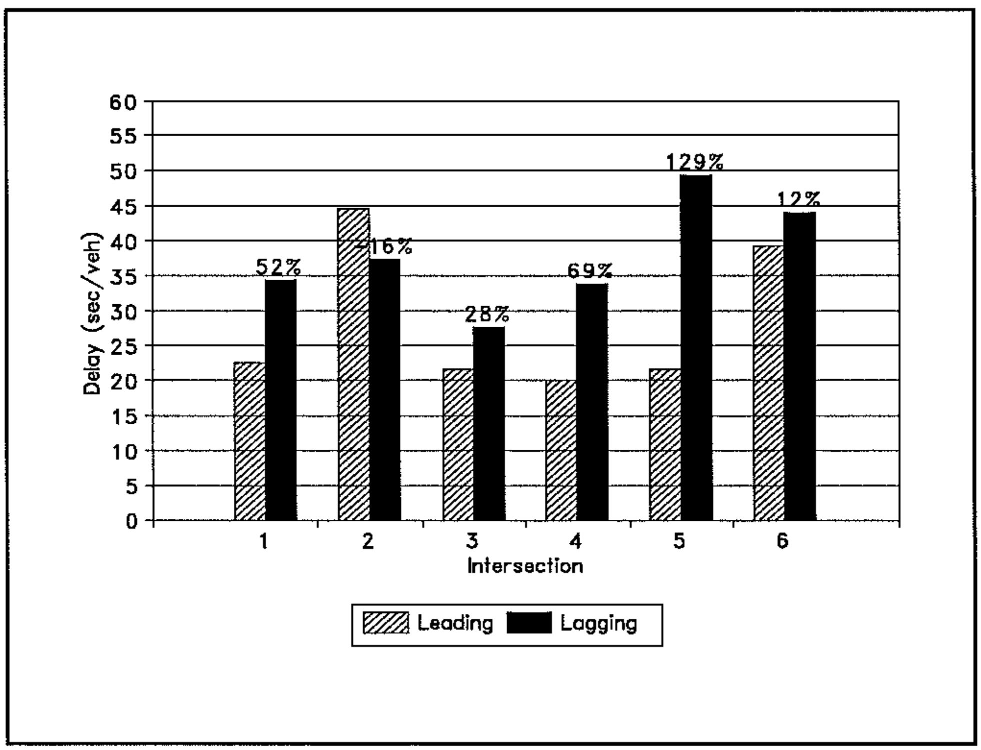

Figure 5-2. Leading vs. Lagging Through/Right-Turn Delay, Phoenix Area.

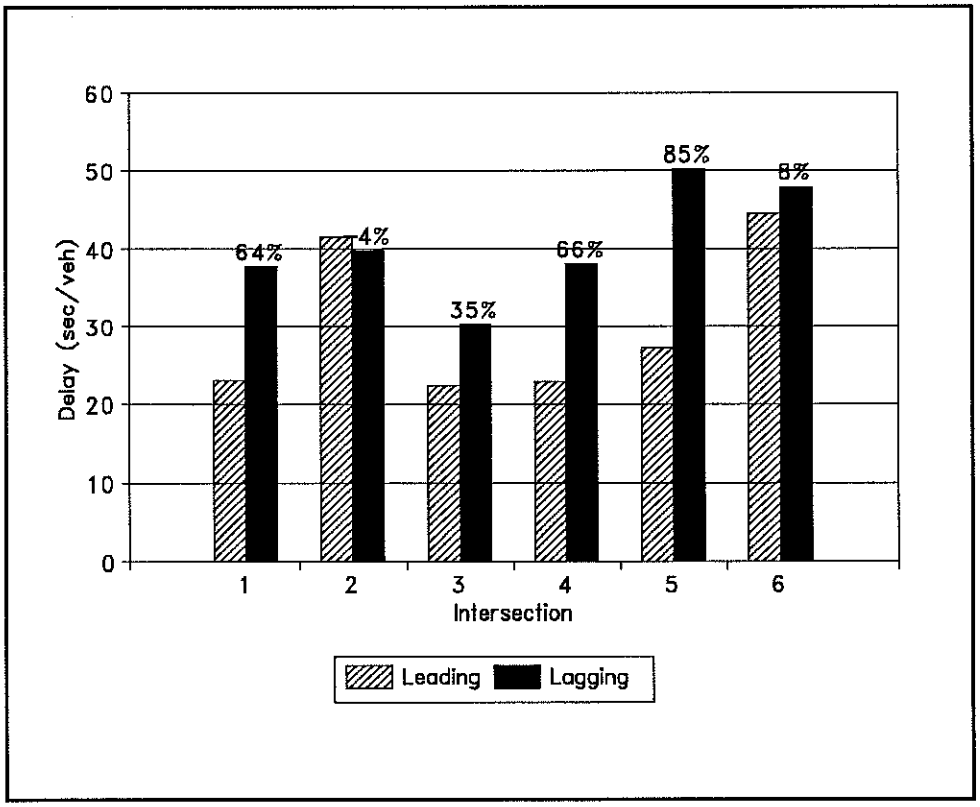

Figure 5-3. Leading vs. Lagging Total Intersection Delay, Phoenix Area

In each case, the statistical test performed was a paired t-test using the difference for each pair as one observed value. A mean of the difference was then calculated. The null hypothesis for the each test was that the difference between the before and after condition is equal to zero. Or, in other words, there is no significant difference in delay for the two conditions. A two tail test was performed at a 95% level of confidence.

The results of the paired data analysis are also presented in Table 5-1. The critical t-value for the test is 2.571. The mean of the difference for left turn vehicles is 17.16 seconds per approach vehicle. This represents an overall increase of 63% in delay with the conversion from a leading to a lagging operation. The calculated t-value is 2.691 (p=.04). The calculated t-value is greater than the critical t-value therefore the test indicates a statistically significant change in delay at the 95% confidence level for left turn vehicles. The mean of the difference for through/right turn vehicles is 9.47 seconds per approach vehicle, or an increase of approximately 46% in delay. The calculated t-value is 2.002 (p=.10). The calculated t-value is less than the critical t-value therefore the null hypothesis can not be rejected at the 95% confidence level on the basis of this test. The mean of the difference for total intersection delay is 10.38 seconds per approach vehicle, or an overall increase in total intersection delay of 42% with the conversion from leading to lagging left turn phasing. The calculated t-value is 2.825 (p=.04). The calculated t-value is greater than the critical t-value therefore the null hypothesis is rejected. On the basis of this test, it is concluded that total intersection delay is significantly greater for the lagging left turn operation.

Leading Versus Combination Operation

Two delay studies were performed at the intersection of Southern Ave./Stewart in Mesa to compare the difference in delay for a leading operation and a combination leading and lagging operation. Southern Avenue is an east/west arterial street. Stewart is a local collector street. The signal operated in a five phase mode in the before condition with protected only phasing on the east and west approaches. The combination phasing operated with leading left turns in the eastbound direction and lagging in the westbound direction. The signal was also operated in the protected only mode in the after condition.

Results

The results of the before and after study are presented in Table 5-2 and Figure 5-4. Delay per intersection left turn approach vehicle decreased by 4.61 seconds, or approximately 12%, in the after condition. The decrease was 1.13 seconds per vehicle for the through/right turn movements. This represents a change of approximately 11%. Total intersection delay decreased by 1.32 seconds per approach vehicle in the after period. Total intersection delay decreased by approximately 9% with the conversion to a combination leading and lagging operation.

This intersection was not included in the leading versus lagging analysis because of the different

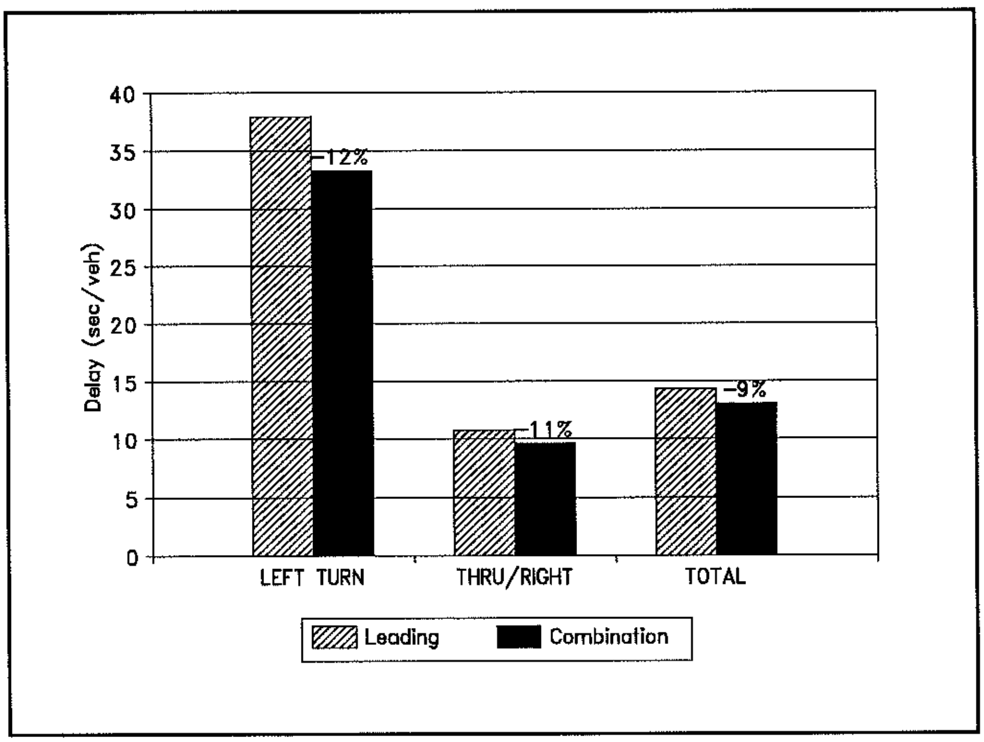

Figure 5-4. Leading vs. Combination Delay, Phoenix Area.

Table 5-2. Leading vs. Combination Intersection Delay, Phoenix Area

		Delay per Approach Vehicle (sec/veh)		
Intersection		**Left Turn**	**Thru/Right**	**Total**
Southern/Stewart	Before	37.86	10.76	14.34
	After	33.25	9.63	13.02
	Difference	-4.61	-1.13	-1.32
	Change	-12%	-11%	-9%

Before Condition: Leading Operation
After Condition: Combination (leading EB/lagging WB)

phasing pattern. A separate statistical analysis was not performed due to the limited sample size. The primary motivation for this study was to evaluate possible improved progression rather than intersection delay. The results of the progression analysis are included in Chapter 7 - Phoenix Area Travel Time Analysis.

Third Car Versus First Car Actuation

A paired comparison was also made between the average delay per vehicle in the 3rd car actuated condition and the 1st car actuated condition. Five intersections were used in the analysis:

- 48th Street/Southern,
- 48th Street/Broadway,
- 35th Avenue/Dunlap,
- 43rd Avenue/Northern, and
- 51st Street/Elliot.

Manual stopped time delay studies were conducted at each intersection prior to any signal timing changes associated with this research. In the before condition, all five intersections operated with leading left turns. Three of the five intersections operated with protected/permissive left turn phasing and 3rd car actuation on all approaches. The 48th Street/Broadway intersection operated with protected only left turns and 1st car actuation on the northbound and southbound approaches. The eastbound and westbound approaches operated with protected/permissive left turn phasing and 3rd car actuation. The 51st Street/Elliot intersection operated in a five phase mode in the before condition with protected/permissive left turn phasing and 3rd car actuation on the eastbound and westbound approaches.

Manual stopped time delay studies were conducted at each intersection with 1st car actuation. All approaches which were 3rd car actuated in the before condition were converted to 1st car actuation in the after condition. The two protected only approaches remained protected only with 1st car actuation in the after condition.

Results

A before and after difference in the average stopped time delay per approach vehicle was calculated for each intersection. A difference was calculated for left turn vehicles, through/right turn vehicles, and total intersection approach vehicles. The percent change in delay from the before to the after condition was also calculated. The results of the Phoenix area intersection analysis of 3rd car versus 1st car left turn actuation are presented in Table 5-3 and Figures 5-5, 5-6, and 5-7.

Average stopped time delay per left turn approach vehicle decreased in the after condition at four of the five intersections studied. The largest decrease occurred at 48th St./Southern, where delay decreased by approximately 32% for left turn vehicles. The 51st St./Elliot intersection recorded a 9% increase in delay for left turn vehicles in the after condition.

Table 5-3. Third Car vs. First Car Intersection Delay, Phoenix Area

Intersection		Delay per Approach Vehicle (sec/veh) Left Turn	Thru/Right	Total
1. 48th St/Southern	Before	54.95	21.56	27.23
	After	37.43	27.66	29.35
	Difference	-17.52	6.10	2.12
	Change	-32%	28%	8%
2. 48th St/Broadway	Before	63.39	39.27	44.51
	After	50.56	31.46	35.78
	Difference	-12.83	-7.81	-8.73
	Change	-20%	-20%	-20%
3. 35th Ave/Dunlap	Before	35.51	30.63	31.25
	After	26.07	29.95	29.48
	Difference	-9.44	-0.68	-1.77
	Change	-27%	-2%	-6%
4. 43rd Ave/Northern	Before	39.95	39.17	39.27
	After	28.19	41.74	40.06
	Difference	-11.76	2.57	0.79
	Change	-29%	7%	2%
5. 51st St/Elliot	Before	29.41	9.55	14.72
	After	31.97	11.11	16.34
	Difference	2.56	1.56	1.62
	Change	9%	16%	11%
Analysis				
	Sample Size	5	5	5
	Mean of Difference	-9.80	0.35	-1.19
	Sample Standard Deviation	7.51	5.17	4.47
	Test Statistic (t)	-2.918	0.150	-0.597
	Significant @ 95%?	yes (p=.04)	no (p=.89)	no (p=.58)

Before Condition: 3rd Car Actuated Leading Operation

After Condition: 1st Car Actuated Leading Operation

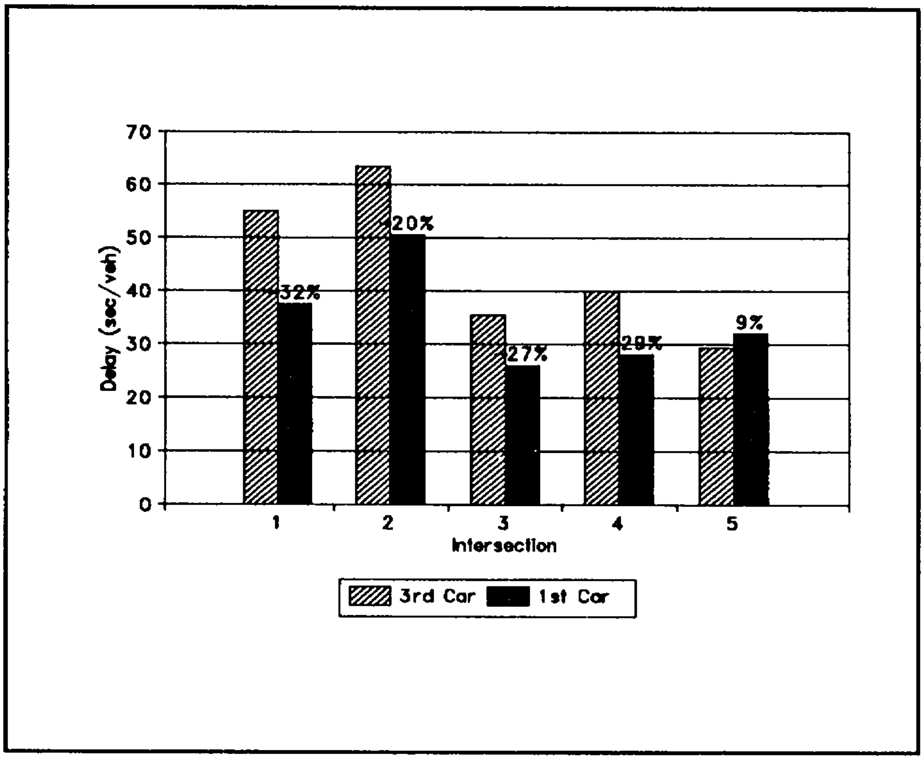

Figure 5-5. Third Car vs. First Car Left Turn Delay, Phoenix Area

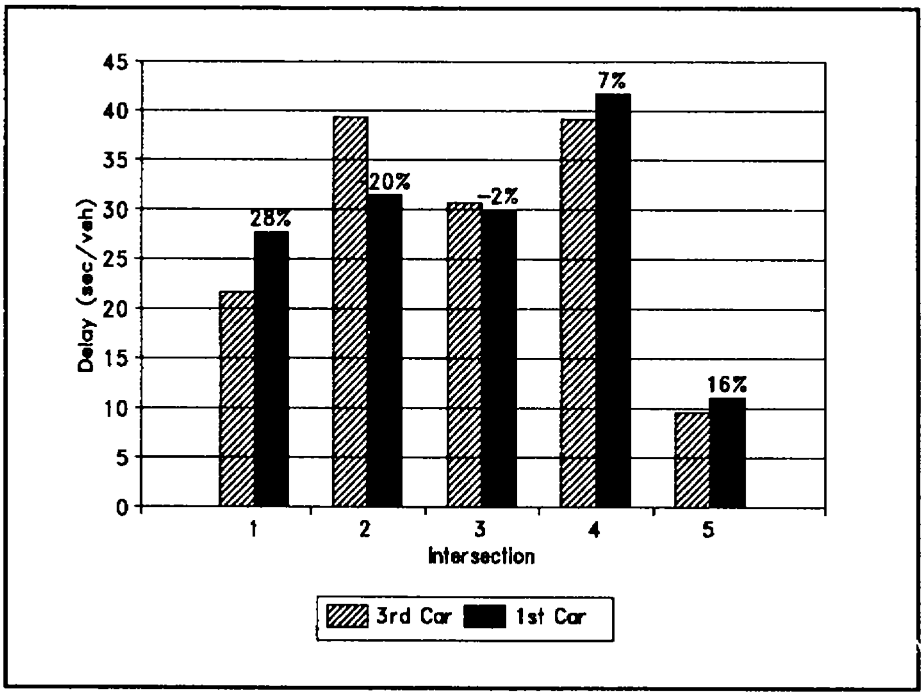

Figure 5-6. Third Car vs. First Car Through/Right-Turn Delay, Phoenix Area

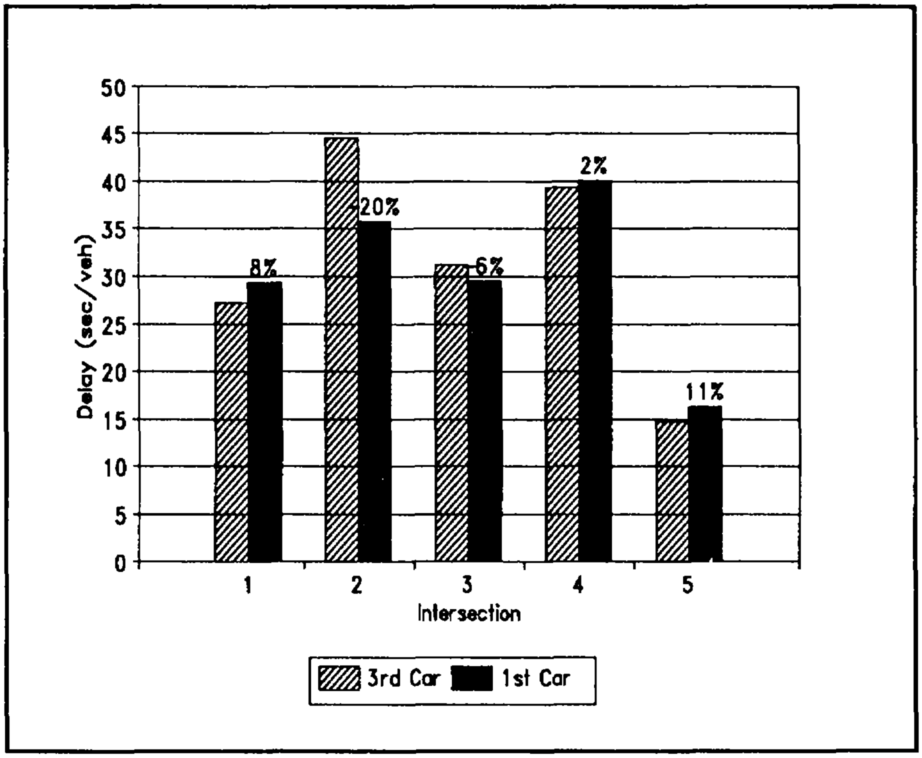

Figure 5-7. Third Car vs. First Car Total Intersection Delay, Phoenix Area

Average delay per through/right turn approach vehicle increased at three of the five intersections studied. The largest increase occurred at 48th St./Southern with a difference of 6.1 seconds per approach through/right turn vehicle. This represents an increase in delay of approximately 28%. Two intersections actually show a decrease in delay for through/right turn vehicles with the conversion to 1st car actuation. The largest decrease occurred at 48th St./Broadway which recorded 7.81 seconds less delay or a decrease of 20% in the after condition.

Average delay per total intersection approach vehicle increased at three of the five intersections studied. The largest percent increase occurred at 51st St./Elliot where delay increased 11% with the conversion to 1st car actuation. The largest overall increase in delay occurred at 48th St./Southern where total intersection delay increased 2.12 seconds per approach vehicle in the after condition. This represents an increase in delay of 8% with the conversion to 1st car actuation. A decrease in total intersection delay occurred at two intersections. Delay decreased approximately 20% at 48th St./Broadway and approximately 6% at 35th Ave./Dunlap.

Statistical Analysis

The same three statistical tests used in the leading versus lagging analysis were performed to evaluate the difference in delay per vehicle for 3rd car and 1st car actuation. The null hypothesis, once again, is that there is no significant difference in delay for the two conditions. A two tail test was performed at a 95% level of confidence.

The results of the paired data analysis are also presented in Table 5-3. The critical t-value for the test is 2.776. The mean of the difference for left turn vehicles is -9.80 seconds per approach vehicle. The calculated t-value is -2.918. (p.=04) The absolute value of the calculated t-value is greater than the critical t-value therefore the test indicates a statistically significant decrease in delay at the 95% confidence level for left turn vehicles with 1st car actuation. The mean of the difference for through/ right turn vehicles is 0.35 seconds per approach vehicle. The calculated t-value is 0.150 (p=.89). The calculated t-value is less than the critical t-value therefore the null hypothesis can not be rejected on the basis of this test. The mean of the difference for total intersection delay is -1.19 seconds per approach vehicle. The calculated t-value is -0.597 (p=.58). The absolute value of the calculated t-value is less than the critical t-value therefore the null hypothesis cannot be rejected. On the basis of this test, it is concluded that total intersection delay is not significantly different at the 95% level in the before and after conditions although there is a significant difference in delay for left turn vehicles.

DISCUSSION OF RESULTS

The results of the leading versus lagging analysis in the Phoenix area indicate a leading operation tends to be more efficient in terms of intersection delay. This finding is somewhat contrary to other experiments documented in the literature (10) However, the results of this current study are not surprising given the current phasing practices for implementation of lagging left arrow operations in the State of Arizona. In this research there were no left turn phase overlaps with the lagging left turn operations. Therefore, the potential benefits associated with phase overlap were lost in the conversion to a lagging operation.

The results of the leading versus combination lead/lag phasing indicate there may be reduced intersection delay with a combination phasing. However, the limited scope of the investigation precludes making any strong conclusions regarding the merits of combination phasing as related to intersection delay.

The results of the 3rd car versus 1st car actuation are perhaps the most interesting results associated with the Phoenix area intersection delay studies. As might be expected, the delay for left turn vehicles was less with 1st car actuation. However, the 51st St./Elliot intersection did record a 9% increase in delay for left turn vehicles in the after condition. However, this intersection does not provide a left turn phase for the north and south approaches. Therefore, the conversion to 1st car actuation did not serve as an advantage for left turners on the north and south approaches.

More surprising, however, were the results recorded for the through/right turn and total intersection delay. The analysis indicates no statistically significant difference at the 95% confidence level in delay per through/right turn vehicle or total approach vehicle with the change to 1st car actuation. This finding may, perhaps, be explained by examining the conditions under which the delay studies were performed.

The 48th St./Broadway intersection recorded a 20% decrease in intersection delay with the conversion to 1st car actuation. This intersection operates very close to capacity during the PM peak. The intersection also recorded slightly higher volumes during the 3rd car actuation study. This increased volume could have pushed the intersection into cycle failure during the course of the 3rd car actuation study.

All the delay studies were performed during the PM peak hour. Five intersections were used in the 3rd car versus 1st car analysis, or a total of 20 left turn approaches. All the intersections used in the analysis were running between 30 and 45 cycles per hour. Left turn volumes exceeded 150 vehicles per hour at 15 of the 20 approaches during the 3rd car actuated studies. These volumes would indicate the left turn arrow is being actuated a large proportion of cycles in the PM peak regardless of 1st car or 3rd car actuation.

The City of Phoenix collected data on the number of left arrow actuations and left turn volumes during the time the intersections of 35th Ave./Dunlap, 43rd Ave./Northern, and 51st St./Elliot were operated in the 1st car actuated mode. The same information was also collected for a typical weekday under 3rd car actuation. The data does indicate an increase in the number of times the arrow is actuated with the conversion to 1st car detection. However, the proportional increase in the number of actuation is much more acute in the off-peak hours than during the PM peak. Therefore, greater delay reductions should be expected in the off-peak rather than the PM peak with 3rd car actuation.

CHAPTER 6

PIMA COUNTY TRAFFIC SIGNAL OPERATION ANALYSIS

INTRODUCTION

As part of this project, an operational analysis of selected intersections in Pima County was undertaken. This analysis involved an examination of traffic and signal operation parameters before and after the conversion from leading to lagging turns. The purpose of this chapter is to document the data collection, analysis, and results of the study.

In 1984, the City of Tucson initiated a program to convert exclusive left turn signal phases from leading to lagging operation. The actual conversion of the signal phases began in 1985. Basically, the general rationale for this change was attributed to the potential for improving the operation of the signalized intersections with exclusive left turn phases. More specific reasons were related to:

- the need for the exclusive turn phase only when the demand in a signal cycle exceeded the capacity of the permitted left turn movement, and

- the influence of the leading left turn on arterial progression.

Given the conversion of the traffic signal operation by the City of Tucson, there was some lack of uniformity in the left turn phasing because the traffic signals under the jurisdiction of the Arizona Department of Transportation and the Pima County Department of Transportation continued to use the leading left turn phase. In order to eliminate the confusion to the motorists, all governmental agencies subsequently converted to the lagging left turn phase when exclusive turn phases are used. The Pima County Department of Transportation converted to the lagging left turn type of operation in the spring and summer of 1987.

The conversion from leading to lagging left turn signals by Pima County in 1987 represented a unique opportunity to examine the effect of the operational change. The time schedule for the conversion prevented the development of a formal funded research program on short notice; however a cooperative data collection effort was organized by Dr. Robert H. Wortman. With the cooperation of the Pima County Department of Transportation and the Arizona Department of Transportation, a "before and after" data collection effort was undertaken at selected intersections. This data set was then used as the basis of the comparative analysis of the left turn phasings in this research study.

Selection of Intersections

The conversion program in Pima County involved a total of thirty-seven signalized intersections in the Tucson area. At some of these intersections, various modifications to the signal operations were made in addition to the conversion of the left turn phasing. At a limited number of intersections, the only planned change was to switch from the leading to the lagging left turn operation; thus these intersections were selected for the "before and after" data collection. The intersections studied are listed in Table 6-1.

Ultimately, the intersection of First Avenue and Ina Road had other changes in the signal phasing

Table 6-1. Delay Study Intersections, Pima County

Intersection	Type of Control (a,b)
Ajo Way / Alvernon Way	4 Phase (c)
Alvernon Way / Irvington Rd.	4 Phase (Protected/Permissive)
Campbell Ave. / Skyline Rd.	3 Phase (Protected)
First Ave. / Ina Rd.	4 Phase (d)
First Ave. / Orange Grove Rd.	3 Phase (Protected/Permissive)
First Ave. / River Rd.	3 Phase (Protected/Permissive)
Ina Rd. / Thornydale Rd.	4 Phase (Protected/Permissive)
Kolb Rd. / Valencia Rd.	4 Phase (Protected)
Palo Verde Rd. / Valencia Rd.	3 Phase (Protected)

(a) The number of phases reflects the basic operation of the intersection. Phase overlaps were used in situations with opposing leading protected left turns.

(b) In the "after" condition, the "protected / permissive" left turn operation obviously becomes "permitted / protected".

(c) At the intersection of Ajo Way and Alvernon Way, a combination of types of control were used. For example, some approaches had protected left turn operations.

(d) At the intersection of First Avenue and Ina Road, a 4-phase signal operation was used in the before condition with protected / permissive left turns on the northbound and westbound approaches. For the after condition, the northbound and southbound approaches on First were treated as separate phases. In addition, the lane use on the northbound approach was changed.

as well as modifications in the lane use. While field data were collected at the site, the intersection was eliminated from the comparative analysis for this reason. In addition, the initiation of construction in the area of Ina Road and Thornydale Road significantly changes the traffic at that location prior to an opportunity to collect the "after" data.

While the changes in signal phasing at these intersections were limited to the conversion of the left turn operations, it must be recognized that there were some modifications in the signal timing. These modifications included the adjustment of green time allocation for specific movements as well as a re-evaluation of the time for the clearance or change interval.

Signal Phasing

Pima County uses actuated control for traffic signals; thus all of the intersections in the study utilized full actuated control. In addition, each of the intersections operated on an isolated basis with no interconnection with adjacent signals.

There was some variation in the treatment of left turn movements in the study intersections. Some of the intersections had permitted plus protected left turn movements while other intersections had protected left turn movements only. Table 6-1 identifies the operation of the left turn signal phasings at each of the study locations.

As has been indicated previously, the intersection of First Avenue and Ina Road ultimately underwent changes in signal operation as well as lane use. These modifications significantly changed the operation of the intersection; thus the intersection was later eliminated from the analysis.

It should be noted that phase overlaps were used for the leading left turn conditions; however, the overlaps were not used with the lagging left turn operations. For example, given an intersection with left turn arrows on all approaches, the signal would operate as an eight phase signal (or four phase with phase overlaps for the turn movements) with the leading left turns. With the lagging left turns, the signal would operate as four phase control (without any overlaps). This type of operation was standard in the Tucson area. In essence, this resulted in the loss of the use of the phase overlap when there were differences in the demand for the left turn movement.

At a limited number of intersections which utilized the protected only left turns, a phase overlap condition would occur with the lagging left turn operation. For example, one intersection had very low westbound approach volumes. For some cycles, the eastbound through and left turn movements would occur at the same time. At another intersection, the side street had low volumes on the north and south approaches. In addition, the westbound approach had very few left turns. Because of the lack of westbound left turns and cycles without traffic on the side street, the eastbound through movement continued along with the eastbound protected left turn. In essence, there was no need for a protected westbound left turn phase in some cycles. This type of operation only occurred at intersections with protected only left turn operations.

With respect to the actual signal timing, the study utilized the signal settings employed by Pima County for the before and after conditions. There was no attempt by the research team to evaluate the signal timing settings used at the intersections. Certainly, it was necessary for Pima County to adjust some of the signal timing settings in addition to changing from the leading to the lagging operation. Part of the reason for the necessity of adjusting the signal timings was associated with the fact that the loss of the left turn phase overlap had an impact on the phasing for the through movements.

DATA COLLECTION

For the field data collection, two time lapse super 8mm movie cameras were used to film the operation of each of the intersections. The location of the cameras was elevated by using trucks with elevating platforms or raised vantage points from nearby terrain. With the use of two cameras, it was possible to simultaneously film all of the intersection approaches. In addition, the cameras had internal clocks with digital displays; thus the filming with the two cameras could be coordinated. All filming was accomplished with the cameras operating on a speed of one frame per second.

The filming of each intersection occurred during the period from 3 PM to 6 PM on weekday afternoons. There was an attempt to schedule the before and after data collection at a specific intersection on the same day of the week even though all intersections were not filmed on the same day of the week. While filming was scheduled for the period from 3 PM to 6 PM, it was not possible to obtain a three hour data set in all cases. Equipment problems in addition to the time required to change the film cartridges resulted in some lost time.

The time period from 3 PM to 6 PM was selected for several reasons. First, this period permitted making observations over a peak hour period. Second, it was possible to schedule the use of the trucks with elevating platforms at the end of the normal workday. Finally, it was difficult to schedule personnel for data collection at other times. While it may have been desirable to collect data at other time periods, it must be recognized that the data collection was accomplished with resource limitations on a cooperative basis. Nevertheless, a rather extensive data set was collected for study.

The before data were collected during the period from middle of March 1987 to the middle of May 1987. This time schedule was necessitated and constrained by the timing of the signal conversions by Pima County. The after data collection began in early October 1987; thus there was a transition period of several months before the collection of the after data. During the fall of 1987, difficulties were encountered which served to disrupt and extend the data collection effort. For example, adverse weather and other demands for the use of the trucks made it impossible to film on some days. In addition, the shortening of the daytime period finally made it necessary to cease the data collection efforts until the spring of 1988. Data collection resumed in the spring of 1988. Difficulties were encountered with the films taken at the intersection of Kolb Road and Valencia Road; thus data collection was repeated at that intersection during the summer of 1990.

ANALYSIS

Using the film record of the intersections during the before and after periods, data which reflected operational parameters were extracted. This operational data for each intersection was then used for the comparative analysis of the leading and lagging left turn phasing. The discussion that follows presents the analysis and results of each of the operational parameters.

Intersection Volume

In the design of the data collection effort, it was recognized that significant changes in volume can have a potential impact on the operational measures of intersection performance. For this reason, a number of precautions were taken in an attempt to minimize the possibility of major changes in volume between the before and after study periods. For example, the initiation of the data collection for the after period was undertaken within several months of the completion of the before data collection. The after data collection was delayed until the fall of 1987 to avoid the possible effect of the summer period. In addition, an effort was made to collect the before and after data at a given intersection on the same day of the week.

The turn movement volumes for each intersection were obtained from the film records for the before and after periods. These volumes were then used to determine the actual changes in approach volumes for the two study periods.

Because of the loss of some time during the data collection process, the raw turn movement volumes were expanded to the equivalent of a three hour period. The total volume was then divided by three to provide an average hourly volume. This procedure resulted in a number that could be used for comparing the before and after periods. Table 6-2 presents the average approach volumes for each intersection.

At most of the study intersections, only minor differences in traffic volumes were observed. Given a relatively short period between the before and after data collection, only small differences would be expected.

Table 6-2. Intersection Total Approach Volumes, Pima County

Intersection	Average Approach Volume (vph)* Before	After	Difference
Ajo Way / Alvernon Way	3644	3523	-3%
Alvernon Way / Irvington Rd.	2788	2882	3%
Campbell Ave. / Skyline Rd.	2527	3070	21%
First Ave. / Orange Grove Rd.	2519	2472	-2%
First Ave. / River Rd.	3379	3107	-8%
Ina Rd. / Thornydale Rd.	3495	**	**
Kolb Rd. / Valencia Rd.	7052	5950	-16%
Palo Verde Rd. / Valencia Rd.	2560	2472	-3%

* The average approach volumes are for the entire intersection. The value in the table reflects the sum of all approaches.

** After values not available for Ina Rd. / Thornydale Rd.

Two exceptions to small changes in the approach volumes can be noted. The intersection of Campbell Avenue and Skyline Drive, however, had a significant increase in traffic volume for which there is no explanation for the cause. The before data set was collected in April 1987, and the after data set was taken the following October. While there was only six months between the data collection periods, there was a 21 percent increase in the approach volumes at that intersection. This increase generally occurred on all approaches and throughout the study period. In essence, there was a major increase in the use of the intersection.

In the second case, there was a 16 percent decrease in the approach volumes at the intersection of Kolb Road and Valencia Road. At this location, there had been a major change in employment in the vicinity of this intersection; thus the after condition was influenced by the reduction in employment.

In terms of later analyses of operational performance, it should be noted that significant increases in traffic volumes were found at only one intersection. In most cases, there was a reduction in the approach volumes.

Arrival of Vehicles

In addition to approach volume, the actual time of arrival of vehicles at an intersection during the signal cycle has an influence on overall performance measures. For this reason, the arrival of vehicles was examined as part of the analysis of before and after conditions.

All of the intersections in the study were operating on an isolated control basis. While there may be some platooning from adjacent signals, the overall arrival pattern should be random in terms of the time that vehicles arrive during a cycle.

The arrival pattern of vehicles for a given intersection was examined by determining the percent of the approach vehicles that had to stop due to the operation of the traffic signal. Basically, the review of the film revealed the approach vehicles that were required to stop as well as the vehicles that were able to pass through the intersection without stopping. The percent vehicles stopped was then calculated by comparing the number of vehicles that stopped to the total approach volume. Table 6-3 summarizes this information for each of the intersections.

A review of Table 6-3 reveals that there was little change in the percent vehicles stopped at half of the intersections. There was increase of about five percent in the stopping vehicles at one intersection, and a decrease of about six percent at another. The greatest measured change was at the intersection of Kolb Road and Valencia Road where there was an increase of ten percent. This difference was based on only the east and west intersection approaches.

Another interesting aspect of the information in Table 6-3 is the consistency of the values for the various intersections. At most of the intersections the percent of stopped vehicles was in the general range of fifty to fifty-five percent. The main exception was the intersection of Palo Verde Road and Valencia Road where the percentage for the before and after conditions was significantly lower than at other intersections. This lower value can be explained by the fact that there is a free flow right turn lane on one of the approaches.

Vehicle Delay

One of the main indicators of intersection performance is vehicle delay. Delay is specified in the *Highway Capacity Manual* (12) as the measure for determining intersection level of service. For this study, the stopped time delay was determined for each of the intersections. The standard procedure for determining stopped delay was used where the number of stopped vehicles is counted at a set interval. The intersections were filmed at one second intervals; thus the delay was determined based on observations taken from the film. While it is common to select fifteen second intervals for observations, a ten second interval was used in this study. The shorter interval was used to improve the accuracy especially for the movements with low approach volumes.

Table 6-4 summarizes the results of the delay analysis and indicates the average stopped delay for the stopped vehicles as well as the total approach vehicles. These values reflect the overall delay for

Table 6-3. Percent of Approach Vehicles Stopped, Pima County

	Percent Stopped	
Intersection	**Before**	**After**
Ajo Way / Alvernon Way	54.1	53.0
Alvernon Way / Irvington Rd.	54.5	53.8
Campbell Ave. / Skyline Rd.	50.7	55.6
First Ave. / Orange Grove Rd.	55.6	49.5
First Ave. / River Rd.	54.4	55.7
Ina Rd. / Thornydale Rd.	60.6	*
Kolb Rd. / Valencia Rd.	60.1**	70.4**
Palo Verde Rd. / Valencia Rd.	31.3	33.3

* After value not available for Ina Rd. / Thornydale Rd.

** At the Kolb Rd. / Valencia Rd. intersection, the values are for the eastbound and westbound approaches only. For the before condition, the percent vehicles stopped for all approaches was 49.2 percent. The after condition value for all approaches was not available.

Table 6-4. Vehicle Delay Comparison, Pima County

Intersection	Delay per Stopped Vehicle (Sec)	Delay Per Approach Vehicle (Sec)
Ajo Way / Alvernon Way		
Before	32.68	17.75
After	39.68	21.04
Difference	7.00 (21%)	3.29 (19%)
Alvernon Way / Irvington Rd.		
Before	22.82	12.44
After	32.32	17.39
Difference	9.50 (+42%)	4.95 (+40%)
Campbell Ave. / Skyline Dr.		
Before	27.45	13.93
After	31.43	17.47
Difference	3.98 (+14%)	3.54 (+25%)
First Ave. / Orange Grove Rd.		
Before	22.88	12.72
After	27.11	13.43
Difference	4.23 (+18%)	0.71 (+6%)
First Ave. / River Rd.		
Before	32.15	17.48
After	33.55	18.68
Difference	1.40 (+4%)	1.20 (+6%)
Ina Rd. / Thornydale Rd.		
Before	33.03	20.01
After	*	*
Difference		
Kolb Rd. / Valencia Rd.		
Before	26.04	12.69
After	*	19.27
Difference		6.58 (+52%)
Palo Verde Way / Valencia Rd.		
Before	19.25	6.03
After	23.58	7.85
Difference	4.33 (+22%)	1.82 (+30%)
Average Change	+20%	+30%

* After value not available

an intersection. More detailed information for each approach and the movement at each approach is contained in Appendix E. At all of the intersections where delay was actually measured, there were increases in the average delay per vehicle. Even for the intersections where there were decreases in the approach volume, the average vehicle delay increased.

In considering the results of the delay analysis, it is important to recognize that the data collection was during the period from 3 PM to 6 PM. For this reason, a true off peak condition was not included in the analysis.

Cycle Length

The final parameter that was included in the analysis of the before and after conditions was cycle length. Again, it should be noted that the research team did not attempt to evaluate the adequacy of the signal timing. The study simply utilized the signal timings that were in place prior to and after the conversion of the left turn signal operation. Upon the implementation of the lagging left turn signals, the Pima County staff did make some adjustments in the signal timing. These adjustments were necessary for the efficient operation of the intersection.

The average signal cycle lengths for the before and after periods for each intersection are given in Table 6-5. A general review of the table reveals that the differences in the cycle lengths vary from intersection to intersection with increases at some of the sites and decreases at other locations. If the differences in cycle length are considered in terms of the type of left turn treatment, there is a trend that can be noted. At intersections where there was a decrease in the cycle length, the permitted/ protected left turn was utilized. The increases in cycle length were at intersections where protected only left turns were utilized. Changes in cycle length, therefore, were a function of whether left turns were permitted along with the through movement or not.

The exception to an increase in cycle lengths with protected only lagging left turns occurred at the intersection of Palo Verde Rd. and Valencia Rd. At this intersection, the average cycle lengths remained virtually the same even with the protected left turn operations. Because of the low approach volumes for some movements, this is one of the intersections that resulted in a phase overlap type of operation. Because of this condition, the average cycle length remained the same.

Table 6-5. Average Cycle Length, Pima County

	Average Cycle Length (Sec)		
Intersection	**Before**	**After**	**Difference**
Ajo Way / Alvernon Way	95.3	114.3	19.0
Alvernon Way / Irvington Rd.	72.6	70.4	-2.2
Campbell Ave. / Skyline Rd.	79.9	90.3	10.4
First Ave. / Orange Grove Rd.	77.3	71.9	-5.4
First Ave. / River Rd.	95.6	90.7	-4.9
Ina Rd. / Thornydale Rd.	85.8	*	
Kolb Rd. / Valencia Rd.	65.7	76.7	11.0
Palo Verde Rd. / Valencia Rd.	62.1	62.6	0.5

* After value not available for Ina Rd. / Thornydale Rd.

DISCUSSION OF RESULTS

In considering the results of the analysis of the Pima County intersections, it must be recognized that:

- all of the study locations were operating with actuated control;
- the signals were basically isolated from other intersections, and there was no coordination with adjacent intersections at the time of the data collection;
- the intersections were not operating at what could be considered as saturated conditions; and
- vehicle queues generally cleared during each cycle.

There was some variation in the measured approach volumes at the study intersections; however only major changes occurred at two intersections. Because the intersections were not operating at saturated conditions, increases in volumes would not necessarily result in significant increases in delay.

Generally, there was little change in the percent vehicles stopped. This would suggest that arrival pattern was random in terms of the signal cycle. For this reason, the effect of platooning should not be a factor with respect to delay calculations and measurements.

It is significant to note that the reduction in cycle length was associated with intersections where permitted left turns were allowed. On the other hand, intersections with protected left turns only had increases in cycle length with the lagging left turn operation. This result is reasonable because of the fact that the opportunity for phase overlap was lost when the lagging left turn was used. In considering this general statement, it must be recognized that low traffic volumes for some movements can result in phase overlap operations with lagging protected only left turns. For this type of condition, the average cycle length did not increase.

The interesting result of the analysis is that vehicle delay increased at all intersections. At the study intersections, there was an average increase of 20% in the delay per stopped vehicle and an average increase of 30% in the delay per approach vehicle. The finding of delay increases is consistent with the results of the Phoenix area studies. Even when there was a decrease in approach volumes, there were increases in delay. Delay might be expected to increase with longer cycle lengths; however delay also increased at intersections with reductions in average cycle length.

CHAPTER 7

TRAVEL TIME STUDY

INTRODUCTION

As part of this research project alternative phasing sequences were tested using travel time data along five routes in Glendale and four routes in Tempe. The patterns tested were:

- All Leading
- All Lagging
- A combination of leading or lagging depending on which best fit the progression.

In addition, a combination phasing was tested along Southern Avenue in Mesa. This chapter documents the timing, data collection, analysis, and results of these travel time studies.

SIGNAL TIMING

In order to obtain a true comparison between leading and lagging left turns, it was necessary to use signal timing patterns developed by a common optimization program. Because of the ease of operation and the numerous runs that would be required as part of the combination portion of the study, FORCAST was utilized to optimize the signals. The first signal timing - all leading left turns, was performed using FORCAST operating on the City of Scottsdale computer. Subsequent runs were performed on the PC-based version of the FORCAST program.

Initial travel time runs were performed along each route to determine travel speeds and link distances. Existing intersection phasing and minimum times were obtained from the City of Glendale and City of Tempe Traffic Engineering staff. Traffic volumes were obtained from the city staff and the Maricopa Association of Governments Transportation Planning Division and were supplemented with turning movement counts made by Lee Engineering.

FORCAST optimizes by calculating a cost for each cycle length. A range of acceptable cycle lengths is input into the program. Based upon the phasing, volumes, and progression priority, FORCAST creates an optimum timing plan for each cycle length. FORCAST then calculates the motorists' cost of each timing plan based upon the main street delay, side street delay, and stops. A stop is equivalent to 20 seconds of either main street or side street delay. FORCAST uses a simple procedure to increase the cost due to a saturated intersection. If the intersection is saturated, FORCAST adds one cycle length to both the main street and side street delay value.

Timing plans were implemented in the study area for AM peak, PM peak and off-peak traffic patterns. Since FORCAST allows for different phasing patterns, the coding and optimization of all leading or all lagging was straightforward. The most difficult part of the timing portion of the project was in determining which combination of lead and lag at each intersection would produce the optimum combination timing.

For the Glendale timing, at each intersection along 51st Avenue there are 16 timing combinations theoretically possible as listed in Table 7-1.

Table 7-1. Travel Time Study Possible Signal Combinations, Phoenix Area

	Approach			
Combination	North	South	East	West
1	Lead	Lead	Lead	Lead
2	Lead	Lead	Lead	Lag
3	Lead	Lead	Lag	Lead
4	Lead	Lead	Lag	Lag
5	Lead	Lag	Lead	Lead
6	Lead	Lag	Lead	Lag
7	Lead	Lag	Lag	Lead
8	Lead	Lag	Lag	Lag
9	Lag	Lead	Lead	Lead
10	Lag	Lead	Lead	Lag
11	Lag	Lead	Lag	Lead
12	Lag	Lead	Lag	Lag
13	Lag	Lag	Lead	Lead
14	Lag	Lag	Lead	Lag
15	Lag	Lag	Lag	Lead
16	Lag	Lag	Lag	Lag

If the four signals along 51st Avenue could be timed with any of the sixteen timing patterns, then in order to determine the optimum phasing combination at each intersection, timing plans would have to be generated for 16^4=65,536 possible combinations. If there are three timing patterns- AM, MID, and PM, possible, the number jumps to 65,536 * 3 = 196,608 possible combinations.

Since all of the signals along 51st Avenue operate in a protective-permissive mode, only four of the sixteen combinations can be utilized in order to avoid the trap. These four combinations are shown in Table 7-2.

This reduced the number of patterns in Glendale to 256 plans per time period or a total of 768 combinations. In Tempe only two signals were changed from leading to lagging requiring the generation of only 48 timing plans.

Table 7-2. Travel Time Study Utilized Signal Combinations, Phoenix Area

	Approach			
Combination	North	South	East	West
1	Lead	Lead	Lead	Lead
4	Lead	Lead	Lag	Lag
13	Lag	Lag	Lead	Lead
16	Lag	Lag	Lag	Lag

There was one additional constraint in that whatever combination was chosen, it had to be implemented for all three time periods of the day. Therefore if Peoria WB had lagging left turn in the AM peak, it had to remain that way for the off-peak and PM peak periods.

Once the timing plans were generated the outputs were scanned for the lowest cost cycle length. This was not necessarily the same cycle length for each combination. A spreadsheet was then created which showed each combination and the costs associated with the AM, PM and off-peak plan. This

Table 7-3. Lowest Cost Timing Plans, City of Glendale

Intersection Phasing				Weighted Cost			
Glendale	Olive	Peoria	Northern	AM	Mid	PM	Total
2	3	2	2	941	1820	2267	5028
2	2	2	2	924	1804	2313	5041
2	3	2	4	989	1838	2276	5103
2	2	2	4	972	1824	2322	5118
2	2	2	1	972	1827	2334	5133
2	3	2	3	1057	1840	2267	5165
2	2	2	3	1106	1826	2313	5245
2	3	4	2	1090	1948	2227	5265
2	3	4	3	1101	1969	2225	5295
2	3	3	2	1128	1821	2354	5302
2	2	3	2	1111	1806	2400	5316
2	2	3	2	1111	1806	2400	5317
2	3	4	4	1139	1968	2251	5357
2	3	3	4	1192	1840	2363	5395
1	2	2	2	943	1902	2552	5396
2	2	3	4	1175	1826	2409	5410
2	3	3	1	1192	1844	2374	5410
2	3	3	3	1219	1841	2354	5414
2	2	3	1	1175	1829	2420	5425
3	3	3	2	1134	1815	2501	5450
2	2	3	3	1235	1827	2400	5462
3	3	3	3	1184	1837	2501	5522
3	3	3	4	1198	1834	2510	5542
2	1	2	2	1068	1849	2639	5556
3	3	3	1	1198	1838	2522	5558
3	3	2	2	956	2246	2391	5592
2	1	2	4	1122	1868	2649	5639
2	1	2	1	1122	1872	2660	5654
2	1	2	3	1175	1870	2639	5684
2	1	3	2	1258	1851	2753	5862
4	3	4	4	1260	2355	2260	5874
2	1	3	3	1272	1872	2753	5897
2	1	3	4	1340	1870	2762	5972
2	1	3	1	1340	1874	2774	5988

Legend:
1 All Lagging
2 All Leading
3 Leading N-S Lagging E-W
4 Leading E-W Lagging N-S

was weighted by the volume to determine the lowest cost plan. The thirty lowest cost plans for Glendale are shown in Table 7-3 and the lowest cost plans for Tempe are shown in Table 7-4.

It should be noted that the lowest cost timing plan for both cities was very similar to the all-leading timing plan. There are two reasons this occurred. By implementing lagging left turns, these must be tied together preventing an overlap scenario. Since there is a loss of efficiency associated with this type of phasing, FORCAST will only choose lagging if the left turn volumes are nearly identical. If lagging left turns are implemented, the greatest benefit is that should the left turn vehicles make their maneuvers during the through movement, ie. finding available gaps in the traffic stream, then it is possible that the protected left turn phase will not be necessary. FORCAST does not have an algorithm which determines if this scenario will occur, therefore, it cannot recognize the benefits of lagging left turns. There may be situations where the left turn traffic volume is light but unbalanced and FORCAST would choose leading operation, while lagging might be a better choice both because it fits better in the progression scheme and because lagging operation might avoid the need for some of the protected phases, which provides more time for the through movement.

DATA COLLECTION

Once the timing plans were implemented into the street, travel time runs were performed using the 'floating car' method. The TIMELAPSE Travelog data collection computer was utilized to collect this data. Once the routes are entered into the computer, the driver simply pushes a button at the beginning of the route and drives to the end of the route.

Table 7-4. Lowest Cost Timing Plans, City of Tempe

Intersection Phasing		Weighted Cost			
Broadway	Southern	AM	Mid	PM	Total
3	3	820	971	444	2234
2	3	836	985	436	2258
3	2	820	1072	391	2282
4	2	832	1038	419	2288
1	2	820	1126	435	2382
1	3	826	1163	463	2451
4	3	836	1167	461	2465
2	4	836	1285	389	2510
2	1	831	1264	420	2515
4	4	816	1405	418	2639
3	1	801	1555	428	2783
4	1	833	1523	448	2805
1	4	809	1600	410	2819
3	4	861	1649	394	2904

Legend:
1 All Leading
2 All Lagging
3 Leading N-S Lagging E-W
4 Leading E-W Lagging N-S

Six travel time runs were performed for each route in each direction for three time periods - AM peak, PM peak, and off-peak. One driver collected all the data in Glendale and a separate driver collected the travel time data for Tempe. The same driver was used for all runs in each city in order to eliminate the variability of different drivers.

Once all of the travel time runs were collected, they were up loaded to ASCII files on the IBM PC. TIMELAPSE has developed a software program which reads these data files and computes the following information:

- Travel Time
- Time in Queue (delay time)
- Stops
- Average speed
- Cruise speed
- Fuel Consumption
- Carbon Monoxide emissions
- HydroCarbons emissions
- Nitrous oxides emissions

An example output for the software program is found in Appendix F.

The six runs were averaged for each route to determine the average stops, delay time and travel time for each route. Each of the estimates for the routes was multiplied by its respective volume to produce a weighted point estimate based upon the route volume. A paired Student's t-test was then performed between each sample. The following comparisons were made:

- Existing leading minus FORCAST optimized leading
- Existing leading minus FORCAST optimized lagging
- Existing leading minus FORCAST optimized combination
- FORCAST leading minus FORCAST lagging
- FORCAST leading minus FORCAST combination
- FORCAST lagging minus FORCAST combination

The estimates of stops, delay and travel time produce three distinct variables for each timing plan. In order to compare the timing plans, it was felt that aggregating these three variables into one variable would be helpful. Because FORCAST develops timing plans which weights the benefit of reduced stops with reduced delay and travel time, a representative cost for each timing plan was developed using the information in *A Manual on User Benefit Analysis of Highway and Bus Transit Improvements*, (13) published by the American Association of State Highway and Transportation Officials, 1977. These values have been updated to 1988 dollars by using the transportation portion of the Consumer Price Index. The following values were utilized.

Parameter	Cost
Stops	$41.00 / 1000 stops
Delay	$ 0.616 / Vehicle-hour idling
Travel Time	$ 3.35 / Vehicle-hour traveling

ANALYSIS

Glendale Travel Time Study

The study area for the Glendale portion of the project was bounded by Grand Avenue to the south, Cactus Road to the north, 43rd Avenue to the east, and 67th Avenue to the west as shown in Figure 7-1. In this study area, all the major arterial - major arterial intersections were operating in a protected-permissive leading left turn mode in the before condition. Optimization of all of the signals within the study area was performed using the FORCAST signal timing program but only the signals along 51st Avenue had the phasing patterns changed during the course of the study.

The five routes chosen for the Glendale study were the following:

- 51st Avenue
- 59th Avenue
- Peoria Avenue
- Olive Avenue
- Northern Avenue

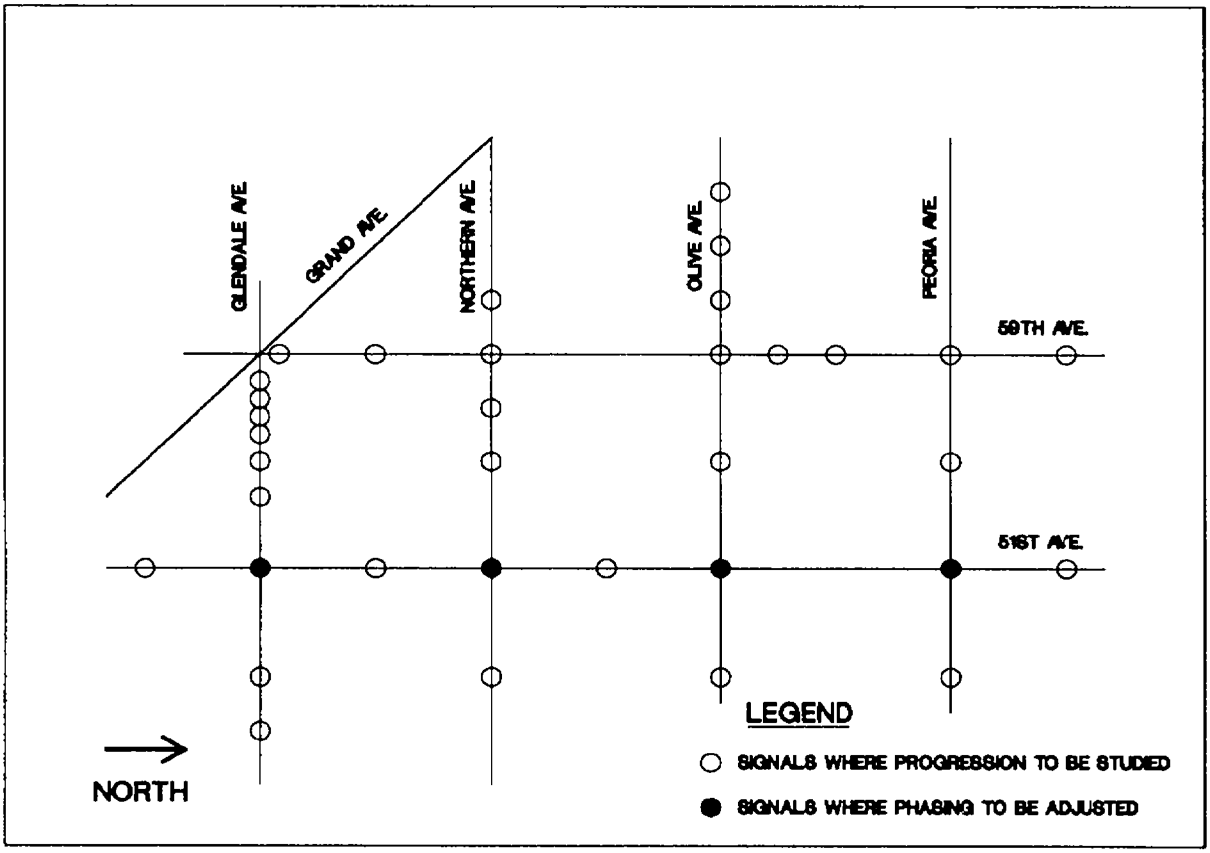

Figure 7-1. Lead-Lag Study Area, City of Glendale

Table 7-5. Travel Time Results, City of Glendale

Route and Time		Weighted Delay (Vehicle - Hours)				Weighted Travel Time (Vehicle Hours)				Weighted Stops (Thousand Vehicle-Stops)			
		Exist. Lead.	FOR. Lead.	FOR. Lag.	FOR. Comb.	Exist. Lead.	FOR. Lead.	FOR. Lag.	FOR. Comb.	Exist. Lead.	FOR. Lead.	FOR. Lag.	FOR. Comb.
51st Ave. NB	AM	7	45	26	42	124	162	143	157	2	6	6	5
51st Ave. NB	MID	20	25	25	12	180	188	186	169	3	3	8	3
51st Ave. NB	OFF	77	40	44	34	475	417	408	405	12	8	8	8
51st Ave. NB	PM	106	81	78	25	413	383	380	317	15	15	12	6
51st Ave. SB	AM	42	64	39	82	359	382	351	385	9	12	9	12
51st Ave. SB	MID	24	49	27	15	186	210	185	170	5	5	5	5
51st Ave. SB	OFF	54	25	22	38	466	426	409	446	12	8	8	8
51st Ave. SB	PM	14	79	90	84	210	285	289	277	6	10	12	10
59th Ave. NB	AM	6	24	19	29	97	115	105	117	3	4	4	5
59th Ave. NB	MID	19	13	17	14	143	131	135	135	3	2	3	3
59th Ave. NB	OFF	44	49	44	55	345	362	333	349	12	8	8	12
59th Ave. NB	PM	53	61	37	33	225	239	195	192	11	9	4	4
59th Ave. SB	AM	26	19	39	21	192	182	206	185	5	2	7	5
59th Ave. SB	MID	22	11	25	9	143	134	153	130	3	3	5	3
59th Ave. SB	OFF	45	43	24	28	320	323	281	297	11	7	7	11
59th Ave. SB	PM	9	37	38	37	92	131	127	123	2	6	6	6
Northern Ave. EB	AM	9	23	16	6	92	106	97	82	4	2	4	2
Northern Ave. EB	MID	14	35	24	16	99	122	105	98	4	5	4	4
Northern Ave. EB	OFF	12	46	20	29	206	230	199	209	4	4	4	4
Northern Ave. EB	PM	28	44	56	48	141	160	174	159	7	11	11	9
Northern Ave. WB	AM	12	48	30	37	98	130	113	119	4	4	4	6
Northern Ave. WB	MID	10	10	9	10	87	86	86	85	3	2	3	3
Northern Ave. WB	OFF	46	3	25	12	274	237	258	243	5	0	5	5
Northern Ave. WB	PM	63	62	29	24	195	186	146	144	11	8	3	5
Olive Ave. EB	AM	18	22	23	58	133	144	140	188	4	4	6	8
Olive Ave. EB	MID	34	18	3	7	176	157	138	136	7	2	2	2
Olive Ave. EB	OFF	52	63	15	8	334	347	286	274	10	10	5	5
Olive Ave. EB	PM	27	55	50	61	168	185	176	195	7	7	7	9
Olive Ave. WB	AM	10	44	29	26	102	150	119	118	2	7	3	2
Olive Ave. WB	MID	26	17	5	7	143	133	120	123	4	2	2	4
Olive Ave. WB	OFF	39	70	35	36	316	359	305	308	10	10	10	10
Olive Ave. WB	PM	70	51	60	34	257	213	226	194	12	6	12	6
Peoria Ave. EB	AM	8	14	9	10	44	50	44	44	1	2	2	1
Peoria Ave. EB	MID	20	7	12	8	114	98	100	99	4	2	2	2
Peoria Ave. EB	OFF	28	22	36	22	218	215	222	210	4	8	8	8
Peoria Ave. EB	PM	18	38	39	37	113	130	137	130	4	4	5	4
Peoria Ave. WB	AM	8	7	9	12	62	59	63	69	2	1	2	3
Peoria Ave. WB	MID	12	11	8	5	90	85	86	81	3	2	3	2
Peoria Ave. WB	OFF	36	35	33	31	219	217	212	211	8	8	8	4
Peoria Ave. WB	PM	11	31	48	46	122	147	161	161	5	5	7	5
Total:		1179	1440	1217	1146	7773	8015	7601	7536	242	222	235	219

The data from these five travel time runs are shown in Table 7-5. Comparisons were made between the different phasing patterns. The comparison between (1) existing leading (2) FORCAST optimized leading (3) FORCAST optimized lagging and (4) FORCAST optimized combination shown in Appendix D with the results shown in Table 7-6.

An equivalent cost/hour which shows an equivalent motorists' cost based upon stopped time delay, travel time and stops is shown in Figure 7-2. This information is broken into the four travel time periods AM peak, midday peak, PM peak, and an off-peak period.

As Table 7-6 suggests, there is a significant difference in travel time and delay between both the FORCAST leading - FORCAST lagging and between the FORCAST leading -FORCAST combination plans. There doesn't seem to be any discernible pattern. If the cost parameter alone is looked at, then it appears that the existing timing plan works best for the AM peak, the combination plan works best for the midday and PM peak, and the lagging plan works best for the off-peak. In the AM peak, the lagging plan also works better than the FORCAST leading or the combination.

It appears, at least from this information, that lagging left turns work best in situations such as an off peak period where left turn volumes are relatively light. In this instance, the extra time that is saved from sometimes avoiding the left turn phase can be given to the through movements resulting in better progression.

Table 7-6. Travel Time Study Comparisons, City of Glendale

Comparison	Least Delay	Level of Significance (p)	Least Travel Time	Level of Significance (p)	Least Stops	Level of Significance (p)
Existing Leading - FORCAST leading	Existing Leading	.07	Existing Leading	.16	FORCAST Leading	.27
Existing Leading - FORCAST lagging	Existing Leading	.08	FORCAST Lagging	.34	Existing Leading	.73
Existing Leading - FORCAST Combination	FORCAST Combination	.86	FORCAST Combination	.27	FORCAST Combination	.26
FORCAST Leading - FORCAST lagging	FORCAST Lagging	.03	FORCAST Lagging	.01	FORCAST Leading	.43
FORCAST Leading - FORCAST Combination	FORCAST Combination	.02	FORCAST Combination	.01	FORCAST Combination	.87
FORCAST Lagging - FORCAST Combination	FORCAST Combination	.47	FORCAST Combination	.58	FORCAST Combination	.29

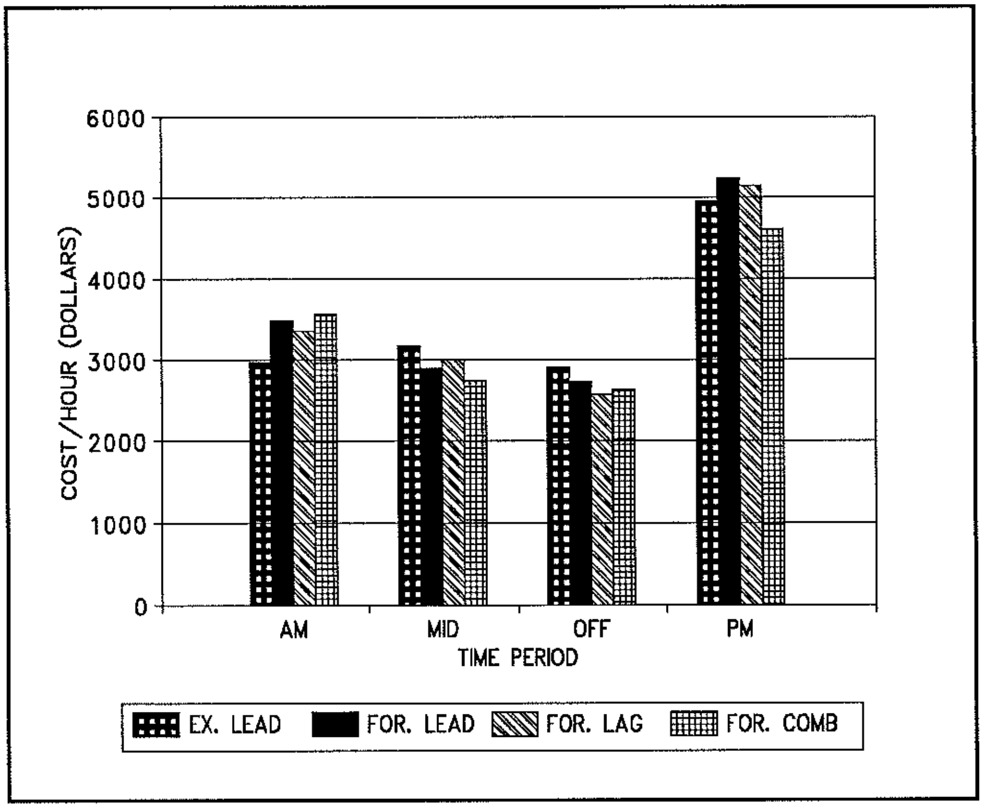

Figure 7-2. Travel Time Study Cost/Hour, City of Glendale

Tempe Travel Time Study

The study area for the Tempe analysis is bounded by Broadway Road to the north, Southern Avenue to the south, 48th Street to the west, and Hardy Drive to the east as shown in Figure 7-3. In this study area, all major arterial - major arterial intersections were operating in protected-permissive leading left turn operation with the exception of the north and south approaches at 48th Street and Broadway. Due to the dual left turns at these approaches they operate in a protected-only leading left turn mode. FORCAST was used to create timing plans for all signals within the study area, however alternate phasing were only implemented at 48th Street and Broadway and 48th Street and Southern.

The routes chosen for this study were the following:

- 48th Street
- Priest Drive
- Southern Avenue
- Broadway Road

The data from these travel time runs are shown in Table 7-7. Comparisons were made between the different phasing patterns. The comparison between (1) existing leading (2) FORCAST optimized leading (3) FORCAST optimized lagging and (4) FORCAST optimized combination is shown in Appendix D and the results from these comparisons are shown in Table 7-8.

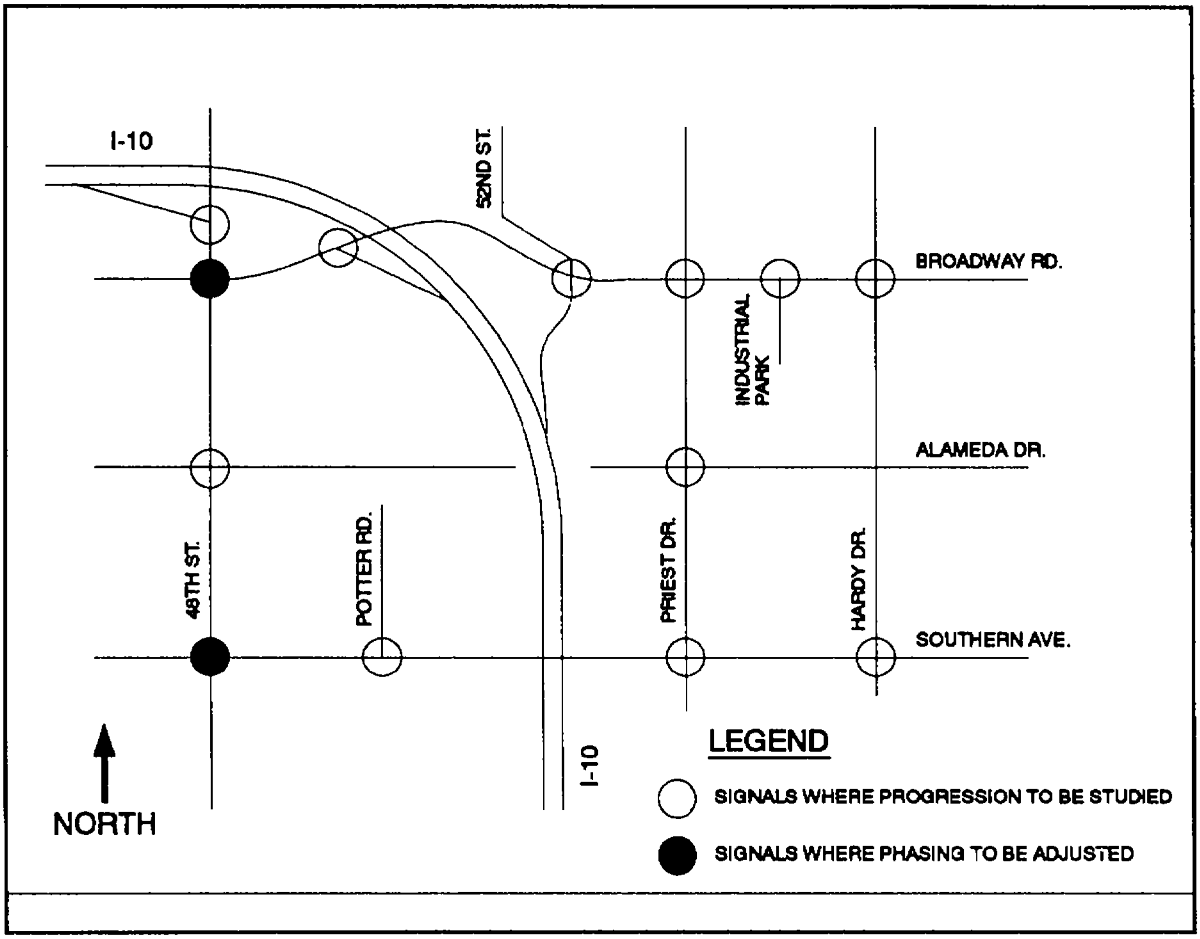

Figure 7-3. Lead-Lag Study Area, City of Tempe

An equivalent cost/hour which shows an equivalent motorists' cost based upon stopped time delay, travel time and stops is shown in Figure 7-4. This information is broken into the three travel time periods AM peak, midday peak and PM peak.

The results shown in Table 7-8 indicate that there is only one significant result in the Tempe travel time data. FORCAST leading had significantly fewer stops than FORCAST lagging.

In viewing Figure 7-4, it is noted that lagging has a higher cost than FORCAST leading or combination in the midday and PM peak, but combination has a higher cost in the AM peak. The cost difference between leading and lagging is least in the AM peak and greatest in the PM peak. In Tempe, at the two intersections where lagging left turns were implemented, there is a very great directional split between left turns at these two intersections in the PM peak. By forcing these two movements together, it has greatly increased the motorists' cost in the PM peak.

Table 7-7. Travel Time Results, City of Tempe

Route and Time		Weighted Delay (Vehicle-Hours) Exist. Lead.	FOR. Lead.	FOR. Lag.	FOR. Comb.	Weighted Travel Time (Vehicle-Hours) Exist. Lead.	FOR. Lead.	FOR. Lag.	FOR. Comb	Weighted Stops (Thousand Vehicle-Stops) Exist. Lead.	FOR. Lead.	FOR. Lag.	FOR. Comb.
48th St.. NB	AM	17	4	3	15	117	101	94	117	3	3	3	7
48th St.. NB	MID	28	4	8	38	164	141	142	176	5	5	5	5
48th St.. NB	PM	25	40	44	42	96	112	117	115	3	3	3	3
48th St.. SB	AM	5	36	22	14	48	79	66	58	1	3	3	4
48th St.. SB	MID	49	14	68	29	246	191	250	203	6	0	12	6
48th St.. SB	PM	44	11	87	48	202	157	239	184	14	5	14	5
Southern Ave. EB	AM	12	16	12	9	55	54	50	47	3	2	2	2
Southern Ave. EB	MID	26	N/A*	18	6	209	N/A*	197	168	9	N/A*	4	4
Southern Ave. EB	PM	54	63	21	5	241	237	199	167	13	9	4	4
Southern Ave. WB	AM	15	58	42	19	149	198	181	151	3	10	10	3
Southern Ave. WB	MID	11	N/A*	47	33	187	N/A*	229	207	0	N/A*	9	5
Southern Ave. WB	PM	23	17	27	13	103	96	104	89	6	2	2	4
Broadway Rd. EB	AM	6	15	14	23	104	110	109	119	2	2	2	7
Broadway Rd. EB	MID	49	106	167	49	483	540	598	466	21	10	21	10
Broadway Rd. EB	PM	22	117	68	120	271	358	310	359	6	17	17	17
Broadway Rd. WB	AM	30	52	34	38	202	208	205	204	12	4	8	8
Broadway Rd. WB	MID	117	72	52	57	550	481	458	467	21	10	10	10
Broadway Rd. WB	PM	100	43	137	79	283	201	301	236	12	4	12	8
Priest Rd. NB	AM	9	31	11	14	79	98	83	85	6	3	3	3
Priest Rd. NB	MID	17	6	3	31	161	149	150	170	6	6	6	11
Priest Rd. NB	PM	8	21	16	25	74	84	82	88	2	2	2	2
Priest Rd. SB	AM	1	4	22	21	34	35	56	53	1	0	1	1
Priest Rd. SB	MID	1	1	1	1	120	130	124	116	0	0	0	0
Priest Rd. SB	PM	19	11	6	16	113	99	99	107	3	3	3	3
Total (Minus Southern Mid)		652	741	865	706	3894	3859	4016	3777	151	104	146	126

* - Not able to collect data due to construction along Southern Ave.

Table 7-8. Travel Time Study Comparisons, City of Tempe

Comparison	Least Delay	Level of Significance (p)	Least Travel Time	Level of Significance (p)	Least Stops	Level of Significance (p)
Existing Leading - FORCAST leading	Existing Leading	.59	FORCAST Leading	.86	FORCAST Leading	.08
Existing Leading - FORCAST lagging	Existing Leading	.16	Existing Leading	.41	FORCAST lagging	.99
Existing Leading - FORCAST Combination	Existing Leading	.69	FORCAST Combination	.43	FORCAST Combination	.35
FORCAST Leading - FORCAST lagging	FORCAST Leading	.47	FORCAST Leading	.37	FORCAST Leading	..05
FORCAST Leading - FORCAST Combination	FORCAST Combination	.78	FORCAST Combination	.56	FORCAST Leading	.13
FORCAST Lagging - FORCAST Combination	FORCAST Combination	.26	FORCAST Combination	.12	FORCAST Combination	.23

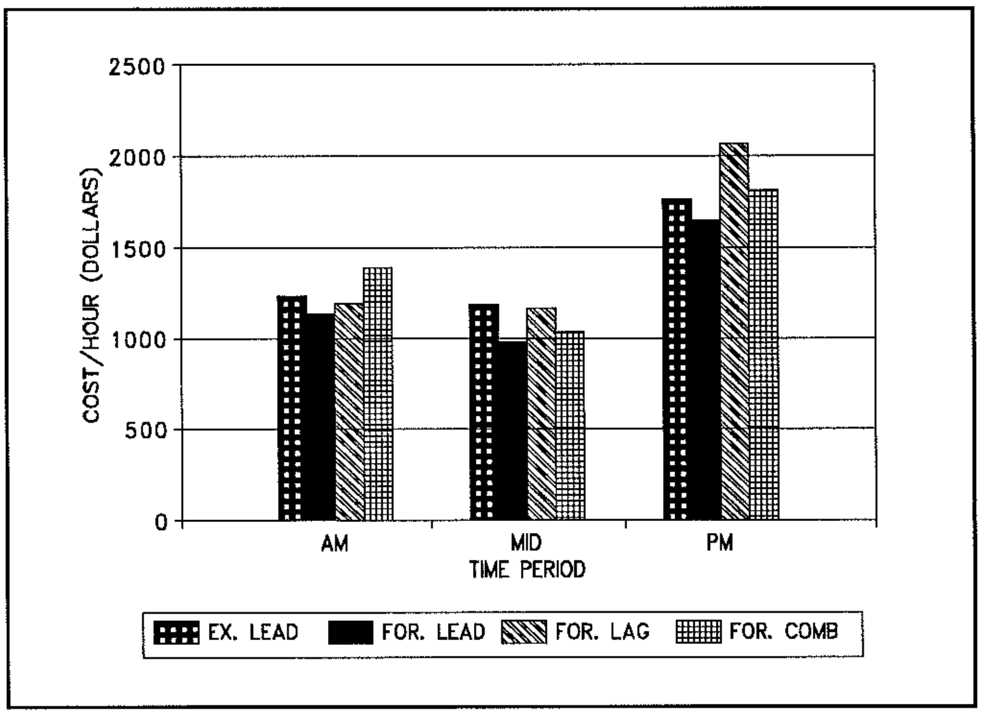

Figure 7-4. Travel Time Study Cost/Hour, City of Tempe

Mesa Travel Time Study

The City of Mesa changed the phasing at Southern and Stewart from Leading east-west to leading east and lagging west. This study area is shown in Figure 7-5. Lee Engineering collected travel time data along Southern Avenue in the AM, Midday and PM peak time periods to determine the effect of this changeover. The results of this change are shown on Table 7-9.

While not significant, these numbers do show a substantial reduction in delay, stops, and travel time due to the changing from an all leading phasing pattern to a combination leading-lagging phasing pattern.

Scottsdale Travel Time Study

In the Spring of 1988, Lee Engineering performed an optimization of the signals within the City of Scottsdale. Optimization was performed using the city's FORCAST computer program. At that time, several travel time runs were performed using the TIMELAPSE Travelog data collector to determine stops, delay, and travel time. These travel time runs were performed by members of the City Council, Transportation Commission and city staff.

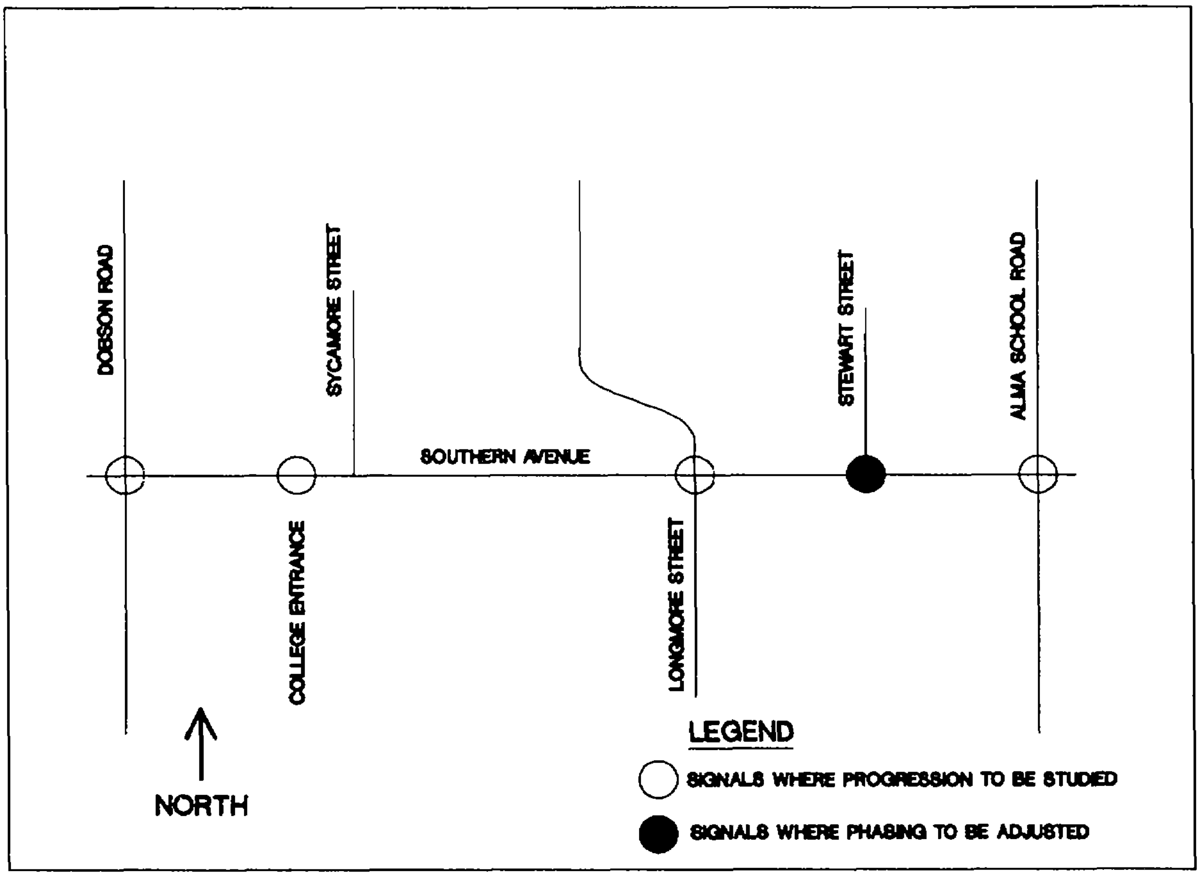

Figure 7-5. Lead-Lag Study Area, City of Mesa

Table 7-9. Travel Time Studies Leading Minus Combination, City of Mesa

		Weighted Delay (Vehicle-hours)			Weighted Travel Time (Vehicle-hours)			Weighted Stops (Thousand Vehicle Stops)		
Route	**Time**	**Leading**	**Lagging**	**Difference**	**Leading**	**Lagging**	**Difference**	**Leading**	**Lagging**	**Difference**
Southern Ave. EB	AM	5	0	5	27	22	6	0.8	0.0	0.8
Southern Ave. EB	MID	52	1	51	129	71	58	5.2	0.0	5.2
Southern Ave. EB	PM	41	8	33	132	90	42	5.9	2.9	2.9
Southern Ave. EB	AM	10	3	8	68	60	8	2.1	2.1	0.0
Southern Ave. EB	MID	5	5	0	76	74	2	2.4	2.4	0.0
Southern Avee EB	PM	0	5	-5	74	66	8	0.0	2.5	-2.5
Total		113	21		506	382		16.4	10.0	
			Sample Size	6		Sample Size	6		Sample Size	6
			Mean Difference	15.197		Mean Difference	20.579		Mean Difference	1.069
			Std Deviation	21.871		Std Deviation	23.436		Std Deviation	2.675
			Test Stat.	1.702		Test Stat.	2.151		Test Stat.	0.979
			Significant	N		Significant	N		Significant	N
			Level of Significance (p)	.15		Level of Significance (p)	.09		Level of Significance (p)	.37

Once the city converted all their signals to lagging phasing, city staff performed another retiming of their signal system in the Spring of 1990. Assuming the volumes along the streets stayed relatively constant over this two year period, another set of travel time runs were performed by city staff.

The results of the leading and lagging phasing is shown in Table D-4 in Appendix D. In some cases, the travel time studies for both the leading and lagging left turns were conducted at different times of the day, specifically the PM peak runs were made between 4 PM and 6 PM in 1988 and generally between 4 PM and 5 PM in 1990. The results shown in Figure F-1 (Appendix F-4) however, have been analyzed for both leading and lagging in those time periods which were common to both plans. This information must be used carefully for several reasons:

- 2 year time period between studies.
- City conversion of numerous intersections from protected to permissive -protected between the two studies.
- Reduction of extension time throughout system.
- Refinement of leading timing patterns after the before studies.
- More recent traffic counts in timing lagging condition.
- Different drivers in before and after studies.

DISCUSSION OF RESULTS

It is difficult to determine if either leading or lagging left turns are a better operation for a given situation. While not statistically significant, lagging left turns appeared to operate better for three time periods in Glendale (based on FORCAST plans).

The combination timing plan worked better than leading or lagging in Glendale for only the Midday and PM peak. In Tempe, the combination was never the lowest-cost plan. This was surprising, for it was felt that by having the opportunity for leading or lagging at a particular intersection would help improve progression. It should be stressed again that the FORCAST timing plan must overcome two obstacles in order to choose lagging left turns for intersection phasing. The fact that it does not recognize left turns made on the permissive period results in it not determining the true best combination plan.

The combination timing plan fared best in Mesa where stops, delay, and travel time were all three reduced substantially. This type of combination phasing is different than those tested in either Glendale or Tempe. The Mesa combination plan was leading eastbound and lagging westbound. In Tempe the phasing tested was leading north-south and lagging east-west. It would appear that a substantial reduction in motorist cost is not very apparent with this type of phasing, but is very apparent with the Mesa phasing. It is important to realize that to implement the Mesa phasing, it is necessary to have either protected only operation, or programmed visibility traffic signal heads as is currently being used in Texas.

In conclusion the following points should be mentioned.

- One of the greatest benefits of lagging left turns is in decreasing the need for a protected left turn phase. This increases the opportunity for larger progression bands through the intersection. In order for a timing program to implement the best phasing, it is necessary for that program to evaluate the left turn movement in conjunction with gaps in the opposing traffic stream. Since FORCAST does not do this, it is not a good program for optimizing the combination phasing. A program which evaluates the gaps in the opposing traffic stream would be a better program. It is likely that FORCAST may pick the wrong timing plans for an area when considering the combination of leading and legging left turns.

- Combination timing seems to work best when leading and lagging are implemented for opposing directions, i.e. leading eastbound and lagging westbound. There does not appear to be much benefit when leading and lagging are implemented for perpendicular directions, i.e., lagging north-south and leading east-west. It is possible however, that the latter condition could result in improvement if the optimization software recognized the left turns made in a permissive manner.

- One benefit for lagging operation may be in locations where left turns are actuated by the first car in the left turn bay. Although this same benefit may be realized in some situations with third car actuation, there are those where sufficient gaps occur in the opposing traffic stream to permit more than two vehicles turning left on the permissive phase. In those cases, lagging would skip the protected phase and 3rd car actuated leading would not. Each time this occurs in a coordinated system, the time saved from omitting the protected phase goes to one of the through green phases thereby increasing the opportunity for improved progression. But, the time saved by omitting the protected phase may be offset by increased delay to through traffic during the simultaneous protected phase when there are few, if any, left turn vehicles on the opposing approach.

- In locations like Tempe, where there is a high directionality with opposing left turn volumes, there is substantial delay associated with lagging operation due to the loss of phase overlap. At locations where lagging is implemented, programmed visibility signal heads might permit phase overlap with permissive-protected operation.

PART IV

PUBLIC PERCEPTION STUDIES

PART IV documents public opinion surveys which were conducted to determine if motorists have a preference for a leading or a lagging operation.

CHAPTER 8 examines the results of public opinion surveys conducted in the Cities of Glendale and Tempe. Both surveys were conducted by taking a random sample of motorists at intersections which had been converted, as part of this research, from leading to lagging left turn operations. The survey instrument chosen for this study was a mail-in questionnaire which contained four questions. The first two questions provided information concerning driver awareness of various left turn operations. The second two questions provided information concerning driver preference for leading or lagging left turn operations. There were 802 responses received from the Glendale questionnaire mailing and 633 from the Tempe mailing. Approximately 49% of the Glendale motorists which responded to the survey indicated the signals are better with leading left arrows, while approximately 42% preferred the lagging operation. The results of the Tempe questionnaire indicated a nearly two-to-one preference for the lagging left arrow operation. One possible reason Tempe motorists may prefer the lagging operation is the close proximity of Tempe and Scottsdale, where lagging left turns are utilized.

CHAPTER 8

PUBLIC AWARENESS AND PERCEPTION

INTRODUCTION

One of the important aspects of this research effort, especially from the perspective of an elected official, deals with the possible preference of either leading or lagging operation. Information concerning motorist preference of leading or lagging left turn operation was obtained through public opinion surveys conducted in the Cities of Glendale and Tempe. Both surveys were conducted by taking a random sample of motorists at intersections which had been converted, as part of this research, from leading to lagging left turn operations.

DATA COLLECTION

The survey instrument chosen for this study was a mail-in questionnaire. A questionnaire containing four multiple choice questions and a space for additional comments was prepared. The questionnaire was designed to be contained on a nine inch by four inch postage paid post card. A copy of the questionnaire used for motorists in the City of Tempe is shown in Figure 8-1. A similar questionnaire was prepared for the City of Glendale survey. The only difference between the two questionnaires were those changes which were necessary to reflect a change in study area location. A cover letter, which accompanied the questionnaire, briefly explained the nature of the study and the various types of left turn operation: leading, lagging, and combination leading and lagging. The first two questions provided information concerning driver familiarity with the study area and left turn operations. The second two questions provided information concerning driver perception of and preference for the various types of left turn operations examined in this research effort.

Two lists, each with 2400 recorded license plates, were generated. The first list was generated from the four intersections converted in the City of Glendale:

- 51st Ave./Glendale,
- 51st Ave./Northern,
- 51st Ave./Olive, and
- 51st Ave./Peoria.

The number of plates recorded was evenly distributed among the four intersections and among the four approaches at each intersection. Ten percent of the plates recorded were from left turning vehicles and 90% from through or right turning vehicles. The sample was also distributed throughout the day with 20% of the plates being recorded between 7:00 and 9:00 AM, 60% between 9:00 AM and 4:00 PM, and 20% between 4:00 and 6:00 PM.

The second list was generated from the two intersections converted in the City of Tempe:

- 48th St./Southern, and
- 48th St./Broadway.

QUESTIONNAIRE

Note: This questionnaire is a part of a research project designed to study traffic signal operation. Your response could help the City of Tempe and the Arizona Department of Transportation improve traffic flow. **Please Respond!** Recently the City of Tempe changed the left turn arrows at 48th Street and Broadway and 48th Street and Southern. There have been three types of left turn arrow operation:

a. All leading left turn arrows (May, 1990 and before)
b. All lagging left turn arrows (early June, 1990)
c. Some leading and some lagging left turn arrows (late June, 1990)

1. How many times per week do you drive in this area?
   ____ 0-5
   ____ 6-10
   ____ 11-15
   ____ 16+
2. Had you noticed this change in left turn arrows?
   ___ yes ___ no
3. Please think about your past and present travel in this area.
   When driving in this area, do you encounter:
   ____ more green lights with all leading left turn arrows
   ____ more green lights with all lagging left turn arrows
   ____ more green lights with some leading and some lagging
   ____ no difference
4. With respect to left turns, are the signals:
   ____ better with leading left turn arrows
   ____ better with lagging left turn arrows
   ____ no difference
5. If one (leading or lagging) is better, why?

   ____

   ____

   ____

   ____

Figure 8-1. Tempe Questionnaire

The Tempe sample was distributed in the same manner as the Glendale sample with the only exception being that the number of plates recorded was evenly distributed between two rather than four intersections.

The license plates were recorded on a typical week day during the time the lagging operation was being tested at these intersections. Each list was searched to eliminate duplicate entries. The Arizona Department of Transportation Motor Vehicle Division then generated two mailing lists of registered vehicle owners for the two samples. Two lists containing approximately 2250 names each were generated from the original two lists of 2400 license plates. The questionnaires were mailed during the time the combination leading and lagging operation was being tested.

ANALYSIS

There were 802 responses received from the Glendale questionnaire mailing. This represents a return rate of approximately 36%. Approximately 100 questionnaires were not deliverable due to incorrect addresses, change of addresses, and similar situations. The results for the Glendale questionnaire are presented in Table 8-1. A 95% confidence interval was calculated, treating each possible response as a binomial parameter. The data indicates a relatively even distribution in the frequency respondents drive in the study area. The data also indicates a high level of recognition of the changes which had occurred to the left arrow operations over the course of the study, with over 85% noticing the change in left turn arrows. Approximately 38% of the Glendale motorists responding indicated they encountered more green lights with the all leading left turn operation. This compares to approximately 16% for the all lagging operation and 24% for the combination leading and lagging operation. Approximately 17% indicated there was no difference in the three types of operations studied. With respect to left turns, approximately 49% indicated the signals are better with leading left arrows, while approximately 42% preferred the lagging operation.

There were 633 responses received from the Tempe questionnaire mailing. This represents a return rate of approximately 28%. Approximately 90 questionnaires were not deliverable. The results for the Tempe questionnaire are presented in Table 8-2. A 95% confidence interval was also calculated. The Tempe data also indicates a relatively even distribution in the frequency respondents drive in the area and a high level of recognition of the changes which had taken place. Approximately 30% of the Tempe motorists responding indicated they encountered more green lights with the all leading operation. This compares to approximately 21% for the all lagging operation and 27% for the combination leading and lagging operation. Again, approximately 17% indicated no difference. With respect to left turns, approximately 30% expressed a preference for the leading operation and approximately 61% preferred the lagging operation.

The two surveys taken together represents a composite of 1435 motorists responses. A composite of the Glendale and Tempe questionnaires is presented in Table 8-3. Nearly twice as many motorists believe they encountered more green lights with the all leading operation as compared to the all lagging operation (34.4% versus 18.3%). However, 25.3% reported they encountered the best progression with the combination leading and lagging operation. With respect to left turns, the composite results indicate a slight motorist preference for the lagging left arrow operation. Approximately 50% expressed a preference for lagging and approximately 41% prefer the leading operation. Approximately 9% expressed no preference.

Table 8-1. Public Awareness and Perception, City of Glendale

Question	Count	Percent	95% Conf. Interval
How many times per week do you drive in this area?			
0-5	218	27.2%	3.1%
6-10	212	26.4%	3.1%
11-15	150	18.7%	2.7%
16+	217	27.1%	3.1%
No response	5	0.6%	0.5%
Had you noticed this change in left turn arrows?			
Yes	691	86.2%	2.4%
No	111	13.8%	2.4%
No response	0	0.0%	0.0%
Please think about your past and present travel in this area. When driving in this area, do you encounter:			
More green lights with all leading left turn arrows	306	38.2%	3.4%
More green lights with all lagging left turn arrows	127	15.8%	2.5%
More green lights with some leading and some lagging	193	24.1%	3.0%
No difference	138	17.2%	2.6%
No response	38	4.7%	1.5%
With respect to left turns, are the signals:			
Better with leading left turn arrows	396	49.4%	3.5%
Better with lagging left turn arrows	333	41.5%	3.4%
No difference	73	9.1%	2.0%
No response	0	0.0%	0.0%
Questionnaires Mailed:	2250		
Number of Responses:	802	36%	

Table 8-2. Public Awareness and Perception, City of Tempe

Question	Count	Percent	95 % Conf. Interval
How many times per week do you drive in this area?			
0-5	160	25.3%	3.4%
6-10	189	29.9%	3.6%
11-15	121	19.1%	3.1%
16+	156	24.6%	3.4%
No response	7	1.1%	0.8%
Had you noticed this change in left turn arrows?			
Yes	513	81.0%	3.1%
No	102	16.1%	2.9%
No response	18	2.8%	1.3%
Please think about your past and present travel in this area. When driving in this area, do you encounter:			
More green lights with all leading left turn arrows	187	29.5%	3.6%
More green lights with all lagging left turn arrows	135	21.3%	3.2%
More green lights with some leading and some lagging	170	26.9%	3.5%
No difference	106	16.7%	2.9%
No response	35	5.5%	1.8%
With respect to left turns, are the signals:			
Leading			
Better with leading left turn arrows	187	29.5%	3.6%
Better with lagging left turn arrows	383	60.5%	3.8%
No difference	54	8.5%	2.2%
No response	9	1.4%	0.9%
Questionnaires Mailed:	2250		
Number of Responses:	633	28%	

Table 8-3. Public Awareness and Perception, Composite

Question	Count	Percent	95 % Conf. Interval
How many times per week do you drive in this area?			
0-5	378	26.3%	2.3%
6-10	401	27.9%	2.3%
11-15	271	18.9%	2.0%
16+	373	26.0%	2.3%
No response	12	0.8%	0.5%
Had you noticed this change in left turn arrows?			
Yes	1204	83.9%	1.9%
No	213	14.8%	1.8%
No response	18	1.3%	0.6%
Please think about your past and present travel in this area. When driving in this area, do you encounter:			
More green lights with all leading left turn arrows	493	34.4%	2.5%
More green lights with all lagging left turn arrows	262	18.3%	2.0%
More green lights with some leading and some lagging	363	25.3%	2.2%
No difference	244	17.0%	1.9%
No response	73	5.1%	1.1%
With respect to left turns, are the signals:			
Leading			
Better with leading left turn arrows	583	40.6%	2.5%
Better with lagging left turn arrows	716	49.9%	2.6%
No difference	127	8.9%	1.5%
No response	9	0.6%	0.4%
Questionnaires Mailed:	4500		
Number of Responses:	1435	32%	

DISCUSSION OF RESULTS

The results of the Glendale and Tempe surveys indicate that the largest proportion of motorists feel they encountered more green lights with the all leading left arrow operation. The composite results indicate 34.4% of the motorists responding believe they encounter more green lights with the all leading operation. This would tend to indicate that a greater number of motorists perceive traffic flow is better with the all leading left arrow operation. The two surveys are also consistent with regard to the perception of the relative merits of a combination leading and lagging left arrow operation. Both surveys indicate a greater number of motorists encountered more green lights with the combination phasing than with the all lagging operation.

The two surveys are not consistent with regard to motorist preference of leading or lagging left arrows. The Glendale survey indicates that, with respect to left turns, the leading left arrow sequence is favored. However, Tempe motorists expressed a preference for the lagging operation. One possible reason Tempe motorists may prefer the lagging left arrow operation is the proximity of the City of Scottsdale and the City of Tempe. The City of Scottsdale implemented lagging left arrows on a city-wide basis in early 1989. City of Tempe motorists are much more likely to drive the streets of Scottsdale than are City of Glendale motorists. Therefore, City of Tempe motorists were in a better position to adjust to the conversion to a lagging operation. A number of City of Tempe motorists included references to the City of Scottsdale in their comments.

The comments were generally instructive. A common theme for those who preferred the leading left arrow operation was consistency and driver expectancy. Lane blockage due to queued left turn vehicles was also a comment commonly made by those who expressed a preference for the leading operation. Safety was a reason given by both those who preferred leading and those who preferred the lagging operation. Some people simply commented on the need for left turn arrow phasing in general.

Finally, it should be noted that this sampling technique does not guarantee an unbiased sample of motorists in the Cities of Glendale and Tempe. The fact that the survey required the respondents to take the initiative to fill out and mail the questionnaire introduces some bias to the sample. But it does provide some indication of the sentiments of those people who feel as though the issue is important enough for them to take the time to fill out the questionnaire and return it. There is no reason to believe this group of people would be more inclined than the public at large to favor a leading or lagging left arrow operation. Therefore, it is felt that this sample does fairly represent the sentiments of motorists in the Cities of Glendale and Tempe.

PART V

CONCLUSIONS

PART V presents the results and conclusions of the research project.

CHAPTER 9 presents the results of the four research questions identified at the initiation of the project. It was found that intersection delay is significantly greater with lagging left turn operation. No significant change in total delay was found with third car actuation of leading protected left turn operation. No significant difference in progression was found between leading, lagging and mixed operation. No significant difference was found in accident experience between leading and lagging operation. There was a mixed response from the motorist preference survey. Glendale drivers felt left turns were better with leading while Tempe drivers felt it was better with lagging.

CHAPTER 10 presents some observations made by the research team during the conduct of the study.

CHAPTER 11 identifies future work which would be of value in this research area.

CHAPTER 9

STUDY RESULTS

In response to the five research questions stated in Chapter 1, the results of the study are as follows:

1. *Is there a difference in intersection delay at isolated intersections between leading and lagging operation?*

   The results of this study indicate significantly greater delay per approach vehicle occurs with lagging operation than with leading operation for the intersections and time periods tested. The Phoenix area studies reflected a 42% increase in delay conversion per approach vehicle with conversion from leading to lagging operation. The same conversion in Pima County resulted in a 30% increase in delay per approach vehicle. It is important to note that the time periods tested were generally PM peak hour conditions. These would not be as likely to have sufficiently low left turn and through volumes to eliminate many protected left turn phases in their lagging condition. It is conceivable that in off peak conditions more of the left turns could be made in a permissive manner therefore skipping the protected left turn phase. Eliminating the protected phases would likely reduce intersection delay.

   Intersection delay was also collected for test intersections with both first car and third car actuation. Although there was no significant difference between the two, this test also was only done in the PM peak hour condition. The probable benefit of third car actuation on intersection delay is most likely in off peak conditions.

2. *Is there a difference in signal progression among leading only, lagging only and mixed operation?*

   There were no statistically significant differences in stops, delay or travel time with the different operating conditions. Additionally, the large number of signal timing optimization runs required to evaluate all combinations of leading and lagging operation makes for a cumbersome, time consuming process. The requirement that the Glendale and Tempe "mixed" operation was limited to either both leading or both lagging on the same street in order to avoid the "trap" restricted potential progression benefit. An additional limitation was that only four of eight multi-phase Glendale intersections and two of four multi-phase Tempe intersections were considered for change to lagging.

   The most promise for benefit from lagging or mixed operation was found in the Mesa study where leading left turn operation was utilized for eastbound traffic and lagging for westbound traffic in the after condition. This was the operation which provided the best east-west progression. This mixed operation was possible without the trap condition because of the use of protected only left turns.

3. *Is there a need to have consistent left turn phasing (leading or lagging) within any given city, urbanized area and throughout the State of Arizona?*

No evidence was found in this study supporting the need to have the same phasing consistency. Although many of the cities prefer consistency within their jurisdiction, a straw poll of the representatives on the Advisory Committee found unanimous agreement that the state should not pass legislation mandating either operation everywhere.

4. *Is there a difference in accident experience between leading and lagging operation?*

   In all three accident studies - Tucson, Pima County and Scottsdale, there was no significant difference in left turn accident history between leading and lagging operation.

5. *Is there a motorist preference between leading and lagging operation?*

   Table 9-1 presents a summary of the information obtained in the motorists' survey. Lagging left turns seem to be more favorably received in Tempe than in Glendale. This could possibly be due to the close proximity of Tempe to Scottsdale, where lagging left turns are utilized.

Table 9-1. Public Perception Results, Phoenix Area

Question	City	
More Green Lights With:	**Glendale**	**Tempe**
Leading	38%	30%
Lagging	16%	21%
Combination	24%	27%
No Difference / No Response	22%	22%
Left Turns Better With:		
Leading	49%	30%
Lagging	42%	61%
No Difference / No Response	9%	10%

CHAPTER 10

THEORETICAL ANALYSIS OF LEADING AND LAGGING LEFT TURNS

INTRODUCTION

As a result of this study, an increased knowledge of the subtle aspects of leading verses lagging left turns was gained both by the research team and the Advisory Committee. Many of the observations made were not supported by statistical analysis due to limited sample sizes or other factors; however, it was believed to be important to try to identify some of the issues related to the question of leading and lagging left turns. This chapter is intended to serve that purpose.

The value of the field data collection and the analyses of that data can be emphasized by the fact that it provided insight to the understanding of the many variables which influence left turn operations. Within the scope of this study and the conditions at the study sites, it was not possible to collect data for all possible combinations of the pertinent variables. The results of the field studies together with a somewhat theoretical analysis yield a comprehensive assessment of leading and lagging left turn operations. This section identifies a number of variables that have an impact on the effectiveness of left turn alternatives.

The variables that should be evaluated when considering leading or lagging left turn operations are generally associated with the signal system, traffic characteristics, as well as the driver. The effectiveness of the application of leading or lagging left turns then becomes a function of the conditions at a specific intersection or location.

Table 10-1 presents a general summary of the variables that should be considered in comparing leading and lagging left turn operations. The variables listed in Table 10-1 are related to operational aspects of the problem area. It must be recognized that safety impact is the result of the decisions

Table 10-1. Decision Variables for Comparing Leading and Lagging Left Turn Operations

Signal Control	
	Application of actuated control
	Fixed versus variable cycle length
	Left-turn operations
	Use of phase overlap
Network Considerations	
	Offset requirements
	Allocation of unused green time
Traffic Characteristics	
	Approach volumes
	Directional distribution of opposing flows
	Acceptable gaps in opposing flows
	Peak versus off peak volume variations
Driver Perception	
	Need for uniformity
	Driver compliance and acceptance

related to many of the variables. For this reason, safety was not included in the list. The discussion that follows provides a more detailed explanation of the variables.

SIGNAL CONTROL

The variables associated with signal control focus on the traffic signal operation at a specific intersection. Generally, these variables are related to use of green time or have an impact on the application and allocation of green time. Included in this category are:

- application of actuated control,
- fixed versus variable cycle length,
- left turn operation,
- use of phase overlaps, and
- lost time.

This discussion assumes that exclusive left turn signals are operated on an actuated basis; thus the issue is whether the through and right turn movements are also operated on actuated control. If the intersection operates with full actuated control, there is the opportunity for the skipping of phases and the adjustment of green time in relation to the traffic demand. These factors will ultimately affect the effectiveness of the left turn operations.

Obviously, full actuated control results in a variable cycle length unless a background cycle is utilized; thus the advantage is that the signal will operate in response to the demand with the potential of reducing wasted green time. As a result, inefficiencies in left turn operations can potentially be offset by efficiencies from the actuated control. With the fixed cycle length, the need and duration of the left turn arrow can have a different result.

The left turn operation variable reflects the specific operation of left turn movements. In essence, the issue is whether left turns occur a) only in a protected phase or b) on a permitted basis in conjunction with a protected phase. One of the noted advantages of the lagging left turn is that the left turn demand may be satisfied by permitted movements; thus the need or duration of the protected movement is reduced. This advantage would not be available for the protected only operation.

One of the major advantages of the leading left turn operation is associated with the use of phase overlaps. In essence, once the minor left turn movement is satisfied, the through traffic on the approach of the heavier left turn movement is released. Given differences in opposing directional flows, the phase overlap can increase the efficiency of the signal operation. There is concern about the "left turn trap" that is created with the application of phase overlaps with lagging left turns. For this reason, jurisdictions frequently operate lagging left turns simultaneously without a phase overlap. Some work is currently being done with signal displays which will potentially eliminate the left turn trap problem. Nevertheless, the use of phase overlaps is a major consideration in terms of signal operation.

One of the elements in the analysis of intersection capacity is lost time. Basically, this is the time during a cycle that is lost due to start up delay or clearance time. Generally, the total lost time in a cycle is a function of the signal phasing. With the leading left turns and phase overlaps, there is less lost time during a cycle than with simultaneous lagging left turns. With the lagging left turn operation, there are more situations that require the stopping and restart of traffic streams.

NETWORK CONSIDERATIONS

The variables associated with network considerations reflect signal coordination concerns. In this group, the variables are:

- offset requirements, and
- the allocation of unused green time.

In the coordination of signal networks, one of the concerns is the offset, or the time difference between the beginning of the green phase at successive intersections. Because of this concern, the issue in the application of the exclusive phase is related to the release of the through traffic at a given intersection. With the leading left turn, the through movement will be released early if the left turn phase is not fully utilized. This means that the through traffic will arrive early at the next intersection; thus the progression is affected.

It has been argued that with the lagging left turn, the duration of the left turn phase does not influence the start of the green phase. In theory, this should resolve the problem of controlling the offset; thus the lagging left turn offers an advantage.

For some network configurations, it is possible that a mixed operation of leading and lagging left turns may provide improved network coordination. The mixed operation could use leading left turns in one direction and lagging left turns in the other or simply mix the use of leading and lagging left turns depending on the offset requirement.

A factor that complicates the signal coordination problem is the allocation of unused left turn green time. If the left turn phase is not fully utilized, then the unused time for that phase must be allocated to some other phase if a fixed cycle length is maintained. With the leading left turn, the through traffic is generally released early; thus the offset with the next intersection is affected.

The allocation of unused green time with the lagging left turn will vary depending on the operation of the signal by a jurisdiction. Some jurisdictions will add the unused left turn time to the beginning of the next phase. In essence, unused left turn time on the main street will be added to the beginning of the side street through movement. Also, the unused side street time will be added to the beginning of the main street through green phase. Other jurisdictions may accumulate all unused time and add it to the beginning of the main street through movement. In either case, it results in the possible early release of the through movement which disrupts the planned offset.

Basically, both types of left turn operations have a potential adverse impact with respect to the planned offset. For a given location, it is necessary to evaluate the probability of having unused green time and the effect on network operations.

TRAFFIC CHARACTERISTICS

Whether the intersection signals utilize left turn actuations only or include actuated control for some or all through movements, there are a number of traffic variables that can influence the left turn operations. These variables include:

- approach volumes,
- directional distribution of opposing flows,
- acceptable gaps in opposing flows, and
- peak versus off peak volume variations.

The approach volumes reflect the general magnitude of the traffic movements as well as the percentage of left turns at a given intersection approach. Basically, the magnitude of the approach volumes will have an impact on the need for the protected left turn movement. As the volumes increase at an intersection, it is less likely that the left turn demand can be accommodated by the permitted operation. For this reason, the probability that the lagging left turn phase will not be needed decreases. In addition, increases in volumes will potentially increase the probability that the left turn phase will be fully utilized.

The directional distribution of the opposing flows affects the possibility of phase overlap operation as well as the advantage of the phase overlap accommodating the through movements. With balanced left turn and through movement approach volumes, the advantage of the phase overlap with the leading left turn operation is eliminated. With the elimination of phase overlap operation, the efficiency for the leading operation is similar to the lagging operation.

The number of acceptable gaps in traffic opposing a permitted left turn movement will influence the permitted left turn capacity. For both a protected/permissive and a permissive/protected operation, the larger the permitted left turn capacity, the less time which must be dedicated to the protected left turn phase. A large left turn volume coupled with a large permitted left turn capacity may overcome some of the disadvantages associated with a lagging operation under the conditions of high directionality. With a lagging operation, the permitted left turn capacity is first fully utilized then the protected phase is used to satisfy the residual left turns. The opposite is true for a leading operation, where the capacity of the protected phase is first exhausted. An accurate evaluation of the permitted left turn capacity is much more critical to the efficient timing of the lagging left arrow operation.

Typically, traffic engineers are concerned with the peak conditions; however the off peak traffic volumes may yield a different set of results in terms of demands on signal operations. For example, the left turn demand in the off peak periods could possibly be satisfied without the need for protected turn phases. With this condition, there would be a potential advantage to the lagging left turn operation due to the fact that the left turn phase would only be used if needed. Similarly, third car actuation could result in more delay reduction in the off peak periods, than in the peak periods. Thus, it is possible that the operation of the intersection could be improved. One of the factors to be considered, therefore, is the difference in peak and off peak traffic conditions. Although traffic engineers are typically concerned with the peak conditions, they should be just as concerned with what happens during the other 22 hours of the day.

DRIVER PERCEPTION

An important element in the operation of any signal system is the perception of the driver. Variables related to driver perception are:

- the need for uniformity, and
- driver compliance and acceptance.

One of the basic considerations for the application of traffic control devices is uniformity. Certainly, there are arguments that leading and lagging left turn operations should not be mixed because of the lack of uniformity. On the other hand, some mixed operations even in Arizona apparently go unnoticed by the motoring public. If the driving population perceives the need for uniform left turn operations, then the effectiveness of mixing the operations will potentially be affected. The true question to be resolved is the importance of uniformity for a particular area.

While not totally unrelated to the uniformity issue, another driver variable is compliance and acceptance. For example, if the drivers comply with the signal display, uniformity may not be as great an issue. Anticipation on the part of the driver, as a result of uniformity, can be dangerous as well. Certainly, it is likely that differing driver populations may yield differing responses in terms of compliance and acceptance.

CHAPTER 11

RECOMMENDATIONS FOR FUTURE WORK

Through the conduct of this study, several questions have arisen which are outside the scope of this project. These are identified here as possible items for future research.

- *Effectiveness of lagging* left turns and third-car actuation in off-peak conditions. The studies of lagging and third car operation conducted in this project were primarily during the PM peak periods. A major potential benefit of both lagging left turns and third car actuation lies in the possibility of eliminating protected left turn phases. In a lagging operation, if the left turns can be accommodated on the permissive green thereby eliminating the protected phase, a reduction in intersection delay should result. Similarly, if fewer than three cars arrive prior to the beginning of a protected left turn phase, delay should be reduced with third car actuation. For this reason evaluation of off-peak delay comparing leading with lagging left turns and third car actuation with first car actuation would be valuable.

- *Effectiveness of leading* left turns in one direction and lagging left turns in opposing direction. As tested at one Mesa intersection, there is the potential for improvement to progression at locations where the platoons arrive at different times to have the left turns lead in the direction the platoon first arrives and lag in the opposing direction. The sample size in this research project was inadequate to make a conclusive statement, however, the Mesa results were very promising.

- *Feasibility of overcoming* "trap" but allowing combination of permissive leading or lagging with phase overlap. Current experimentation in Texas is evaluating the use of a 5 section programmed visibility head (Dallas signal face display) for left turn drivers. This head would continue to display a circular green indication to left turn drivers whose concurrent through movement is terminated but whose opposing through movement continues. Although this method has good results (14) the signal display violates section 4B-12 (3.2.)(15, p.4B-12), of the MUTCD which states:

  2. ***During the permitted left turn movement, all signal indications on the approach shall display all a.c. CIRCULAR GREEN indication.***

  A formal request has been made by the Texas State Department of Highways and Public Transportation to revise the MUTCD to allow the Dallas signal face display. Appendix G shows the typical phasing and special (Dallas) phasing being tested.

- *Signal optimization software* which evaluates options of combinations of leading and lagging left turns and which predicts protected lagging phase duration based on permissive left turn phase capacity. Although PASSER II will evaluate the combination of leading and lagging left turns on arterial streets, the research team is unaware of software which will accomplish this for a grid. In this research project the FORCAST optimization software developed by Computran Systems, Inc. was used with all possible combinations of leading and lagging left turns within the constraints of either both leading or both lagging on the two approaches of the same street. In the lagging situation however, FORCAST does not consider the permissive left turns which can be made during the through movements, therefore it overestimates the green time required for the protected lagging phase. This results in a higher cost prediction than actually would result. It is impossible therefore to determine the true lowest cost phase combination. A software program should be developed which:

  - Evaluates all possible combinations of leading and lagging left turns for a grid, and,

  - Includes a gap acceptance algorithm which will predict the number of left turns which can be made during a permissive period and the resulting required protected lagging phase durations.

- *Evaluation of "trade off" of uniformity* verses the benefit of varying left turns between leading and lagging under different conditions. There is a perceived value of having a uniform left turn treatment. It is unlikely that this could or should be made completely uniform throughout the country, state or even an individual city. On the other hand, it is logical that the greatest efficiency of system operation (i.e. least delay) would come from the ability of varying between leading and lagging left turns not only from intersection to intersection, but also from approach to approach at a given intersection. The optimum performance likely would result from analyzing all combinations of leading and lagging left turns at each approach for each time period evaluated. In this case, an approach might be leading in one period of the day and lagging in another. Figure 11-1 graphically portrays this trade off. This analysis would necessitate evaluating a very large number of combinations of phase options. It may be desirable to develop software which would perform a two step optimization process.

  - Coarse level analysis to determine best combination of leading and lagging left turn phase, and

  - A more detailed analysis similar to TRANSYT 7F

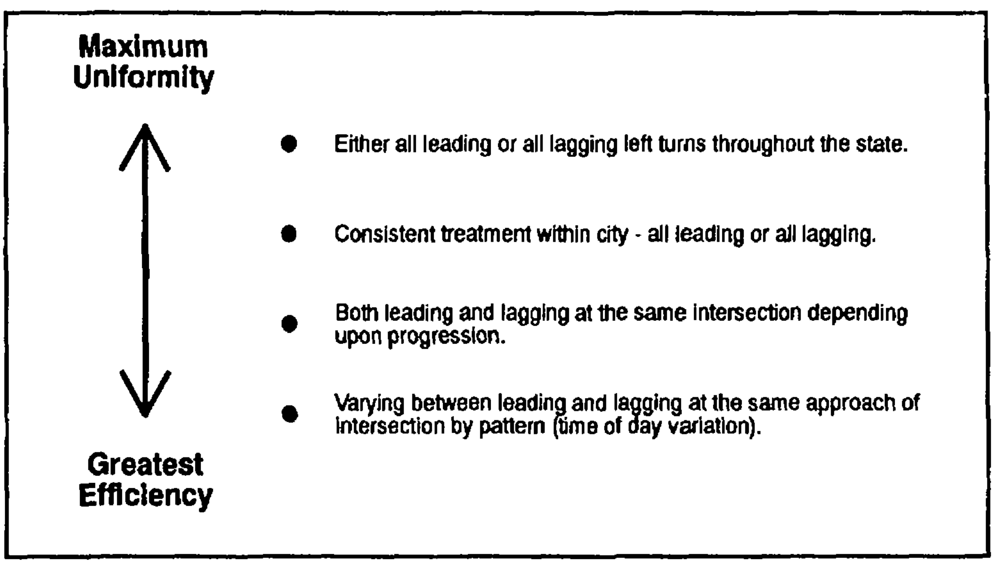

Figure 11-1. Hierarchy of Left Turn Uniformity

- *Additional work* would include an evaluation of driver expectancy of left turns vary between leading and lagging by time of day. As stated previously, it is expected that a reduction in delay should result from considering all possible combinations of phasing for each time period being optimized. A study should be made of possible driver confusion resulting from such a treatment.

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APPENDIX A

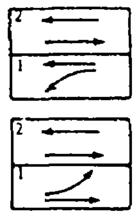
PHASE SELECTION GUIDELINE FOR LEFT-TURN PHASES

APPENDIX A

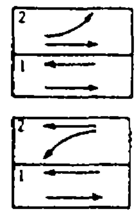
PHASE SELECTION GUIDELINE FOR

LEFT-TURN PHASES

LAGGING LEFT

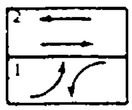


LEADING LEFT

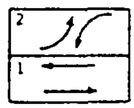


- Best Suited to T-Intersections or where opposing left-turns are prohibited.
- Can be used with either pretimed or actuated control.
- Use should be kept to a minimum where no left turn lane exists due to possibility of one movement blocking the other.

LEADING DUAL LEFT-TURN

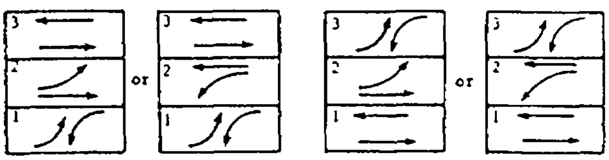


LAGGING DUAL LEFT-TURN



BASIC THREE PHASE

- Simplest and least expensive.
- Can be provided with pretimed or single ring actuated controllers.
- Should be kept to minimum since both left-turns receive exactly the same green time.
- Lagging left should only be used when system operation needs dominate safety consideration.



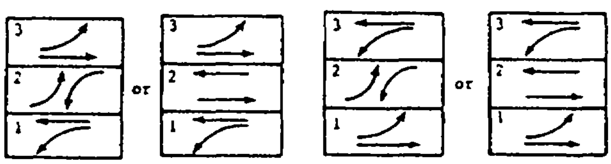
OVERLAP PHASING

- Best suited to dual ring actuated controllers.
- Most efficient method of providing protected only left-turn phase.
- Through movement is allowed to move when there is no opposing left-turn traffic.

LEAD-LAG LEFT-TURN PHASING

SPLIT PHASING

- Best when left-turn volume is very heavy in both directions and is equal or greater than the companion through movement.
- Especially effective when one of the through lanes must be used as an optional left-turn lane or where a separate left-turn lane can not be provided.



OVERLAP PHASING

- Either pretimed or dual ring actuated controller can be used.
- As efficient as dual left with overlap phasing.
- Should be used only for interconnected signals where it's use will increase the width of the progressive band.

APPENDIX B

SCOTTSDALE ACCIDENT ANALYSIS

TABLE B-1

SCOTTSDALE ACCIDENT ANALYSIS

TEST INTERSECTION LEFT TURN ACCIDENT SUMMARY

					1985							
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS						0	0	0	0	0	0	0
SCOTTSDALE/THOMAS						2	1	0	2	1	0	1
HAYDEN/McDOWELL						0	0	0	0	1	1	0
MILLER/INDIAN SCHOOL						1	1	1	1	0	0	0
68TH STREET/INDIAN SCH						0	0	0	1	0	0	0
HAYDEN/CHAPARRAL						0	0	0	1	0	0	0
SCOTTSDALE/CAMELBACK						0	0	0	0	0	0	0
HAYDEN/McDONALD						0	1	0	1	1	0	1
PIMA/SHEA						0	0	0	0	0	0	0

					1986							
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0	0	0	0	0	0	0	0	0	0	0
SCOTTSDALE/THOMAS	1	1	1	2	0	1	2	2	4	0	0	1
HAYDEN/McDOWELL	0	1	0	0	0	0	0	0	0	0	0	0
MILLER/INDIAN SCHOOL	2	0	0	1	0	1	1	0	0	0	0	2
68TH STREET/INDIAN SCH	1	1	0	0	0	1	0	0	0	0	0	0
HAYDEN/CHAPARRAL	0	0	0	0	0	0	1	0	0	0	0	0
SCOTTSDALE/CAMELBACK	0	0	0	0	0	0	0	0	0	0	0	0
HAYDEN/McDONALD	0	0	1	1	0	0	0	0	0	0	0	0
PIMA/SHEA	0	0	0	0	0	0	0	0	0	0	0	0

					1987							
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0	0	1	0	0	0	0	0	0	0	0
SCOTTSDALE/THOMAS	0	0	4	1	1	1	1	2	0	2	0	3
HAYDEN/McDOWELL	0	0	1	1	0	0	0	0	0	1	1	0
MILLER/INDIAN SCHOOL	0	0	1	0	1	0	0	1	0	0	0	1
68TH STREET/INDIAN SCH	0	0	0	1	0	0	0	0	0	0	0	0
HAYDEN/CHAPARRAL	0	0	0	0	0	0	0	0	0	1	0	0
SCOTTSDALE/CAMELBACK	0	0	0	0	0	0	0	0	0	0	0	0
HAYDEN/McDONALD	0	0	0	0	0	0	0	0	0	0	0	0
PIMA/SHEA	0	0	0	0	0	0	0	0	0	0	0	0

TABLE B-1

CONTINUED

1988												
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0	0	0	0	0	0	0	0	0	0	0
SCOTTSDALE/THOMAS	0	1	3	0	3	1	2	1	0	2	1	0
HAYDEN/McDOWELL	0	0	1	0	0	1	1	0	0	0	1	1
MILLER/INDIAN SCHOOL	0	1	0	1	1	1	0	0	1	2	0	1
68TH STREET/INDIAN SCH	0	0	0	0	0	0	1	0	0	0	0	0
HAYDEN/CHAPARRAL	0	0	0	1	0	0	0	0	0	0	0	0
SCOTTSDALE/CAMELBACK	0	0	0	0	1	0	0	0	1	0	0	0
HAYDEN/McDONALD	1	0	0	0	0	0	0	0	1	0	0	0
PIMA/SHEA	0	0	1	0	0	0	0	0	0	0	0	0

1989												
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0	0	0	0	0	0	0	0	0	0	0
SCOTTSDALE/THOMAS	0	1	0	1	0	1	1	1	0	0	0	0
HAYDEN/McDOWELL	0	0	0	0	0	0	0	1	0	1	0	0
MILLER/INDIAN SCHOOL	0	0	0	0	1	0	1	1	0	0	1	0
68TH STREET/INDIAN SCH	0	0	0	0	0	0	1	0	0	1	0	0
HAYDEN/CHAPARRAL	0	0	0	0	0	0	0	0	0	1	0	0
SCOTTSDALE/CAMELBACK	0	0	0	0	0	0	0	0	0	0	0	0
HAYDEN/McDONALD	0	1	0	0	0	0	0	1	1	0	0	0
PIMA/SHEA	1	0	0	0	0	0	0	0	0	0	0	0

1990												
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0										
SCOTTSDALE/THOMAS	1	1										
HAYDEN/McDOWELL	1	1										
MILLER/INDIAN SCHOOL	1	1										
68TH STREET/INDIAN SCH	0	0										
HAYDEN/CHAPARRAL	0	0										
SCOTTSDALE/CAMELBACK	1	1										
HAYDEN/McDONALD	0	0										
PIMA/SHEA	0	0										

TABLE B-2

SCOTTSDALE ACCIDENT ANALYSIS

TEST INTERSECTION TOTAL ACCIDENT SUMMARY

	1985											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS						0	0	0	0	0	0	0
SCOTTSDALE/THOMAS						2	4	2	4	1	0	3
MILLER/INDIAN SCHOOL						3	1	2	2	1	0	1
HAYDEN/McDOWELL												
68TH STREET/INDIAN SCH												
HAYDEN/CHAPARRAL												
SCOTTSDALE/CAMELBACK												
HAYDEN/McDONALD												
PIMA/SHEA												

	1986											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0	0	0	0	0	0	0	0	0	0	0
SCOTTSDALE/THOMAS	0	0	0	0	0	0	0	3	5	1	4	4
MILLER/INDIAN SCHOOL	2	4	1	5	1	5	2	0	1	0	1	3
HAYDEN/McDOWELL	0	1	0	1	2	0	0	0	4	4	3	2
68TH STREET/INDIAN SCH	2	3	2	0	1	2	1	1	0	1	1	3
HAYDEN/CHAPARRAL	0	1	0	0	1	1	2	0	2	1	1	0
SCOTTSDALE/CAMELBACK	3	3	1	1	2	0	1	0	0	1	2	0
HAYDEN/McDONALD	1	0	2	1	2	0	0	0	0	2	0	1
PIMA/SHEA	0	0	1	1	0	0	0	0	0	0	1	2

	1987											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	2	0	0	1	0	0	1	0	0	0	0	0
SCOTTSDALE/THOMAS	2	3	6	3	3	2	3	3	1	3	1	3
MILLER/INDIAN SCHOOL	0	1	2	0	1	1	0	2	1	0	0	1
HAYDEN/McDOWELL	2	0	1	3	2	3	3	0	2	2	3	1
68TH STREET/INDIAN SCH	0	2	1	3	2	1	1	2	0	0	1	3
HAYDEN/CHAPARRAL	1	1	1	0	0	2	2	0	0	2	1	0
SCOTTSDALE/CAMELBACK	1	1	1	3	0	0	0	0	0	0	2	2
HAYDEN/McDONALD	1	2	0	2	0	0	2	2	4	0	3	0
PIMA/SHEA	0	0	0	1	0	3	0	1	0	0	0	0

TABLE B-2

CONTINUED

1988												
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0	0	0	0	0	0	1	0	0	0	0
SCOTTSDALE/THOMAS	2	1	4	1	3	3	2	1	1	3	2	2
MILLER/INDIAN SCHOOL	0	4	0	2	3	1	1	1	1	3	2	2
HAYDEN/McDOWELL	1	0	3	2	0	3	5	0	0	0	3	4
68TH STREET/INDIAN SCH	2	3	0	2	0	0	1	0	0	4	2	0
HAYDEN/CHAPARRAL	1	3	0	1	1	0	0	1	0	3	0	0
SCOTTSDALE/CAMELBACK	0	0	0	1	1	4	1	1	3	4	0	1
HAYDEN/McDONALD	3	3	0	2	1	1	0	2	2	3	0	3
PIMA/SHEA	2	0	1	0	3	1	2	2	0	1	1	0
1989												
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0	1	0	0	0	0	0	0	0	0	0
SCOTTSDALE/THOMAS	0	1	2	1	2	4	1	1	1	0	1	1
MILLER/INDIAN SCHOOL	2	0	0	0	1	0	1	2	0	2	2	0
HAYDEN/McDOWELL	1	1	2	0	2	1	2	3	2	1	6	5
68TH STREET/INDIAN SCH	0	0	2	0	0	1	1	2	0	1	0	1
HAYDEN/CHAPARRAL	1	0	1	0	0	2	1	0	0	1	0	1
SCOTTSDALE/CAMELBACK	1	3	1	0	2	1	2	0	1	1	2	1
HAYDEN/McDONALD	1	6	1	0	2	1	2	1	2	1	0	0
PIMA/SHEA	2	0	1	1	0	2	0	1	0	0	1	0
1990												
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
61ST PLACE/THOMAS	0	0										
SCOTTSDALE/THOMAS	2	4										
MILLER/INDIAN SCHOOL	2	2										
HAYDEN/McDOWELL	1	1										
68TH STREET/INDIAN SCH	0	2										
HAYDEN/CHAPARRAL	0	1										
SCOTTSDALE/CAMELBACK	2	3										
HAYDEN/McDONALD	2	1										
PIMA/SHEA	0	2										

TABLE B-3

SCOTTSDALE ACCIDENT ANALYSIS

CONTROL INTERSECTION ACCIDENT SUMMARY

	1985											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60TH STREET/THOMAS						0	2	0	0	0	0	1
64TH STREET/CAMELBACK						0	0	0	0	0	1	0
64TH STREET/CACTUS						0	0	0	1	0	1	2
68TH STREET/OAK						0	0	0	0	0	0	0
68TH STREET/OSBORN						0	0	0	0	0	1	0
70TH STREET/MCDOWELL						1	1	2	0	1	1	1
70TH STREET/OSBORN						0	1	0	0	0	0	0
70TH PLACE/CAMELBACK						1	1	0	0	0	0	1
71ST STREET/CAMELBACK						0	0	2	0	0	0	0
71ST PLACE/SHEA						2	1	1	2	3	0	4
SCOTTSDALE/ROOSEVELT						1	1	0	1	3	2	1
SCOTTSDALE/OAK						0	0	0	1	1	1	2
SCOTTSDALE/EARLL						0	1	0	0	1	2	0
SCOTTSDALE/FIFTH AVENUE						0	0	0	0	1	1	1
SCOTTSDALE/FASHION SQUARE						0	0	0	0	0	0	1
SCOTTSDALE/JACKRABBIT						1	0	0	0	0	0	0
SCOTTSDALE/MERCER						1	1	1	0	1	1	0
SCOTTSDALE/CHOLLA						0	0	0	1	1	0	0
SCOTTSDALE/SWEETWATER						1	0	0	0	1	0	0
SCOTTSDALE/PINNACLE PEAK						0	2	0	0	0	1	0
CIVIC CENTER/OSBORN						2	0	0	0	0	1	0
74TH STREET/MCDOWELL						1	1	2	1	3	0	3
75TH STREET/INDIAN SCHOOL						1	0	0	1	0	0	0
MILLER/MCKELLIPS						1	0	0	0	1	2	0
MILLER/CHAPARRAL						0	0	0	0	0	0	1
MILLER/MCDONALD						0	0	1	1	0	1	1
MILLER/SHEA						0	0	0	0	0	0	0
77TH STREET/MCDOWELL						0	0	0	0	0	0	1
HAYDEN/OAK						2	1	1	1	1	1	0
HAYDEN/JACKRABBIT						1	0	0	1	0	0	0
HAYDEN/INDIAN BEND						1	0	0	0	0	2	2
82ND STREET/INDIAN SCHOOL						1	0	0	0	0	0	0
GRANITE REEF ROAD/THOMAS						2	0	1	2	1	0	1
GRANITE REEF/CAMELBACK						0	1	0	0	0	0	0
GRANITE REEF/CHAPARRAL						1	0	1	1	0	0	0
GRANITE REEF/MCDONALD						0	0	0	0	0	1	0
PIMA/MOUNTIAN VIEW						0	0	2	0	0	0	0

TABLE B-3

CONTINUED

	1986											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60TH STREET/THOMAS	0	0	1	0	0	2	0	0	0	0	0	0
64TH STREET/CAMELBACK	0	0	1	1	1	0	2	0	2	0	0	0
64TH STREET/CACTUS	0	0	0	0	1	0	1	0	1	0	0	0
66TH STREET/OAK	0	0	0	0	0	0	0	0	1	1	0	0
68TH STREET/OSBORN	0	2	1	2	1	0	0	3	2	0	0	0
70TH STREET/MCDOWELL	0	1	0	0	2	1	0	0	0	0	0	2
70TH STREET/OSBORN	0	0	1	1	0	0	0	1	0	0	0	0
70TH PLACE/CAMELBACK	0	0	0	0	0	0	1	1	0	0	1	0
71ST STREET/CAMELBACK	2	1	0	0	0	1	0	1	1	0	2	3
71ST PLACE/SHEA	3	1	2	3	3	1	3	3	0	1	1	1
SCOTTSDALE/ROOSEVELT	1	1	2	2	1	0	1	1	1	4	5	1
SCOTTSDALE/OAK	1	1	0	1	1	0	1	1	0	1	2	0
SCOTTSDALE/EARLL	0	0	1	0	1	2	0	0	1	1	0	1
SCOTTSDALE/FIFTH AVENUE	0	1	0	0	0	0	0	0	0	0	0	1
SCOTTSDALE/FASHION SQUARE	0	0	0	0	0	0	0	0	0	1	0	1
SCOTTSDALE/JACKRABBIT	1	0	1	0	0	0	0	0	1	0	1	0
SCOTTSDALE/MERCER	0	2	0	0	0	0	0	1	1	0	3	1
SCOTTSDALE/CHOLLA	0	0	0	1	0	0	1	0	0	0	0	0
SCOTTSDALE/SWEETWATER	0	1	1	0	0	0	1	1	2	0	1	0
SCOTTSDALE/PINNACLE PEAK	0	1	0	1	1	1	0	1	0	0	0	0
CIVIC CENTER/OSBORN	0	1	0	0	0	0	0	0	0	0	0	1
74TH STREET/MCDOWELL	1	3	2	3	2	2	2	1	2	1	1	0
75TH STREET/INDIAN SCHOOL	0	1	1	2	0	0	3	0	0	1	1	1
MILLER/MCKELLIPS	0	0	0	0	1	0	0	0	1	0	1	0
MILLER/CHAPARRAL	1	1	2	0	0	1	0	0	0	1	1	0
MILLER/MCDONALD	0	0	0	0	1	0	0	0	0	1	2	0
MILLER/SHEA	0	0	0	0	0	1	1	0	0	0	0	1
77TH STREET/MCDOWELL	0	0	1	0	0	0	0	2	0	1	1	0
HAYDEN/OAK	0	2	0	1	0	3	0	0	2	1	2	1
HAYDEN/JACKRABBIT	0	0	0	2	1	1	1	0	1	0	0	0
HAYDEN/INDIAN BEND	2	0	2	1	1	1	1	0	2	0	5	4
82ND STREET/INDIAN SCHOOL	0	1	0	0	1	2	0	0	0	0	0	0
GRANITE REEF ROAD/THOMAS	2	0	2	0	3	0	0	0	0	3	1	0
GRANITE REEF/CAMELBACK	2	0	0	0	0	0	0	0	0	0	0	0
GRANITE REEF/CHAPARRAL	0	0	3	0	0	0	0	0	0	1	0	0
GRANITE REEF/MCDONALD	1	0	0	0	0	0	0	0	0	2	2	0
PIMA/MOUNTIAN VIEW	0	0	0	0	0	0	0	1	1	3	1	0

TABLE B-3

CONTINUED

	1987											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60TH STREET/THOMAS	2	0	1	1	0	1	0	0	0	1	0	1
64TH STREET/CAMELBACK	0	0	1	0	1	0	0	2	0	0	0	2
64TH STREET/CACTUS	1	0	1	0	0	0	0	1	0	1	0	2
68TH STREET/OAK	1	0	1	0	0	0	0	0	0	0	0	0
68TH STREET/OSBORN	1	1	2	0	0	2	0	0	1	0	0	2
70TH STREET/MCDOWELL	1	1	0	0	1	0	1	0	0	1	0	1
70TH STREET/OSBORN	1	0	1	1	0	0	0	0	0	1	0	0
70TH PLACE/CAMELBACK	0	0	0	1	0	1	0	2	0	0	0	0
71ST STREET/CAMELBACK	1	1	1	0	0	0	0	3	1	0	1	0
71ST PLACE/SHEA	0	1	0	0	2	3	2	0	0	1	1	1
SCOTTSDALE/ROOSEVELT	0	4	5	1	1	2	2	1	1	2	0	0
SCOTTSDALE/OAK	1	2	0	4	1	3	1	0	0	0	2	2
SCOTTSDALE/EARLL	1	0	0	0	0	0	0	0	0	1	0	0
SCOTTSDALE/FIFTH AVENUE	0	0	0	0	0	0	1	0	0	2	1	0
SCOTTSDALE/FASHION SQUARE	0	1	0	0	0	0	0	0	1	1	4	3
SCOTTSDALE/JACKRABBIT	1	0	1	0	1	0	1	0	0	1	0	0
SCOTTSDALE/MERCER	0	0	2	0	0	2	0	1	1	0	1	0
SCOTTSDALE/CHOLLA	0	0	1	0	0	1	0	0	0	1	0	0
SCOTTSDALE/SWEETWATER	0	0	1	0	0	0	0	0	1	1	0	0
SCOTTSDALE/PINNACLE PEAK	1	0	1	0	1	0	0	0	0	0	0	0
CIVIC CENTER/OSBORN	1	0	1	0	1	0	1	2	1	0	0	0
74TH STREET/MCDOWELL	1	6	3	2	1	0	4	4	0	2	3	3
75TH STREET/INDIAN SCHOOL	1	0	0	2	0	2	0	0	1	0	0	1
MILLER/MCKELLIPS	0	2	1	0	3	0	1	0	1	1	0	1
MILLER/CHAPARRAL	2	1	0	0	1	0	0	0	1	0	1	0
MILLER/MCDONALD	1	0	0	0	1	0	0	0	1	0	0	0
MILLER/SHEA	0	1	0	0	2	0	0	0	0	0	1	1
77TH STREET/MCDOWELL	0	0	3	1	2	1	2	0	0	0	1	0
HAYDEN/OAK	0	2	2	1	0	0	2	0	1	0	2	0
HAYDEN/JACKRABBIT	1	0	0	2	1	0	0	0	0	0	1	1
HAYDEN/INDIAN BEND	1	0	0	2	0	1	0	1	1	2	1	1
82ND STREET/INDIAN SCHOOL	1	0	1	0	0	0	0	1	0	0	0	1
GRANITE REEF ROAD/THOMAS	1	1	2	1	4	3	2	0	1	2	1	0
GRANITE REEF/CAMELBACK	1	0	0	0	0	0	0	0	0	0	0	0
GRANITE REEF/CHAPARRAL	1	1	1	0	0	1	0	0	2	1	0	0
GRANITE REEF/MCDONALD	2	2	1	1	0	0	0	0	1	0	0	2
PIMA/MOUNTIAN VIEW	0	0	0	0	0	1	0	1	3	0	1	1

TABLE B-3

CONTINUED

	1988											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60TH STREET/THOMAS	1	1	0	0	0	0	0	0	0	0	1	1
64TH STREET/CAMELBACK	0	1	2	0	1	2	0	0	1	2	0	0
64TH STREET/CACTUS	0	1	0	0	2	1	0	0	1	1	1	0
68TH STREET/OAK	0	0	0	1	0	0	0	1	2	0	0	0
68TH STREET/OSBORN	0	0	0	0	0	2	2	1	0	0	0	2
70TH STREET/MCDOWELL	0	1	0	1	1	0	0	0	1	0	0	0
70TH STREET/OSBORN	0	0	0	0	1	0	0	0	0	0	0	0
70TH PLACE/CAMELBACK	1	0	0	1	0	0	0	0	0	0	0	0
71ST STREET/CAMELBACK	1	1	0	0	1	1	1	0	1	0	0	0
71ST PLACE/SHEA	1	1	1	0	1	4	3	0	2	0	2	1
SCOTTSDALE/ROOSEVELT	7	2	2	1	2	4	1	2	0	0	0	0
SCOTTSDALE/OAK	0	1	0	3	0	0	1	1	0	1	0	0
SCOTTSDALE/EARLL	0	0	0	1	0	0	0	1	0	0	2	0
SCOTTSDALE/FIFTH AVENUE	0	1	1	0	0	0	0	0	0	0	0	0
SCOTTSDALE/FASHION SQUARE	1	1	2	0	0	0	0	1	1	2	0	0
SCOTTSDALE/JACKRABBIT	1	0	0	0	1	0	1	0	1	0	0	1
SCOTTSDALE/MERCER	0	1	0	1	1	2	0	0	1	1	0	2
SCOTTSDALE/CHOLLA	0	0	2	0	0	0	0	0	0	0	0	0
SCOTTSDALE/SWEETWATER	0	0	1	3	0	1	0	0	0	0	0	0
SCOTTSDALE/PINNACLE PEAK	0	0	1	0	0	0	0	0	0	0	0	0
CIVIC CENTER/OSBORN	0	0	1	1	1	0	0	1	0	0	0	0
74TH STREET/MCDOWELL	0	2	1	3	1	2	0	0	0	0	0	0
75TH STREET/INDIAN SCHOOL	0	4	1	1	0	0	1	0	1	2	0	1
MILLER/MCKELLIPS	2	0	1	0	0	2	0	0	0	1	0	0
MILLER/CHAPARRAL	1	1	0	1	1	0	0	0	1	1	0	1
MILLER/MCDONALD	1	0	1	0	2	1	0	0	1	0	1	1
MILLER/SHEA	0	1	1	0	3	0	0	1	0	1	3	0
77TH STREET/MCDOWELL	0	1	0	0	1	0	0	1	0	0	0	3
HAYDEN/OAK	0	1	1	3	0	1	1	1	1	0	0	1
HAYDEN/JACKRABBIT	0	0	0	0	0	0	0	0	0	0	0	1
HAYDEN/INDIAN BEND	1	2	0	0	0	3	0	0	2	0	0	0
82ND STREET/INDIAN SCHOOL	0	0	0	0	0	0	0	0	0	1	0	0
GRANITE REEF ROAD/THOMAS	2	1	1	2	1	0	1	3	0	1	0	3
GRANITE REEF/CAMELBACK	0	0	0	0	0	0	0	0	0	0	0	0
GRANITE REEF/CHAPARRAL	1	0	0	0	1	0	0	0	0	0	3	0
GRANITE REEF/MCDONALD	0	0	2	0	0	1	0	0	1	0	0	1
PIMA/MOUNTIAN VIEW	1	0	1	1	0	1	1	1	0	1	0	0

TABLE B-3

CONTINUED

	1989											
INTERSECTION	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60TH STREET/THOMAS	1	0	0	0	0	0	0	1	0	0	0	0
64TH STREET/CAMELBACK	0	1	0	0	2	1	0	1	0	1	0	1
64TH STREET/CACTUS	0	0	0	0	0	0	0	0	0	1	0	1
68TH STREET/OAK	0	0	1	0	0	0	0	0	0	0	0	0
68TH STREET/OSBORN	0	1	1	0	1	1	0	0	0	1	1	2
70TH STREET/MCDOWELL	0	0	0	0	0	0	0	1	1	0	0	0
70TH STREET/OSBORN	0	0	0	0	0	0	0	0	0	0	2	0
70TH PLACE/CAMELBACK	0	2	0	0	0	1	0	0	0	0	0	0
71ST STREET/CAMELBACK	1	1	0	0	1	0	5	0	0	0	0	2
71ST PLACE/SHEA	0	0	0	1	1	1	0	2	3	2	0	0
SCOTTSDALE/ROOSEVELT	1	0	0	0	2	2	13	2	1	1	1	1
SCOTTSDALE/OAK	1	1	0	0	0	5	1	0	0	0	0	0
SCOTTSDALE/EARLL	0	1	0	0	0	0	0	1	0	0	0	0
SCOTTSDALE/FIFTH AVENUE	0	1	0	0	0	1	0	0	1	2	0	0
SCOTTSDALE/FASHION SQUARE	1	0	2	0	0	1	0	0	0	0	1	0
SCOTTSDALE/JACKRABBIT	0	0	0	0	1	0	0	0	0	1	0	0
SCOTTSDALE/MERCER	0	0	0	0	0	1	0	1	0	0	0	0
SCOTTSDALE/CHOLLA	0	0	0	0	1	0	0	0	0	0	0	0
SCOTTSDALE/SWEETWATER	0	1	0	1	1	0	0	0	1	1	0	0
SCOTTSDALE/PINNACLE PEAK	0	1	0	0	0	0	0	0	2	0	0	0
CIVIC CENTER/OSBORN	0	0	1	0	0	1	2	1	0	0	1	0
74TH STREET/MCDOWELL	1	2	2	0	1	0	0	2	1	2	0	2
75TH STREET/INDIAN SCHOOL	0	0	2	0	0	3	1	1	0	0	0	0
MILLER/MCKELLIPS	0	1	0	0	0	1	1	0	0	0	0	0
MILLER/CHAPARRAL	1	0	0	0	0	0	0	1	0	0	1	0
MILLER/MCDONALD	1	1	0	0	0	1	0	0	0	0	1	0
MILLER/SHEA	2	0	0	1	0	0	2	0	1	1	0	0
77TH STREET/MCDOWELL	0	0	1	0	0	0	0	2	0	0	0	1
HAYDEN/OAK	0	1	0	0	0	0	1	1	0	0	0	0
HAYDEN/JACKRABBIT	0	2	1	1	0	0	0	1	1	0	0	1
HAYDEN/INDIAN BEND	0	0	3	0	1	3	1	0	1	3	5	0
82ND STREET/INDIAN SCHOOL	1	0	0	1	1	1	0	1	0	0	0	0
GRANITE REEF ROAD/THOMAS	1	1	2	1	3	3	3	1	0	0	0	1
GRANITE REEF/CAMELBACK	0	0	0	0	1	0	0	0	1	0	0	0
GRANITE REEF/CHAPARRAL	1	1	0	0	0	0	0	0	0	0	1	0
GRANITE REEF/MCDONALD	0	0	1	0	2	1	0	2	0	1	0	0
PIMA/MOUNTIAN VIEW	0	0	1	0	2	0	0	0	2	0	0	1

TABLE B-3

CONTINUED

INTERSECTION	1990 Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60TH STREET/THOMAS	1	0										
64TH STREET/CAMELBACK	1	0										
64TH STREET/CACTUS	0	0										
68TH STREET/OAK	0	0										
68TH STREET/OSBORN	1	0										
70TH STREET/MCDOWELL	0	0										
70TH STREET/OSBORN	0	1										
70TH PLACE/CAMELBACK	0	0										
71ST STREET/CAMELBACK	0	1										
71ST PLACE/SHEA	1	2										
SCOTTSDALE/ROOSEVELT	2	1										
SCOTTSDALE/OAK	2	3										
SCOTTSDALE/EARLL	0	0										
SCOTTSDALE/FIFTH AVENUE	1	1										
SCOTTSDALE/FASHION SQUARE	0	2										
SCOTTSDALE/JACKRABBIT	0	0										
SCOTTSDALE/MERCER	1	0										
SCOTTSDALE/CHOLLA	0	0										
SCOTTSDALE/SWEETWATER	0	0										
SCOTTSDALE/PINNACLE PEAK	2	0										
CIVIC CENTER/OSBORN	0	0										
74TH STREET/MCDOWELL	1	0										
75TH STREET/INDIAN SCHOOL	0	1										
MILLER/MCKELLIPS	1	1										
MILLER/CHAPARRAL	0	0										
MILLER/MCDONALD	0	0										
MILLER/SHEA	0	0										
77TH STREET/MCDOWELL	0	1										
HAYDEN/OAK	0	1										
HAYDEN/JACKRABBIT	0	0										
HAYDEN/INDIAN BEND	0	0										
82ND STREET/INDIAN SCHOOL	0	1										
GRANITE REEF ROAD/THOMAS	0	0										
GRANITE REEF/CAMELBACK	0	0										
GRANITE REEF/CHAPARRAL	1	0										
GRANITE REEF/MCDONALD	0	1										
PIMA/MOUNTIAN VIEW	0	2										

APPENDIX C

PIMA COUNTY/TUCSON ACCIDENT ANALYSIS

TABLE C-1
DURATION OF STUDY PERIODS

INTERSECTION	NUMBER OF DAYS IN ANALYSIS PERIODS BEFORE	AFTER
AJO WAY / PALO VERDE RD.	730	878
ALVERNON WAY / IRVINGTON RD.	311	878
ALVERNON WAY / VALENCIA RD.	730	884
CAMPBELL AVE. / RIVER RD.	730	863
CRAYCROFT AVE. / RIVER RD.	730	861
CRAYCROFT AVE. / SUNRISE DR.	730	857
DODGE BLVD. / RIVER RD.	417	862
DOS HOMBRES / TANQUE VERDE RD	730	856
FIRST AVE. / INA RD.	730	857
FIRST AVE. / ORANGE GROVE RD.	730	877
FIRST AVE. / RIVER RD.	231	877
INA RD. / LA CANADA DR.	730	848
INA RD. / LA CHOLLA BLVD.	730	847
INA RD. / OLDFATHER RD.	730	826
INA RD. / THORNYDALE RD.	730	827
KOLB RD. / VALENCIA RD.	325	823
LA CHOLLA BLVD. / ORANGE GROVE RD.	331	871
MISSION RD. / VALENCIA RD.	730	883
ORANGE GROVE RD. / SKYLINE DR.	730	863
RIVER RD. / SWAN RD.	562	711
SUNRISE DR. / SWAN RD.	913	793

TABLE C-2
WILCOXEN SIGNED - RANKS TEST
PIMA COUNTY ACCIDENT LEFT TURN RATE ANALYSIS

INTERSECTION	ACCIDENT RATE DIFFERENCE	RANK W/ SIGN
AJO WAY / PALO VERDE RD.		
NORTHBOUND	- 0.094	- 7
SOUTHBOUND	+ 0.180	+ 15
ALVERNON WAY / IRVINGTON RD.		
SOUTHBOUND	-	
EASTBOUND	- 1.274	- 34
ALVERNON WAY / VALENCIA RD.		
EASTBOUND	- 0.598	- 30
WESTBOUND	+ 0.537	+ 28
CAMPBELL AVE. / RIVER RD.		
NORTHBOUND	- 0.569	- 29
WESTBOUND	+ 0.155	+ 11
CRAYCROFT AVE. / RIVER RD.		
SOUTHBOUND	- 0.060	- 3
WESTBOUND	-	
CRAYCROFT AVE. / SUNRISE DR.		
NORTHBOUND (P)	-	
DODGE BLVD. / RIVER RD.		
WESTBOUND	+ 0.319	+ 22
DOS HOMBRES / TANQUE VERDE RD		
EASTBOUND	+ 0.109	+ 9
WESTBOUND	+ 0.098	+ 8
FIRST AVE. / INA RD.		
NORTHBOUND *	+ 0.288	+ 21
FIRST AVE. / ORANGE GROVE RD.		
NORTHBOUND	- 0.156	- 12
SOUTHBOUND	+ 0.262	+ 20
FIRST AVE. / RIVER RD.		
NORTHBOUND	+ 0.092	+ 6
SOUTHBOUND	+ 0.693	+ 31
INA RD. / LA CANADA DR.		
EASTBOUND	- 0.458	- 24
WESTBOUND	- 0.053	- 2
INA RD. / LA CHOLLA BLVD.		

TABLE C-2
CONT.

INTERSECTION	ACCIDENT RATE DIFFERENCE	RANK W/ SIGN
INA RD. / LA CHOLLA BLVD.		
EASTBOUND	+ 0.081	+ 4
WESTBOUND	+ 0.033	+ 1
INA RD. / OLDFATHER RD.		
EASTBOUND	- 0.245	- 19
INA RD. / THORNYDALE RD.		
NORTHBOUND	- 0.891	- 33
SOUTHBOUND	- 0.483	- 26
EASTBOUND	+ 0.148	+ 10
WESTBOUND	+ 0.521	+ 27
KOLB RD. / VALENCIA RD.		
NORTHBOUND (P)	-	
SOUTHBOUND (P)	-	
EASTBOUND (P)	+ 0.730	+ 32
WESTBOUND (P)	-	
LA CHOLLA BLVD. / ORANGE GROVE RD.		
NORTHBOUND (P)	-	
SOUTHBOUND (P)	- 0.197	- 17
EASTBOUND (P)	- 0.184	- 16
WESTBOUND (P)	+ 0.384	+ 23
MISSION RD. / VALENCIA RD.		
EASTBOUND	- 0.216	- 18
WESTBOUND	+ 0.090	+ 5
ORANGE GROVE RD. / SKYLINE DR.		
NORTHBOUND (P)	- 0.176	- 14
RIVER RD. / SWAN RD.		
NORTHBOUND	- 0.170	- 13
WESTBOUND	-	
SUNRISE DR. / SWAN RD.		
SOUTHBOUND *	+ 0.466	+ 25
	T(+) = + 298	T(-) = -297

P - Protected only left-turns

* - Protected/permitted left-turns in the before period
and protected left turns in the after period.

TABLE C-3
WILCOXEN SIGNED - RANKS TEST
PIMA COUNTY ANALYSIS OF NUMBER OF LEFT-TURN ACCIDENTS

INTERSECTION	ACCIDENT DIFFERENCE	RANK
AJO WAY / PALO VERDE RD.		
NORTHBOUND	0.41	+8
SOUTHBOUND	-1.00	-19
ALVERNON WAY / IRVINGTON RD.		
EASTBOUND	2.90	+33
ALVERNON WAY / VALENCIA RD.		
EASTBOUND	2.39	+29
WESTBOUND	-2.00	-26.5
CAMPBELL AVE. / RIVER RD.		
NORTHBOUND	2.89	+32
WESTBOUND	0.04	+1
CRAYCROFT AVE. / RIVER RD.		
SOUTHBOUND	0.19	+4
DODGE BLVD. / RIVER RD.		
WESTBOUND	-0.46	-11
DOS HOMBRES / TANQUE VERDE RD		
EASTBOUND	-0.57	-15.5
WESTBOUND	-0.57	-15.5
FIRST AVE. / INA RD.		
NORTHBOUND *	-0.50	-12.5
FIRST AVE. / ORANGE GROVE RD.		
NORTHBOUND	-1.08	-20
SOUTHBOUND	-0.50	-12.5
FIRST AVE. / RIVER RD.		
NORTHBOUND	-0.34	-6
SOUTHBOUND	-2.68	-30
INA RD. / LA CANADA DR.		
EASTBOUND	2.81	+31
WESTBOUND	0.36	+7
INA RD. / LA CHOLLA BLVD.		
EASTBOUND	-0.14	-2
WESTBOUND	0.16	+3
INA RD. / OLDFATHER RD.		
EASTBOUND	1.33	+23

TABLE C-3
CONT.

INTERSECTION	ACCIDENT DIFFERENCE	RANK
INA RD. / THORNYDALE RD.		
NORTHBOUND	2.96	+34
SOUTHBOUND	1.70	+25
EASTBOUND	-0.56	-14
WESTBOUND	-2.00	-26.5
KOLB RD. / VALENCIA RD.		
EASTBOUND (P)	-2.25	-28
LA CHOLLA BLVD. / ORANGE GROVE RD.		
SOUTHBOUND (P)	0.42	+9.5
EASTBOUND (P)	0.42	+9.5
WESTBOUND (P)	-1.10	-21
MISSION RD. / VALENCIA RD.		
EASTBOUND	0.83	+18
WESTBOUND	-0.26	-5
ORANGE GROVE RD. SKYLINE DR.		
NORTHBOUND (P)	1.51	+24
RIVER RD. / SWAN RD.		
NORTHBOUND	1.13	+22
SUNRISE DR. / SWAN RD.		
SOUTHBOUND *	0.80	+17
	T(+) = 330	T(-) = -265

TABLE C-4
WILCOXEN TEST BASED ON LEFT TURN ACCIDENT RATES
(CITY OF TUCSON ARTERIAL/ARTERIAL INTERSECTIONS)

INTERSECTION	ACCIDENT RATE DIFFERENCE	RANK W/ SIGN
AJO WAY / MISSION RD.	- 0.032	- 5
AJO WAY / INTERSTATE 19	0	0
AJO WAY / 12TH AVE.	+ 0.249	+ 41
ALVERNON WAY / BROADWAY BLVD.	+ 0.151	+ 29
ALVERNON WAY / 22ND ST.	- 0.696	+ 49
BROADWAY BLVD. / CAMPBELL AVE.	- 0.544	- 47
BROADWAY BLVD. / COUNTRY CLUB RD.	+ 0.052	- 11.5
BROADWAY BLVD. / CRAYCROFT RD.	+ 0.055	+ 14
BROADWAY BLVD. / KOLB RD.	+ 0.038	+ 7.5
BROADWAY BLVD. / SWAN RD.	+ 0.044	+ 9
BROADWAY BLVD. / WILMOT RD.	+ 0.231	+ 39
CAMPBELL AVE. / FORT LOWELL RD.	+ 0.176	+ 34
CAMPBELL AVE. / GRANT RD.	- 0.053	- 13
CAMPBELL AVE. / SPEEDWAY BLVD.	+ 0.038	+ 7.5
CONGRESS ST. / GRANADA AVE.	- 0.036	- 6
CONGRESS ST. / INTERSTATE 10	+ 0.217	+ 37
COUNTRY CLUB RD. / GRANT RD.	- 0.141	- 27
COUNTRY CLUB RD. / SPEEDWAY BLVD.	- 0.007	- 2
COUNTRY CLUB RD. / VALENCIA RD.	- 0.153	- 30
CRAYCROFT RD. / GOLF LINKS RD.	- 0.350	- 44
CRAYCROFT RD. / 22ND ST.	- 0.052	- 11.5
FORT LOWELL RD. / ORACLE RD.	- 0.405	- 45
GOLF LINKS RD. / KOLB RD.	- 0.145	- 28
GOLF LINKS RD. / WILMOT RD.	- 0.072	- 17.5
GRANT RD. / ORACLE RD.	- 0.046	- 10
GRANT RD. / STONE AVE.	+ 0.021	+ 4
GRANT RD. / SWAN RD.	+ 0.166	+ 33
GRANT (KOLB) RD. / TANQUE VERDE RD.	- 0.077	- 19.5
GRANT RD. / FIRST AVE.	+ 0.161	+ 32
GRANT RD. / INTERSTATE 10	- 0.417	- 46
KOLB RD. / SPEEDWAY BLVD.	+ 0.105	+ 22.5
KOLB RD. / 22ND ST.	+ 0.177	+ 35.5
MAIN AVE. / SPEEDWAY BLVD.	+ 0.105	+ 22.5
MIRACLE MILE / ORACLE RD.	- 0.336	- 43

TABLE C-4
CONT.

INTERSECTION	ACCIDENT RATE DIFFERENCE	RANK W/ SIGN
NOGALES HIGHWAY / VALENCIA RD.	+ 0.238	+ 40
ORACLE RD. / PRINCE RD.	- 0.077	- 19.5
ORACLE RD. / RIVER RD.	- 0.127	- 25
ORACLE RD. / WETMORE RD.	- 0.225	- 38
SPEEDWAY BLVD. / STONE AVE.	+ 0.159	+ 31
SPEEDWAY BLVD. / SWAN RD.	- 0.017	- 3
SPEEDWAY BLVD. / WILMOT RD.	- 0.108	- 24
SPEEDWAY BLVD. / INTERSTATE 10	+ 0.177	+ 35.5
ST. MARY'S RD. / INTERSTATE 10	+ 0.265	+ 42
SWAN RD. / 22ND ST.	+ 0.596	+ 48
VALENCIA RD. / 12TH AVE.	- 0.002	- 1
WETMORE RD. / FIRST AVE.	+ 0.072	+ 17.5
WILMOT RD. / 5TH ST.	- 0.084	- 21
WILMOT RD. / 22ND ST.	- 0.069	- 16
INTERSTATE 10 / 22ND ST.	+ 0.134	+ 26
5TH AVE. / INTERSTATE 10	+ 0.063	+ 15
	T (+) = + 669.5	T (-) = - 553.5

TABLE C-5

WILCOXEN TEST BASED ON LEFT-TURN ACCIDENT RATES

(CITY OF TUCSON ARTERIAL/COLLECTOR INTERSECTIONS)

INTERSECTION	ACCIDENT DIFFERENCE	RANK W/ SIGN
ALVERNON WAY / 29TH ST.	+ 0.151	+ 10
AUTO MALL DR. / ORACLE RD.	- 0.056	- 4
BROADWAY BLVD. / COLUMBUS BLVD.	+ 0.101	+ 5
BROADWAY BLVD. / RANDOLPH WAY	+ 0.037	+ 3
BROADWAY BLVD. / ROSEMONT BLVD.	- 0.002	- 1
CHERRY AVE. / 22ND ST.	- 0.136	- 6
COLUMBUS BLVD. / 22ND ST.	+ 0.256	+ 12
GRANT RD. / WILMOT RD.	- 0.137	- 7.5
LIMBERLOST RD. / FIRST AVE.	- 0.014	- 2
ORACLE RD. / ROGER RD.	- 0.141	+ 9
SANTA CLARA AVE. / VALENCIA RD.	- 0.178	+ 7.5
TUCSON BLVD. / VALENCIA RD.	0.474	- 11
	T (+) = + 46.5	T (-) = - 31.5

TABLE C-6

WILCOXEN TEST BASED ON NUMBER OF LEFT-TURN ACCIDENTS

(CITY OF TUCSON ARTERIAL/COLLECTOR INTERSECTIONS)

INTERSECTION	DIFFERENCE IN ACCIDENTS PER YEAR	RANK W/ SIGN
ALVERNON WAY / 29TH ST.	-2.67	- 9
AUTO MALL DR. / ORACLE RD.	1.17	+ 4
BROADWAY BLVD. / COLUMBUS BLVD.	-2.83	-10
BROADWAY BLVD. / RANDOLPH WAY	-0.17	- 1.5
BROADWAY BLVD. / ROSEMONT BLVD.	-0.17	- 1.5
CHERRY AVE. / 22ND ST.	1.83	+ 8
COLUMBUS BLVD. / 22ND ST.	-4.67	-12
GRANT RD. / WILMOT RD.	3.17	11
LIMBERLOST RD. / FIRST AVE.	0.33	+ 3
ORACLE RD. / ROGER RD.	-1.50	- 6
SANTA CLARA AVE. / VALENCIA RD.	-1.33	- 5
TUCSON BLVD. / VALENCIA RD.	1.67	+ 7

T (+) = 33 T (-) = -45

TABLE C-7

WILCOXEN TEST BASED ON NUMBER OF LEFT-TURN ACCIDENTS

(CITY OF TUCSON ARTERIAL/ARTERIAL INTERSECTIONS)

INTERSECTION	DIFFERENCE IN LEFT-TURN ACCIDENTS	RANK W/ SIGN
AJO WAY / MISSION RD.	0.50	7.5
AJO WAY / 12TH AVE.	-3.00	-34
ALVERNON WAY / BROADWAY BLVD.	-3.17	-35.5
ALVERNON WAY / 22ND ST.	-16.50	-49
BROADWAY BLVD. / CAMPBELL AVE.	8.33	47
BROADWAY BLVD. / COUNTRY CLUB RD.	1.33	16.5
BROADWAY BLVD. / CRAYCROFT RD.	-1.33	-16.5
BROADWAY BLVD. / KOLB RD.	0.17	2.5
BROADWAY BLVD. / SWAN RD.	-0.83	-13
BROADWAY BLVD. / WILMOT RD.	-4.50	-40.5
CAMPBELL AVE. / FORT LOWELL RD.	-3.67	-38.5
CAMPBELL AVE. / GRANT RD.	3.67	38.5
CAMPBELL AVE. / SPEEDWAY BLVD.	0.17	2.5
CONGRESS ST. / GRANADA AVE.	0.50	7.5
CONGRESS ST. / INTERSTATE 10	-2.17	-27
COUNTRY CLUB RD. / GRANT RD.	2.83	32.5
COUNTRY CLUB RD. / SPEEDWAY BLVD.	-0.33	-5
COUNTRY CLUB RD. / VALENCIA RD.	1.67	23
CRAYCROFT RD. / GOLF LINKS RD.	7.50	46
CRAYCROFT RD. / 22ND ST.	0.67	11
FORT LOWELL RD. / ORACLE RD.	7.00	45
GOLF LINKS RD. / KOLB RD.	2.67	31
GOLF LINKS RD. / WILMOT RD.	1.00	14.5
GRANT RD. / ORACLE RD.	1.00	14.5
GRANT RD. / STONE AVE.	-0.50	-7.5
GRANT RD. / SWAN RD.	-2.50	-30
GRANT (KOLB) RD./TANQUE VERDE RD.	1.50	19.5
GRANT RD. / FIRST AVE.	-3.50	-37
GRANT RD. / INTERSTATE 10	5.83	43

TABLE C-7
CONT.

INTERSECTION	DIFFERENCE IN LEFT-TURN ACCIDENTS	RANK W/ SIGN
KOLB RD. / SPEEDWAY BLVD.	-2.33	-28.5
KOLB RD. / 22ND ST.	-4.83	-42
MAIN AVE. / SPEEDWAY BLVD.	-1.83	-25
MIRACLE MILE / ORACLE RD.	6.33	44
NOGALES HIGHWAY / VALENCIA RD.	-2.83	-32.5
ORACLE RD. / PRINCE RD.	1.67	23
ORACLE RD. / RIVER RD.	2.33	28.5
ORACLE RD. / WETMORE RD.	4.50	40.5
SPEEDWAY BLVD. / STONE AVE.	-1.67	-23
SPEEDWAY BLVD. / SWAN RD.	-0.17	-2.5
SPEEDWAY BLVD. / WILMOT RD.	2.00	26
SPEEDWAY BLVD. / INTERSTATE 10	-1.50	-19.5
ST. MARY'S RD. / INTERSTATE 10	-3.17	-35.5
SWAN RD. / 22ND ST.	-11.83	-48
VALENCIA RD. / 12TH AVE.	-0.67	-11
WETMORE RD. / FIRST AVE.	0.17	2.5
WILMOT RD. / 5TH ST.	0.50	7.5
WILMOT RD. / 22ND ST.	1.50	19.5
INTERSTATE 10 / 22ND ST.	-1.5	-19.5
5TH AVE. / INTERSTATE 10	-0.67	-11

T (+) = 593.50 T (-) = -631.50

TABLE C-8
TOTAL EQUIVALENT ACCIDENTS PER YEAR
PIMA COUNTY INTERSECTIONS

INTERSECTION		NUMBER OF ACCIDENTS BEFORE	AFTER	DIFFERENCE	% CHANGE
AJO WAY / PALO VERDE RD.					
	NB/SB *	15.50	7.47	-8.03	-51.81
ALVERNON WAY / IRVINGTON RD.					
	SB/EB	3.53	9.54	6.01	170.25
ALVERNON WAY / VALENCIA RD.					
	EB/WB	9.00	7.02	-1.98	-22.00
CAMPBELL AVE. / RIVER RD.					
	NB/WB	11.00	13.56	2.56	23.27
CRAYCROFT AVE. / RIVER RD.					
	SB/WB	4.50	7.63	3.13	69.56
CRAYCROFT AVE. / SUNRISE DR.					
	NB (P)	2.50	1.28	-1.22	-48.80
DODGE BLVD. / RIVER RD.					
	WB	1.75	0.85	-0.90	-51.43
DOS HOMBRES / TANQUE VERDE RD					
	EB/WB	12.00	8.09	-3.91	-32.58
FIRST AVE. / INA RD.					
	NB **	3.00	2.55	-0.45	-15.00
FIRST AVE. / ORANGE GROVE RD.					
	NB/SB	8.00	7.08	-0.92	-11.50
FIRST AVE. / RIVER RD.					
	NB/SB	23.81	10.42	-13.39	-56.24
INA RD. / LA CANADA DR.					
	EB/WB	10.00	15.52	5.52	55.20
INA RD. / LA CHOLLA BLVD.					
	EB/WB	12.50	16.38	3.88	31.04
INA RD. / OLD FATHER RD.					
	EB	1.50	4.42	2.92	194.67
INA RD. / THORNYDALE RD.					
	NB/SB/EB/WB	17.50	23.79	6.29	35.94
KOLB RD. / VALENCIA RD.					
	NB/SB/EB/WB (P)	13.48	11.11	-2.37	-17.58
LA CHOLLA BLVD. / ORANGE GROVE RD.					
	NB/SB/EB/WB (P)	6.59	10.04	3.45	52.35
MISSION RD. / VALENCIA RD.					
	EB/WB	13.50	6.61	-6.89	-51.04

TABLE C-8
CONT.

INTERSECTION		NUMBER OF ACCIDENTS BEFORE	AFTER	DIFFERENCE	% CHANGE
ORANGE GROVE RD. / SKYLINE DR.					
	NB (P)	10.50	12.71	2.21	21.05
RIVER RD. / SWAN RD.					
	WB	3.90	6.67	2.77	71.03
SUNRISE DR. / SWAN RD.					
	SB **	2.80	1.38	-1.42	-50.71
TOTAL		186.86	184.12	-2.74	-1.47

NOTE: * indicates the approaches included in the analysis
(P) indicates approaches with protected only left-turns
** protected/permiteed left-turns in the before period and protected left turns in the after period

TABLE C-9
WILLCOXEN TEST BASED ON TOTAL ACCIDENTS
PIMA COUNTY INTERSECTIONS

INTERSECTION		DIFFERENCE IN TOTAL INTERSECTION ACCIDENTS	RANK W/ SIGN
AJO WAY / PALO VERDE RD.			
	NB/SB*	-8.03	-20
ALVERNON WAY / IRVINGTON RD.			
	SB/EB	6.01	17
ALVERNON WAY / VALENCIA RD.			
	EB/WB	-1.98	-6
CAMPBELL AVE. / RIVER RD.			
	NB/WB	2.56	9
CRAYCROFT AVE. / RIVER RD.			
	SB/WB	3.13	12
CRAYCROFT AVE. / SUNRISE DR.			
	NB(P)	-1.22	-4
DODGE BLVD. / RIVER RD.			
	WB	-0.90	-2
DOS HOMBRES / TANQUE VERDE RD			
	EB/WB	-3.91	-15
FIRST AVE. / INA RD.			
	NB**	-0.45	-1
FIRST AVE. / ORANGE GROVE RD.			
	NB/SB	-0.92	-3
FIRST AVE. / RIVER RD.			
	NB/SB	13.39	21
INA RD. / LA CANADA DR.			
	EB/WB	5.52	16
INA RD. / LA CHOLLA BLVD.			
	EB/WB	3.88	14
INA RD. / OLD FATHER RD.			
	EB	2.92	11
INA RD. / THORNYDALE RD.			
	NB/SB/EB/WB	6.29	18
KOLB RD. / VALENCIA RD.			
	NB/SB/EB/WB(P)	-2.37	-8
LA CHOLLA BLVD. / ORANGE GROVE RD.			
	NB/SB/EB/WB(P)	3.45	13
MISSION RD. / VALENCIA RD.			
	EB/WB	-6.89	-19
ORANGE GROVE RD. / SKYLINE DR.			
	NB(P)	2.21	7

TABLE C-9

CONT.

INTERSECTION		DIFFERENCE IN TOTAL INTERSECTION ACCIDENTS	RANK W/ SIGN
RIVER RD. / SWAN RD.			
	WB	2.77	10
SUNRISE DR. / SWAN RD.			
	SB**	-1.42	5
T (-) = 88 T (+) = 151			

NOTE: * indicates the approaches included in the analysis

P indicates approaches with protected only left-turns

** protected/permitted left-turns in the before period and protected left-turns in the after period.

TABLE C-10
TOTAL INTERSECTION ACCIDENTS
(CITY OF TUCSON ARTERIAL/ARTERIAL INTERSECTIONS)

	INTERSECTION ACCIDENTS PER YEAR			
INTERSECTION	BEFORE	AFTER	DIFFERENCE	% CHANGE
AJO WAY / MISSION RD.	12.33	14.5	2.17	17.60
AJO WAY / INTERSTATE 19	16.33	15.0	-1.33	-8.14
AJO WAY / 12TH AVE.	25.00	19.0	-6.00	-24.00
ALVERNON WAY / BROADWAY BLVD.	24.00	24.5	0.50	2.08
ALVERNON WAY / 22ND ST.	65.33	38.5	-26.83	-41.07
BROADWAY BLVD. / CAMPBELL AVE.	20.33	27.0	6.67	32.81
BROADWAY BLVD. / COUNTRY CLUB RD.	21.00	14.5	-6.50	-30.95
BROADWAY BLVD. / CRAYCROFT RD.	15.33	8.0	-7.33	-47.81
BROADWAY BLVD. / KOLB RD.	19.67	11.5	-8.17	-41.54
BROADWAY BLVD. / SWAN RD.	21.33	26.0	4.67	21.89
BROADWAY BLVD. / WILMOT RD.	22.33	9.5	-12.83	-57.46
CAMPBELL AVE. / FORT LOWELL RD.	20.00	22.5	2.50	12.50
CAMPBELL AVE. / GRANT RD.	24.67	29.5	4.83	19.58
CAMPBELL AVE. / SPEEDWAY BLVD.	34.00	38.0	4.00	11.76
CONGRESS ST. / GRANADA AVE.	5.67	7.0	1.33	23.46
CONGRESS ST. / INTERSTATE 10	28.00	18.0	-10.00	-35.71
COUNTRY CLUB RD. / GRANT RD.	17.67	22.5	4.83	27.33
COUNTRY CLUB RD. / SPEEDWAY BLVD.	6.67	9.0	2.33	34.93
COUNTRY CLUB RD. / VALENCIA RD.	2.33	4.0	1.67	71.67
CRAYCROFT RD. / GOLF LINKS RD.	10.33	20.5	10.17	98.45
CRAYCROFT RD. / 22ND ST.	42.00	41.5	-0.50	-1.19
FORT LOWELL RD. / ORACLE RD.	3.33	20.5	17.17	515.62
GOLF LINKS RD. / KOLB RD.	17.67	22.0	4.33	24.50
GOLF LINKS RD. / WILMOT RD.	19.00	18.5	-0.50	-2.63
GRANT RD. / ORACLE RD.	21.67	22.0	0.33	1.52
GRANT RD. / STONE AVE.	23.00	23.0	0.00	0.00
GRANT RD. / SWAN RD.	29.33	28.0	-1.33	-4.53
GRANT (KOLB) RD./TANQUE VERDE RD.	19.33	22.0	2.67	13.81

TABLE C-10
CONT.

INTERSECTION	INTERSECTION ACCIDENTS PER YEAR BEFORE	AFTER	DIFFERENCE	% CHANGE
GRANT RD. / FIRST AVE.	22.67	23.5	0.83	3.66
GRANT RD. / INTERSTATE 10	21.67	25.0	3.33	15.37
KOLB RD. / SPEEDWAY BLVD.	20.00	20.5	0.50	2.50
KOLB RD. / 22ND ST.	32.67	34.0	1.33	4.07
MAIN AVE. / SPEEDWAY BLVD.	13.33	6.5	-6.83	-51.24
MIRACLE MILE / ORACLE RD.	5.33	17.5	12.17	228.33
NOGALES HIGHWAY / VALENCIA RD.	17.67	16.0	-1.67	-9.45
ORACLE RD. / PRINCE RD.	41.33	35.0	-6.33	-15.32
ORACLE RD. / RIVER RD.	11.33	17.0	5.67	50.04
ORACLE RD. / WETMORE RD.	11.33	21.5	10.17	89.76
SPEEDWAY BLVD. / STONE AVE.	24.67	18.5	-6.17	-25.01
SPEEDWAY BLVD. / SWAN RD.	6.67	7.0	0.33	4.95
SPEEDWAY BLVD. / WILMOT RD.	15.00	14.5	-0.50	-3.33
SPEEDWAY BLVD. / INTERSTATE 10	20.67	9.5	-11.17	-54.04
ST. MARY'S RD. / INTERSTATE 10	22.67	5.5	-17.17	-75.74
SWAN RD. / 22ND ST.	24.33	20.0	-4.33	-17.80
VALENCIA RD. / 12TH AVE.	14.00	16.0	2.00	14.29
WETMORE RD. / FIRST AVE.	3.33	5.0	1.67	50.15
WILMOT RD. / 5TH ST.	11.67	9.5	-2.17	-18.59
WILMOT RD. / 22ND ST.	32.67	30.5	-2.17	-6.64
INTERSTATE 10 / 22ND ST.	33.00	12.5	-20.50	-62.12
5TH AVE. / INTERSTATE 10	21.33	12.0	-9.33	-43.74
TOTAL	1014.99	953.5	-61.49	-6.06

TABLE C-11
WILLCOXEN TEST BASED ON TOTAL INTERSECTION ACCIDENTS
(CITY OF TUCSON ARTERIAL/ARTERIAL INTERSECTIONS)

INTERSECTIONS	DIFFERENCE IN TOTAL INTERSECTION ACCIDENTS	RANK W/ SIGN
AJO WAY / MISSION RD.	2.17	19
AJO WAY / INTERSTATE 19	-1.33	-11.5
AJO WAY / 12TH AVE.	-6.00	-32
ALVERNON WAY / BROADWAY BLVD.	0.5	5.5
ALVERNON WAY / 22ND ST.	-26.83	-49
BROADWAY BLVD. / CAMPBELL AVE.	6.67	36
BROADWAY BLVD. / COUNTRY CLUB RD.	-6.50	-35
BROADWAY BLVD. / CRAYCROFT RD.	-7.33	-38
BROADWAY BLVD. / KOLB RD.	-8.17	-39
BROADWAY BLVD. / SWAN RD.	4.67	28
BROADWAY BLVD. / WILMOT RD.	-12.83	-46
CAMPBELL AVE. / FORT LOWELL RD.	2.50	22
CAMPBELL AVE. / GRANT RD.	4.83	29.5
CAMPBELL AVE. / SPEEDWAY BLVD.	4.00	25
CONGRESS ST. / GRANADA AVE.	1.33	11.5
CONGRESS ST. / INTERSTATE 10	-10.00	-41
COUNTRY CLUB RD. / GRANT RD.	4.83	29.5
COUNTRY CLUB RD. / SPEEDWAY BLVD.	2.33	21
COUNTRY CLUB RD. / VALENCIA RD	1.67	15
CRAYCROFT RD. / GOLF LINKS RD	10.17	42.5
CRAYCROFT RD. / 22ND ST.	-0.50	-5.5
FORT LOWELL RD. / ORACLE RD.	17.17	47.5
GOLF LINKS RD. / KOLB RD.	4.33	26.5
GOLF LINKS RD. / WILMOT RD.	-0.50	-5.5
GRANT RD. / ORACLE RD.	0.33	1.5
GRANT RD. / STONE AVE.	0.00	-
GRANT RD. / SWAN RD.	-1.33	-11.5
GRANT (KOLB) RD./TANQUE VERDE RD.	2.67	23
GRANT RD. / FIRST AVE.	0.83	9
GRANT RD. / INTERSTATE 10	3.33	24

TABLE C-11
CONT.

INTERSECTIONS	DIFFERENCE IN TOTAL INTERSECTION ACCIDENTS	RANK W/ SIGN
KOLB RD. / SPEEDWAY BLVD.	0.50	5.5
KOLB RD. / 22ND ST.	1.33	11.5
MAIN AVE. / SPEEDWAY BLVD.	-6.83	-37
MIRACLE MILE / ORACLE RD.	12.17	4.5
NOGALES HIGHWAY / VALENCIA RD.	-1.67	-15
ORACLE RD. / PRINCE RD.	-6.33	-34
ORACLE RD. / RIVER RD.	5.67	31
ORACLE RD. / WETMORE RD.	10.17	42.5
SPEEDWAY BLVD. / STONE AVE.	-6.17	-33
SPEEDWAY BLVD. / SWAN RD.	0.33	1.5
SPEEDWAY BLVD. / WILMOT RD.	-0.50	-5.5
SPEEDWAY BLVD. / INTERSTATE 10	-11.17	-44
ST. MARY'S RD. / INTERSTATE 10	-17.17	-47.5
SWAN RD. / 22ND ST.	-4.33	-26.5
VALENCIA RD. / 12TH AVE.	2.00	17
WETMORE RD. / FIRST AVE.	1.67	15
WILMOT RD. / 5TH ST.	-2.17	-19
WILMOT RD. / 22ND ST.	-2.17	-19
INTERSTATE 10 / 22ND ST.	-20.50	-48
5TH AVE. / INTERSTATE 10	-9.33	-40

T (+) = 544.50 T (-) = -682.50

TABLE C-12
TOTAL INTERSECTION ACCIDENTS PER YEAR
(CITY OF TUCSON ARTERIAL/COLLECTOR INTERSECTIONS)

INTERSECTION	INTERSECTION ACCIDENTS PER YEAR BEFORE	AFTER	DIFFERENCE	% CHANGE
ALVERNON WAY / 29TH ST.	30.67	18.50	-12.17	-39.68
AUTO MALL DR. / ORACLE RD.	9.33	3.00	-6.33	-67.85
BROADWAY BLVD. / COLUMBUS BLVD.	12.67	10.00	-2.67	-21.07
BROADWAY BLVD. / RANDOLPH WAY	8.33	8.50	0.17	2.04
BROADWAY BLVD. / ROSEMONT BLVD.	8.67	11.50	2.83	32.64
CHERRY AVE. / 22ND ST.	14.00	17.00	3.00	21.43
COLUMBUS BLVD. / 22ND ST.	20.33	8.50	-11.83	-58.19
GRANT RD. / WILMOT RD.	10.00	14.50	4.50	45.00
LIMBERLOST RD. / FIRST AVE.	7.33	6.50	-0.83	-11.32
ORACLE RD. / ROGER RD.	8.67	7.00	-1.67	-19.26
SANTA CLARA AVE. / VALENCIA RD.	6.67	3.50	-3.17	-47.53
TUCSON BLVD. / VALENCIA RD.	10.00	12.00	2.00	20.00
TOTAL	146.67	120.50	-26.17	-17.84

TABLE C-13

WILCOXEN TEST BASED ON TOTAL INTERSECTION ACCIDENTS

(CITY OF TUCSON ARTERIAL/COLLECTOR INTERSECTIONS)

INTERSECTIONS	DIFFERENCE IN ACCIDENTS PER YEAR	RANK W/ SIGN
ALVERNON WAY / 29TH ST.	-12.17	-12
AUTO MALL DR. / ORACLE RD.	-6.33	-10
BROADWAY BLVD. / COLUMBUS BLVD.	-2.67	-5
BROADWAY BLVD. / RANDOLPH WAY	0.17	1
BROADWAY BLVD. / ROSEMONT BLVD.	2.83	6
CHERRY AVE. / 22ND ST.	3.00	7
COLUMBUS BLVD. / 22ND ST.	-11.83	-11
GRANT RD. / WILMOT RD.	4.50	9
LIMBERLOST RD. / FIRST AVE.	-0.83	-2
ORACLE RD. / ROGER RD.	-1.67	-3
SANTA CLARA AVE. / VALENCIA RD.	-3.17	-8
TUCSON BLVD. / VALENCIA RD.	2.00	4

T (+) = 27 T (-) = 51

APPENDIX D

PHOENIX AREA INTERSECTION ANALYSIS

D-1 SUMMARY WORKSHEETS FOR INTERSECTION DELAY

D-2 CITY OF PHOENIX 3RD CAR ACTUATION STUDY

APPENDIX D-1

SUMMARY WORKSHEETS FOR INTERSECTION DELAY STUDY

INTERSECTION: Glendale & 51st Ave Leading 3rd Car
SURVEY DATE: 01/10/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	17	622	49	101	71	255	96	510	639	150	326	606	233	1488	1721
5:00-5:15 pm	12	507	53	98	90	250	135	579	519	151	340	714	290	1434	1724
5:15-5:30 pm	16	608	99	69	41	171	56	331	624	168	212	387	212	1179	1391
5:30-5:45 pm	9	639	60	77	33	132	23	246	648	137	165	269	125	1094	1219
TOTAL	54	2376	261	345	235	808	310	1666	2430	606	1043	1976	860	5195	6055

INTERSECTION: Glendale & 51st Ave Leading 3rd Car
SURVEY DATE: 01/10/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	13	304	30	162	41	177	44	218	317	192	218	262	128	861	989
5:00-5:15 pm	15	286	39	155	43	200	50	261	301	194	243	311	147	902	1049
5:15-5:30 pm	13	314	42	132	23	155	42	240	327	174	178	282	120	841	961
5:30-5:45 pm	13	303	41	166	26	162	27	220	316	207	188	247	107	851	958
TOTAL	54	1207	152	615	133	694	163	939	1261	767	827	1102	502	3455	3957

INTERSECTION: Glendale & 51st Ave Leading 3rd Car
SURVEY DATE: 01/10/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	20	31	25	9	26	22	33	35	30	12	22	35	27	26	26
5:00-5:15 pm	12	27	20	9	31	19	41	33	26	12	21	34	30	24	25
5:15-5:30 pm	18	29	35	8	27	17	20	21	29	14	18	21	27	21	22
5:30-5:45 pm	10	32	22	7	19	12	13	17	31	10	13	16	18	19	19
TOTAL	15	30	26	8	27	17	29	27	29	12	19	27	25.70	22.55	22.95

INTERSECTION: Glendale & 51st Ave Lagging
SURVEY DATE: 04/2/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	26	359	158	503	155	384	161	537	385	661	539	698	500	1783	2283
4:45-5:00 pm	23	431	133	396	146	315	252	649	454	529	461	901	554	1791	2345
5:00-5:15 pm	6	626	278	437	198	525	198	860	632	715	723	1058	680	2448	3128
5:15-5:30 pm	33	283	161	373	146	304	149	1035	316	534	450	1184	489	1995	2484
TOTAL	88	1699	730	1709	645	1528	760	3081	1787	2439	2173	3841	2223	8017	10240

INTERSECTION: Glendale & 51st Ave Lagging
SURVEY DATE: 04/02/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	14	270	44	166	42	193	45	233	284	210	235	278	145	862	1007
5:00-5:15 pm	10	281	58	198	36	187	52	191	291	256	223	243	156	857	1013
5:15-5:30 pm	9	362	53	153	35	180	42	230	371	206	215	272	139	925	1064
5:30-5:45 pm	11	293	43	172	37	143	46	250	304	215	180	296	137	858	995
TOTAL	44	1206	198	689	150	703	185	904	1250	887	853	1089	577	3502	4079

INTERSECTION: Glendale & 51st Ave Lagging
SURVEY DATE: 04/02/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	28	20	54	45	55	30	54	35	20	47	34	38	52	31	34
5:00-5:15 pm	35	23	34	30	61	25	73	51	23	31	31	56	53	31	35
5:15-5:30 pm	10	26	79	43	85	44	71	56	26	52	50	58	73	40	44
5:30-5:45 pm	45	14	56	33	59	32	49	62	16	37	38	60	54	35	37
TOTAL	30	21	55	37	65	33	62	51	21	41	38	53	57.79	34.34	37.66

INTERSECTION: Northern & 51st Ave Leading 3rd Car
SURVEY DATE: 01/09/90 Delay

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
5:00-5:15 pm	45	1504	77	164	106	718	42	573	1549	241	824	615	270	2959	3229
5:15-5:30 pm	43	1172	73	184	82	753	45	1056	1215	257	835	1101	243	3165	3408
5:30-5:45 pm	82	2005	66	180	85	379	39	684	2087	246	464	723	272	3248	3520
5:45-6:00 pm	43	1237	70	190	92	270	24	499	1280	260	362	523	229	2196	2425
TOTAL	213	5918	286	718	365	2120	150	2812	6131	1004	2485	2962	1014	11568	12582

INTERSECTION: Northern & 51st Ave Leading 3rd Car
SURVEY DATE: 01/09/90 Volume

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
5:00-5:15 pm	49	320	38	161	51	272	38	264	369	199	323	302	176	1017	1193
5:15-5:30 pm	46	294	35	151	44	244	24	290	340	186	288	314	149	979	1128
5:30-5:45 pm	60	280	39	157	38	243	29	283	340	196	281	312	166	963	1129
5:45-6:00 pm	51	281	45	143	46	236	14	274	332	188	282	288	156	934	1090
TOTAL	206	1175	157	612	179	995	105	1111	1381	769	1174	1216	647	3893	4540

INTERSECTION: Northern & 51st Ave Leading 3rd Car
SURVEY DATE: 01/09/90 Delay Per Vehicle (sec/veh)

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
5:00-5:15 pm	14	71	30	15	31	40	17	33	63	18	38	31	23	44	41
5:15-5:30 pm	14	60	31	18	28	46	28	55	54	21	43	53	24	48	45
5:30-5:45 pm	21	107	25	17	34	23	20	36	92	19	25	35	25	51	47
5:45-6:00 pm	13	66	23	20	30	17	26	27	58	21	19	27	22	35	33
TOTAL	16	76	27	18	31	32	21	38	67	20	32	37	23.51	44.57	41.57

INTERSECTION: Northern & 51st Ave. Lagging
SURVEY DATE: 04/09/90 Delay

Time Period	Northbound Lt	Th/Rt	Southbound Lt	Th/Rt	Eastbound Lt	Th/Rt	Westbound Lt	Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Th/Rt	Total
5:00-5:15 pm	205	1258	135	414	340	911	134	458	1463	549	1251	592	814	3041	3855
5:15-5:30 pm	89	1020	157	251	241	1010	71	575	1109	408	1251	646	558	2856	3414
5:30-5:45 pm	156	926	148	296	242	627	28	1092	1082	444	869	1120	574	2941	3515
5:45-6:00 pm	60	392	104	311	176	522	61	298	452	415	698	359	401	1523	1924
TOTAL	510	3596	544	1272	999	3070	294	2423	4106	1816	4069	2717	2347	10361	12708

INTERSECTION: Northern & 51st Ave Lagging
SURVEY DATE: 04/09/90 Volume

Time Period	Northbound Lt	Th/Rt	Southbound Lt	Th/Rt	Eastbound Lt	Th/Rt	Westbound Lt	Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Th/Rt	Total
5:00-5:15 pm	61	340	34	173	46	280	29	312	401	207	326	341	170	1105	1275
5:15-5:30 pm	49	342	37	158	41	281	22	286	391	195	322	308	149	1067	1216
5:30-5:45 pm	61	333	32	164	46	214	28	326	394	196	260	354	167	1037	1204
5:45-6:00 pm	39	317	39	149	39	210	23	279	356	188	249	302	140	955	1095
TOTAL	210	1332	142	644	172	985	102	1203	1542	786	1157	1305	626	4164	4790

INTERSECTION: Northern & 51st Ave Lagging
SURVEY DATE: 04/09/90 Delay Per Vehicle (sec/veh)

Time Period	Northbound Lt	Th/Rt	Southbound Lt	Th/Rt	Eastbound Lt	Th/Rt	Westbound Lt	Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Th/Rt	Total
5:00-5:15 pm	50	56	60	36	111	49	69	22	55	40	58	26	72	41	45
5:15-5:30 pm	27	45	64	24	88	54	48	30	43	31	58	31	56	40	42
5:30-5:45 pm	38	42	69	27	79	44	15	50	41	34	50	47	52	43	44
5:45-6:00 pm	23	19	40	31	68	37	40	16	19	33	42	18	43	24	26
TOTAL	36	40	57	30	87	47	43	30	40	35	53	31	56.24	37.32	39.80

INTERSECTION: Olive & 51st Ave. LEADING 3RD CAR
SURVEY DATE: 01/11/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	64	477	102	198	81	207	52	670	541	300	288	722	299	1552	1851
5:15-5:30 pm	78	734	107	180	58	185	40	681	812	287	243	721	283	1780	2063
5:30-5:45 pm	87	531	120	212	82	186	88	492	618	332	268	580	377	1421	1798
5:45-6:00 pm	84	520	102	117	44	154	41	325	604	219	198	366	271	1116	1387
TOTAL	313	2262	431	707	265	732	221	2168	2575	1138	997	2389	1230	5869	7099

INTERSECTION: Olive & 51st Ave. LEADING 3RD CAR
SURVEY DATE: 01/11/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	67	349	35	197	36	170	31	320	416	232	206	351	169	1036	1205
5:15-5:30 pm	58	329	44	189	30	190	39	367	387	233	220	406	171	1075	1246
5:30-5:45 pm	59	323	51	171	29	162	40	345	382	222	191	385	179	1001	1180
5:45-6:00 pm	44	317	45	178	28	176	35	297	361	223	204	332	152	968	1120
TOTAL	228	1318	175	735	123	698	145	1329	1546	910	821	1474	671	4080	4751

INTERSECTION: Olive & 51st Ave. LEADING 3RD CAR
SURVEY DATE: 01/11/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	14	21	44	15	34	18	25	31	20	19	21	31	27	22	23
5:15-5:30 pm	20	33	36	14	29	15	15	28	31	18	17	27	25	25	25
5:30-5:45 pm	22	25.	35	19	42	17	33	21	24	22	21	23	32	21	23
5:45-6:00 pm	29	25	34	10	24	13	18	16	25	15	15	17	27	17	19
TOTAL	21	26	37	14	32	16	23	24	25	19	18	24	27.50	21.58	22.41

INTERSECTION: Olive & 51st Ave. LAGGING
SURVEY DATE: 04/13/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:05-5:20 pm	159	342	269	419	105	359	239	1197	501	688	464	1436	772	2317	3089
5:20-5:35 pm	93	389	181	349	63	293	168	1067	482	530	356	1235	505	2098	2603
5:35-5:50 pm	77	515	118	262	80	297	111	496	592	380	377	607	386	1570	1956
5:50-6:05 pm	67	251	108	256	81	325	59	364	318	364	406	423	315	1196	1511
TOTAL	396	1497	676	1286	329	1274	577	3124	1893	1962	1603	3701	1978	7181	9159

INTERSECTION: Olive & 51st Ave. LAGGING
SURVEY DATE: 04/13/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:05-5:20 pm	60	351	50	179	34	176	51	345	411	229	210	396	195	1051	1246
5:20-5:35 pm	61	370	48	183	25	175	41	363	431	231	200	404	175	1091	1266
5:35-5:50 pm	56	261	40	149	27	147	43	354	317	189	174	397	166	911	1077
5:50-6:05 pm	37	244	33	149	24	163	25	286	281	182	187	311	119	842	961
TOTAL	214	1226	171	660	110	661	160	1348	1440	831	771	1508	655	3895	4550

INTERSECTION: Olive & 51st Ave. LAGGING
SURVEY DATE: 04/13/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	40	15	81	35	46	31	70	52	18	45	33	54	59	33	37
5:15-5:30 pm	23	16	57	29	38	25	61	44	17	34	27	46	43	29	31
5:30-5:45 pm	21	30	44	26	44	30	39	21	28	30	33	23	35	26	27
5:45-6:00 pm	27	15	49	26	51	30	35	19	17	30	33	20	40	21	24
TOTAL	28	18	59	29	45	29	54	35	20	35	31	37	45.30	27.65	30.19

INTERSECTION: Peoria & 51st Ave. Leading 3rd Car
SURVEY DATE: 01/04/90

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	46	228	76	210	61	185	135	648	274	286	246	783	318	1271	1589
4:45-5:00 pm	58	226	118	211	69	369	145	602	284	329	438	747	390	1408	1798
5:00-5:15 pm	122	297	62	240	53	183	264	578	419	302	236	842	501	1298	1799
5:15-5:30 pm	84	350	76	248	54	213	247	645	434	324	267	892	461	1456	1917
TOTAL	310	1101	332	909	237	950	791	2473	1411	1241	1187	3264	1670	5433	7103

INTERSECTION: Peoria & 51st Ave. Leading 3rd Car
SURVEY DATE: 01/04/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	27	218	28	156	20	159	28	271	245	184	179	299	103	804	907
4:45-5:00 pm	32	252	44	205	28	240	45	335	284	249	268	380	149	1032	1181
5:00-5:15 pm	61	214	47	209	27	269	45	390	275	256	296	435	180	1082	1262
5:15-5:30 pm	62	277	44	207	20	252	38	407	339	251	272	445	164	1143	1307
TOTAL	182	961	163	777	95	920	156	1403	1143	940	1015	1559	596	4061	4657

INTERSECTION: Peoria & 51st Ave. Leading 3rd Car
SURVEY DATE: 01/04/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	26	16	41	20	46	17	72	36	17	23	21	39	46	24	26
4:45-5:00 pm	27	13	40	15	37	23	48	27	15	20	25	29	39	20	23
5:00-5:15 pm	30	21	20	17	29	10	88	22	23	18	12	29	42	18	21
5:15-5:30 pm	20	19	26	18	41	13	98	24	19	19	15	30	42	19	22
TOTAL	26	17	31	18	37	15	76	26	19	20	18	31	42.03	20.07	22.88

INTERSECTION: Peoria & 51st Ave. Lagging
SURVEY DATE: 04/26/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	217	289	319	407	259	950	139	956	506	726	1209	1095	934	2602	3536
4:45-5:00 pm	162	343	406	395	129	744	140	754	505	801	873	894	837	2236	3073
5:00-5:15 pm	104	401	150	297	181	858	140	1157	505	447	1039	1297	575	2713	3288
5:15-5:30 pm	148	524	255	378	113	977	140	816	672	633	1090	956	656	2695	3351
TOTAL	631	1557	1130	1477	682	3529	559	3683	2188	2607	4211	4242	3002	10246	13248

INTERSECTION: Peoria & 51st Ave. Lagging
SURVEY DATE: 04/26/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	41	232	35	336	44	258	37	269	273	371	302	306	157	1095	1252
4:45-5:00 pm	57	200	43	352	55	307	32	253	257	395	362	285	187	1112	1299
5:00-5:15 pm	51	178	37	398	52	298	32	246	229	435	350	278	172	1120	1292
5:15-5:30 pm	48	225	44	395	48	305	30	291	273	439	353	321	170	1216	1386
TOTAL	197	835	159	1481	199	1168	131	1059	1032	1640	1367	1190	686	4543	5229

INTERSECTION: Peoria & 51st Ave. Lagging
SURVEY DATE: 04/26/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	79	19	137	18	88	55	56	53	28	29	60	54	89	36	42
4:45-5:00 pm	43	26	142	17	35	36	66	45	29	30	36	47	67	30	35
5:00-5:15 pm	31	34	61	11	52	43	66	71	33	15	45	70	50	36	38
5:15-5:30 pm	46	35	87	14	35	48	70	42	37	22	46	45	58	33	36
TOTAL	48	28	107	15	51	45	64	52	32	24	46	53	65.64	33.83	38.00

INTERSECTION: Dunlap & 35th Ave. Leading 3rd Car
SURVEY DATE: 05/03/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:15-4:30 pm	116	1088	105	415	110	127	73	312	1204	520	237	385	404	1942	2346
4:30-4:45 pm	120	981	119	603	141	154	116	565	1101	722	295	681	496	2303	2799
4:45-5:00 pm	112	1461	83	490	135	221	81	641	1573	573	356	722	411	2813	3224
5:00-5:15 pm	157	2182	70	609	189	252	103	600	2339	679	441	703	519	3643	4162
TOTAL	505	5712	377	2117	575	754	373	2118	6217	2494	1329	2491	1830	10701	12531

INTERSECTION: Dunlap & 35th Ave. Leading 3rd Car
SURVEY DATE: 05/03/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:15-4:30 pm	53	383	38	253	48	230	41	335	436	291	278	376	180	1201	1381
4:30-4:45 pm	54	424	40	258	55	255	50	410	478	298	310	460	199	1347	1546
4:45-5:00 pm	50	370	45	250	51	239	55	393	420	295	290	448	201	1252	1453
5:00-5:15 pm	49	441	23	330	69	257	52	413	490	353	326	465	193	1441	1634
TOTAL	206	1618	146	1091	223	981	198	1551	1824	1237	1204	1749	773	5241	6014

INTERSECTION: Dunlap & 35th Ave. Leading 3rd Car
SURVEY DATE: 05/03/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:15-4:30 pm	33	43	41	25	34	8	27	14	41	27	13	15	34	24	25
4:30-4:45 pm	33	35	45	35	38	9	35	21	35	36	14	22	37	26	27
4:45-5:00 pm	34	59	28	29	40	14	22	24	56	29	18	24	31	34	33
5:00-5:15 pm	48	74	46	28	41	15	30	22	72	29	20	23	40	38	38
TOTAL	37	53	39	29	39	12	28	20	51	30	17	21	35.51	30.63	31.25

INTERSECTION: Dunlap & 35th Ave. Leading 1st Car
SURVEY DATE: 05/16/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:15-4:30 pm	55	944	47	741	55	80	52	448	999	788	135	500	209	2213	2422
4:30-4:45 pm	69	472	57	749	90	201	51	638	541	806	291	689	267	2060	2327
4:45-5:00 pm	86	1561	58	757	121	166	60	717	1647	815	287	777	325	3201	3526
5:00-5:15 pm	113	954	91	764	96	278	107	740	1067	855	374	847	407	2736	3143
TOTAL	323	3931	253	3011	362	725	270	2543	4254	3264	1087	2813	1208	10210	11418

INTERSECTION: Dunlap & 35th Ave. Leading 1st Car
SURVEY DATE: 05/16/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:15-4:30 pm	32	371	26	249	30	235	45	353	403	275	265	398	133	1208	1341
4:30-4:45 pm	35	362	24	263	51	267	40	383	397	287	318	423	150	1275	1425
4:45-5:00 pm	53	416	30	252	66	253	52	380	469	282	319	432	201	1301	1502
5:00-5:15 pm	62	377	32	324	54	248	63	381	439	356	302	444	211	1330	1541
TOTAL	182	1526	112	1088	201	1003	200	1497	1708	1200	1204	1697	695	5114	5809

INTERSECTION: Dunlap & 35th Ave. Leading 1st Car
SURVEY DATE: 05/16/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:15-4:30 pm	26	38	27	45	28	5	17	19	37	43	8	19	24	27	27
4:30-4:45 pm	30	20	36	43	26	11	19	25	20	42	14	24	27	24	24
4:45-5:00 pm	24	56	29	45	28	10	17	28	53	43	13	27	24	37	35
5:00-5:15 pm	27	38	43	35	27	17	25	29	36	36	19	29	29	31	31
TOTAL	27	39	34	42	27	11	20	25	37	41	14	25	26.07	29.95	29.48

INTERSECTION: Northern & 43rd Ave. Leading 3rd Car
SURVEY DATE: 05/1/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	99	413	113	243	186	479	116	987	512	356	665	1103	514	2122	2636
4:45-5:00 pm	60	1192	122	135	117	802	100	1102	1252	257	919	1202	399	3231	3630
5:00-5:15 pm	112	1031	88	243	242	505	96	846	1143	331	747	942	538	2625	3163
5:15-5:45 pm	162	2766	84	184	191	456	155	2181	2928	268	647	2336	592	5587	6179
TOTAL	433	5402	407	805	736	2242	467	5116	5835	1212	2978	5583	2043	13565	15608

INTERSECTION: Northern & 43rd Ave. Leading 3rd Car
SURVEY DATE: 05/1/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	53	422	31	188	60	273	37	353	475	219	333	390	181	1236	1417
4:45-5:00 pm	51	439	41	170	49	244	46	397	490	211	293	443	187	1250	1437
5:00-5:15 pm	58	438	46	193	59	280	29	408	496	239	339	437	192	1319	1511
5:15-5:45 pm	66	537	34	192	57	247	50	414	603	226	304	464	207	1390	1597
TOTAL	228	1836	152	743	225	1044	162	1572	2064	895	1269	1734	767	5195	5962

INTERSECTION: Northern & 43rd Ave. Leading 3rd Car
SURVEY DATE: 05/1/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	28	15	55	19	47	26	47	42	16	24	30	42	43	26	28
4:45-5:00 pm	18	41	45	12	36	49	33	42	38	18	47	41	32	39	38
5:00-5:15 pm	29	35	29	19	62	27	50	31	35	21	33	32	42	30	31
5:15-5:45 pm	37	77	37	14	50	28	47	79	73	18	32	76	43	60	58
TOTAL	28	44	40	16	49	32	43	49	42	20	35	48	39.95	39.17	39.27

INTERSECTION: Northern & 43rd Ave. Leading 1st Car
SURVEY DATE: 05/17/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	81	598	67	284	104	832	71	466	679	351	936	537	323	2180	2503
5:15-5:30 pm	79	802	85	246	99	1211	61	1396	881	331	1310	1457	324	3655	3979
5:30-5:45 pm	82	1127	86	269	122	1045	62	1674	1209	355	1167	1736	352	4115	4467
5:45-6:00 pm	163	1573	89	304	106	735	43	2090	1736	393	841	2133	401	4702	5103
TOTAL	405	4100	327	1103	431	3823	237	5626	4505	1430	4254	5863	1400	14652	16052

INTERSECTION: Northern & 43rd Ave. Leading 1st Car
SURVEY DATE: 05/17/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	51	418	37	189	44	249	44	372	469	226	293	416	176	1228	1404
4:45-5:00 pm	53	433	45	204	44	276	37	433	486	249	320	470	179	1346	1525
5:00-5:15 pm	55	465	39	195	67	256	34	406	520	234	323	440	195	1322	1517
5:15-5:45 pm	62	446	47	204	53	287	33	432	508	251	340	465	195	1369	1564
TOTAL	221	1762	168	792	208	1068	148	1643	1983	960	1276	1791	745	5265	6010

INTERSECTION: Northern & 43rd Ave. Leading 1st Car
SURVEY DATE: 05/17/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	24	21	27	23	35	50	24	19	22	23	48	19	28	27	27
4:45-5:00 pm	22	28	28	18	34	66	25	48	27	20	61	47	27	41	39
5:00-5:15 pm	22	36	33	21	27	61	27	62	35	23	54	59	27	47	44
5:15-5:45 pm	39	53	28	22	30	38	20	73	51	23	37	69	31	52	49
TOTAL	27	35	29	21	31	54	24	51	34	22	50	49	28.19	41.74	40.06

INTERSECTION: Elliot & 51st St. Leading 3rd Car
SURVEY DATE: 05/21/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	57	86	575	116	24	143	100	96	143	691	167	196	756	441	1197
5:15-5:30 pm	54	64	53	28	14	102	85	111	118	81	116	196	206	305	511
5:30-5:45 pm	38	66	70	28	8	111	91	88	104	98	119	179	207	293	500
5:45-6:00 pm	56	31	65	38	19	107	81	66	87	103	126	147	221	242	463
TOTAL	205	247	763	210	65	463	357	361	452	973	528	718	1390	1281	2671

INTERSECTION: Elliot & 51st St. Leading 3rd Car
SURVEY DATE: 05/21/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	38	85	93	107	17	162	83	209	123	200	179	292	231	563	794
5:15-5:30 pm	20	66	51	55	15	161	80	219	86	106	176	299	166	501	667
5:30-5:45 pm	24	66	40	38	12	178	83	216	90	78	190	299	159	498	657
5:45-6:00 pm	23	49	34	44	15	155	81	202	72	78	170	283	153	450	603
TOTAL	105	266	218	244	59	656	327	846	371	462	715	1173	709	2012	2721

INTERSECTION: Elliot & 51st St. Leading 3rd Car
SURVEY DATE: 05/21/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	23	15	93	16	21	13	18	7	17	52	14	10	49	12	23
5:15-5:30 pm	41	15	16	8	14	10	16	8	21	11	10	10	19	9	11
5:30-5:45 pm	24	15.	26	11	10	9	16	6	17	19	9	9	20	9	11
5:45-6:00 pm	37	9	29	13	19	10	15	5	18	20	11	8	22	8	12
TOTAL	29	14	53	13	17	11	16	6	18	32	11	9	29.41	9.55	14.72

INTERSECTION: Elliot & 51st St. Leading 1st Car
SURVEY DATE: 05/14/90 Delay

Time Period	Northbound Lt	Th/Rt	Southbound Lt	Th/Rt	Eastbound Lt	Th/Rt	Westbound Lt	Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Th/Rt	Total
5:00-5:15 pm	60	84	607	119	7	137	94	127	144	726	144	221	768	467	1235
5:15-5:30 pm	34	40	94	53	5	144	64	125	74	147	149	189	197	362	559
5:30-5:45 pm	34	33	86	51	13	169	97	91	67	137	182	188	230	344	574
5:45-6:00 pm	56	50	80	21	6	124	59	84	106	101	130	143	201	279	480
TOTAL	184	207	867	244	31	574	314	427	391	1111	605	741	1396	1452	2848

INTERSECTION: Elliot & 51st St. Leading 1st Car
SURVEY DATE: 05/14/90 Volume

Time Period	Northbound Lt	Th/Rt	Southbound Lt	Th/Rt	Eastbound Lt	Th/Rt	Westbound Lt	Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Th/Rt	Total
5:00-5:15 pm	25	80	89	103	15	159	70	193	105	192	174	263	199	535	734
5:15-5:30 pm	9	61	51	41	8	153	72	232	70	92	161	304	140	487	627
5:30-5:45 pm	19	52	43	46	18	186	79	194	71	89	204	273	159	478	637
5:45-6:00 pm	26	63	44	24	10	160	77	213	89	68	170	290	157	460	617
TOTAL	79	256	227	214	51	658	298	832	335	441	709	1130	655	1960	2615

INTERSECTION: Elliot & 51st St. Leading 1st Car
SURVEY DATE: 05/14/90 Delay Per Vehicle (sec/veh)

Time Period	Northbound Lt	Th/Rt	Southbound Lt	Th/Rt	Eastbound Lt	Th/Rt	Westbound Lt	Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Th/Rt	Total
5:00-5:15 pm	36	16	102	17	7	13	20	10	21	57	12	13	58	13	25
5:15-5:30 pm	57	10	28	19	9	14	13	8	16	24	14	9	21	11	13
5:30-5:45 pm	27	10	30	17	11	14	18	7	14	23	13	10	22	11	14
5:45-6:00 pm	32	12	27	13	9	12	11	6	18	22	11	7	19	9	12
TOTAL	35	12	57	17	9	13	16	8	18	38	13	10	31.97	11.11	16.34

INTERSECTION: Broadway & 48th St. 3RD CAR LEADING
SURVEY DATE: 04/5/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Total	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	166	402	360	751	335	1796	304	403	568	1111	2131	707	1165	3352	4517
4:45-5:00 pm	134	356	367	968	516	1905	345	295	490	1335	2421	640	1362	3524	4886
5:00-5:15 pm	130	423	292	518	599	1200	206	318	553	810	1799	524	1227	2459	3686
5:15-5:30 pm	118	340	440	705	496	1356	652	434	458	1145	1852	1086	1706	2835	4541
TOTAL	548	1521	1459	2942	1946	6257	1507	1450	2069	4401	8203	2957	5460	12170	17630

INTERSECTION: Broadway & 48th St. 3RD CAR LEADING
SURVEY DATE: 04/5/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	60	260	105	334	93	431	63	131	320	439	524	194	321	1156	1477
4:45-5:00 pm	48	204	123	380	96	483	61	112	252	503	579	173	328	1179	1507
5:00-5:15 pm	48	253	106	382	93	442	47	121	301	488	535	168	294	1198	1492
5:15-5:30 pm	51	235	125	360	104	404	69	117	286	485	508	186	349	1116	1465
TOTAL	207	952	459	1456	386	1760	240	481	1159	1915	2146	721	1292	4649	5941

INTERSECTION: Broadway & 48th St. 3RD CAR LEADING
SURVEY DATE: 04/5/90 Delay per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	42	23	51	34	54	63	72	46	27	38	61	55	54	43	46
4:45-5:00 pm	42	26	45	38	81	59	85	40	29	40	63	55	62	45	49
5:00-5:15 pm	41	25	41	20	97	41	66	39	28	25	50	47	63	31	37
5:15-5:30 pm	35	22	53	29	72	50	142	56	24	35	55	88	73	38	46
TOTAL	40	24	48	30	76	53	94	45	27	34	57	62	63.39	39.27	44.51

INTERSECTION: Broadway & 48th St. 1ST CAR LEADING
SURVEY DATE: 05/09/90 Delay

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
4:30-4:45 pm	157	402	231	448	252	1131	158	263	559	679	1383	421	798	2244	3042
4:45-5:00 pm	145	350	357	791	151	834	310	149	495	1148	985	459	963	2124	3087
5:00-5:15 pm	129	499	341	588	449	1416	329	249	628	929	1865	578	1248	2752	4000
5:15-5:30 pm	180	440	373	817	461	915	463	250	620	1190	1376	713	1477	2422	3899
TOTAL	611	1691	1302	2644	1313	4296	1260	911	2302	3946	5609	2171	4486	9542	14028

INTERSECTION: Broadway & 48th St. 1ST CAR LEADING
SURVEY DATE: 05/09/90 Volume

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
4:30-4:45 pm	48	210	99	304	92	467	64	139	258	403	559	203	303	1120	1423
4:45-5:00 pm	46	187	135	401	78	370	46	120	233	536	448	166	305	1078	1383
5:00-5:15 pm	50	257	132	354	108	483	69	116	307	486	591	185	359	1210	1569
5:15-5:30 pm	59	226	146	392	84	413	75	111	285	538	497	186	364	1142	1506
TOTAL	203	880	512	1451	362	1733	254	486	1083	1963	2095	740	1331	4550	5881

INTERSECTION: Broadway & 48th St. 1ST CAR LEADING
SURVEY DATE: 05/09/90 Delay per Vehicle (sec/veh)

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
4:30-4:45 pm	49	29	35	22	41	36	37	28	33	25	37	31	40	30	32
4:45-5:00 pm	47	28	40	30	29	34	101	19	32	32	33	41	47	30	33
5:00-5:15 pm	39	29	39	25	62	44	72	32	31	29	47	47	52	34	38
5:15-5:30 pm	46	29	38	31	82	33	93	34	33	33	42	58	61	32	39
TOTAL	45	29	38	27	54	37	74	28	32	30	40	44	50.56	31.46	35.78

INTERSECTION: Broadway & 48th St. Lagging
SURVEY DATE: 06/12/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	151	530	354	774	410	1081	218	307	681	1128	1491	525	1133	2692	3825
4:45-5:00 pm	142	322	453	675	122	888	240	248	464	1128	1010	488	957	2133	3090
5:00-5:15 pm	266	621	854	1680	358	1727	289	268	887	2534	2085	557	1767	4296	6063
5:15-5:30 pm	144	417	837	1419	370	1329	321	256	561	2256	1699	577	1672	3421	5093
TOTAL	703	1890	2498	4548	1260	5025	1068	1079	2593	7046	6285	2147	5529	12542	18071

INTERSECTION: Broadway & 48th St. Lagging
SURVEY DATE: 06/12/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	48	225	131	313	89	340	59	139	273	444	429	198	327	1017	1344
4:45-5:00 pm	37	171	140	370	53	353	48	88	208	510	406	136	278	982	1260
5:00-5:15 pm	74	276	155	419	101	371	78	91	350	574	472	169	408	1157	1565
5:15-5:30 pm	42	236	147	397	82	397	95	93	278	544	479	188	366	1123	1489
TOTAL	201	908	573	1499	325	1461	280	411	1109	2072	1786	691	1379	4279	5658

INTERSECTION: Broadway & 48th St. Lagging
SURVEY DATE: 06/12/90 Delay Per Vehicle (veh/sec)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:30-4:45 pm	47	35	41	37	69	48	55	33	37	38	52	40	52	40	43
4:45-5:00 pm	58	28	49	27	35	38	75	42	33	33	37	54	52	33	37
5:00-5:15 pm	54	34	83	60	53	70	56	44	38	66	66	49	65	56	58
5:15-5:30 pm	51	27	85	54	68	50	51	41	30	62	53	46	69	46	51
TOTAL	52	31	65	46	58	52	57	39	35	51	53	47	60.14	43.97	47.91

INTERSECTION: Southern & 48th St. Leading 3rd Car
SURVEY DATE: 05/15/90 Delay

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
5:00 - 5:15 pm	75	227	135	450	52	591	252	74	302	585	643	326	514	1342	1856
5:15 - 5:30 pm	135	172	368	551	87	879	575	73	307	919	966	648	1165	1675	2840
5:30 - 5:45 pm	55	215	207	453	57	646	416	63	270	660	703	479	735	1377	2112
5:45 - 6:00 pm	46	114	96	330	59	266	74	51	160	426	325	125	275	761	1036
TOTAL	311	728	806	1784	255	2382	1317	261	1039	2590	2637	1578	2689	5155	7844

INTERSECTION: Southern & 48th St. Leading 3rd Car
SURVEY DATE: 05/15/90 Volume

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
5:00 - 5:15 pm	16	150	74	382	19	309	59	196	166	456	328	255	168	1037	1205
5:15 - 5:30 pm	33	116	93	361	24	325	69	151	149	454	349	220	219	953	1172
5:30 - 5:45 pm	20	111	79	351	22	279	68	167	131	430	301	235	189	908	1097
5:45 - 6:00 pm	17	93	70	267	27	208	44	121	110	337	235	165	158	689	847
TOTAL	86	470	316	1361	92	1121	240	635	556	1677	1213	875	734	3587	4321

INTERSECTION: Southern & 48th St. 3RD CAR LEADING
SURVEY DATE: 05/15/90 Delay Per Vehicle (sec/veh)

Time Period	Northbound Lt	Northbound Th/Rt	Southbound Lt	Southbound Th/Rt	Eastbound Lt	Eastbound Th/Rt	Westbound Lt	Westbound Th/Rt	NB Total	SB Total	EB Total	WB Total	Intersection Totals Lt	Intersection Totals Th/Rt	Intersection Totals Total
5:00 - 5:15 pm	70	23	27	18	41	29	64	6	27	19	29	19	46	19	23
5:15 - 5:30 pm	61	22	59	23	54	41	125	7	31	30	42	44	80	26	36
5:30 - 5:45 pm	41	29	39	19	39	35	92	6	31	23	35	31	58	23	29
5:45 - 6:00 pm	41	18	21	19	33	19	25	6	22	19	21	11	26	17	18
TOTAL	54	23	38	20	42	32	82	6	28	23	33	27	54.95	21.56	27.23

INTERSECTION: 48th St. & Southern Leading 1st Car
SURVEY DATE: 05/08/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Total		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	82	205	211	970	51	913	320	115	287	1181	964	435	664	2203	2867
5:15-5:30 pm	134	157	281	869	48	885	171	49	291	1150	933	220	634	1960	2594
5:30-5:45 pm	82	127	195	921	43	630	78	48	209	1116	673	126	398	1726	2124
5:45-6:00 pm	35	134	102	415	33	381	78	51	169	517	414	129	248	981	1229
TOTAL	333	623	789	3175	175	2809	647	263	956	3964	2984	910	1944	6870	8814

INTERSECTION: 48th St. & Southern Leading 1st Car
SURVEY DATE: 05/08/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Total		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	18	140	96	377	24	290	67	186	158	473	314	253	205	993	1198
5:15-5:30 pm	25	177	75	357	22	316	62	145	202	432	338	207	184	995	1179
5:30-5:45 pm	19	124	70	393	43	286	72	158	143	463	329	230	204	961	1165
5:45-6:00 pm	23	110	80	259	18	244	65	163	133	339	262	228	186	776	962
TOTAL	85	551	321	1386	107	1136	266	652	636	1707	1243	918	779	3725	4504

INTERSECTION: Southern & 48th St. Leading 1st Car
SURVEY DATE: 05/08/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Total		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00 - 5:15	68	22	33	39	32	47	72	9	27	37	46	26	49	33	36
5:15 - 5:30	80	13	56	37	33	42	41	5	22	40	41	16	52	30	33
5:30 - 5:45	65	15	42	35	15	33	16	5	22	36	31	8	29	27	27
5:45 - 6:00	23	18	19	24	28	23	18	5	19	23	24	8	20	19	19
TOTAL	59	17	37	34	25	37	36	6	23	35	36	15	37.43	27.66	29.35

INTERSECTION: Southern & 48th St. Lagging
SURVEY DATE: 06/13/90 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00-5:15 pm	84	197	275	1805	21	1063	556	508	281	2080	1084	1064	936	3573	4509
5:15-5:30 pm	75	243	465	2679	54	1062	221	415	318	3144	1116	636	815	4399	5214
5:30-5:45 pm	113	229	260	1542	54	489	178	294	342	1802	543	472	605	2554	3159
5:45-6:00 pm	103	131	127	213	26	265	178	160	234	340	291	338	434	769	1203
TOTAL	375	800	1127	6239	155	2879	1133	1377	1175	7366	3034	2510	2790	11295	14085

INTERSECTION: Southern & 48th St. Lagging
SURVEY DATE: 06/13/90 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00 - 5:15	25	116	82	358	19	306	77	171	141	440	325	248	203	951	1154
5:15 - 5:30	17	109	106	393	13	286	73	153	126	499	299	226	209	941	1150
5:30 - 5:45	27	121	76	363	32	251	56	132	148	439	283	188	191	867	1058
5:45 - 6:00	19	94	72	275	21	211	47	99	113	347	232	146	159	679	838
TOTAL	88	440	336	1389	85	1054	253	555	528	1725	1139	808	762	3438	4200

INTERSECTION: Southern & 48th St. Lagging
SURVEY DATE: 06/13/90 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
5:00 - 5:15	50	25	50	76	17	52	108	45	30	71	50	64	69	56	59
5:15 - 5:30	66	33	66	102	62	56	45	41	38	95	56	42	58	70	68
5:30 - 5:45	63	28	51	64	25	29	48	33	35	62	29	38	48	44	45
5:45 - 6:00	81	21	26	12	19	19	57	24	31	15	19	35	41	17	22
TOTAL	64	27	50	67	27	41	67	37	33	64	40	47	54.92	49.28	50.30

INTERSECTION: Southern & Stewart Leading
SURVEY DATE: 09/14/89 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	15 Min Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	65	54	43	20	104	193	103	120	119	63	297	223	315	387	702
5:00-5:15 pm	71	45	52	18	66	300	86	138	116	70	366	224	275	501	776
5:15-5:30 pm	60	36	24	26	76	298	94	144	96	50	374	238	254	504	758
5:30-5:45 pm	56	58	29	28	98	383	91	226	114	57	481	317	274	695	969
TOTAL	252	193	148	92	344	1174	374	628	445	240	1518	1002	1118	2087	3205

INTERSECTION: Southern & Stewart Leading
SURVEY DATE: 09/14/89 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	15 Min Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	30	60	23	27	39	281	32	268	90	50	320	300	124	636	760
5:00-5:15 pm	31	55	24	19	23	347	29	305	86	43	370	334	107	726	833
5:15-5:30 pm	27	44	17	25	26	350	35	322	71	42	376	357	105	741	846
5:30-5:45 pm	28	53	15	26	24	368	40	359	81	41	392	399	107	806	913
TOTAL	116	212	79	97	112	1346	136	1254	328	176	1458	1390	443	2909	3352

INTERSECTION: Southern & Stewart Leading
SURVEY DATE: 09/14/89 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	15 Min Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	33	14	28	11	40	10	48	7	20	19	14	11	38	9	14
5:00-5:15 pm	34	12	33	14	43	13	44	7	20	24	15	10	39	10	14
5:15-5:30 pm	33	12	21	16	44	13	40	7	20	18	15	10	36	10	13
5:30-5:45 pm	30	16	29	16	61	16	34	9	21	21	18	12	38	13	16
TOTAL	33	14	28	14	46	13	41	8	20	20	16	11	37.86	10.76	14.34

INTERSECTION: Southern & Stewart Combination
SURVEY DATE: 10/12/89 Delay

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	15 Min Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	67	93	38	25	116	229	54	138	160	63	345	192	275	485	760
5:00-5:15 pm	72	101	23	33	62	257	43	131	173	56	319	174	200	522	722
5:15-5:30 pm	75	43	42	32	135	235	56	131	118	74	370	187	308	441	749
5:30-5:45 pm	84	56	72	30	86	193	50	137	140	102	279	187	292	416	708
TOTAL	298	293	175	120	399	914	203	537	591	295	1313	740	1075	1864	2939

INTERSECTION: Southern & Stewart Combination
SURVEY DATE: 10/12/89 Volume

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	15 Min Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	32	64	20	23	29	310	34	304	96	43	339	338	115	701	816
5:00-5:15 pm	35	58	13	21	35	357	31	297	93	34	392	328	114	733	847
5:15-5:30 pm	35	41	34	24	28	319	40	329	76	58	347	369	137	713	850
5:30-5:45 pm	28	49	23	31	32	361	36	314	77	54	393	350	119	755	874
TOTAL	130	212	90	99	124	1347	141	1244	342	189	1471	1385	485	2902	3387

INTERSECTION: Southern & Stewart Combination
SURVEY DATE: 10/12/89 Delay Per Vehicle (sec/veh)

	Northbound		Southbound		Eastbound		Westbound		NB	SB	EB	WB	15 Min Intersection Totals		
Time Period	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Lt	Th/Rt	Total	Total	Total	Total	Lt	Th/Rt	Total
4:45-5:00 pm	31	22	29	16	60	11	24	7	25	22	15	9	36	10	14
5:00-5:15 pm	31	26	27	24	27	11	21	7	28	25	12	8	26	11	13
5:15-5:30 pm	32	16	19	20	72	11	21	6	23	19	16	8	34	9	13
5:30-5:45 pm	45	17	47	15	40	8	21	7	27	28	11	8	37	8	12
TOTAL	34	21	29	18	48	10	22	6	26	23	13	8	33.25	9.63	13.02

APPENDIX D-2

CITY OF PHOENIX 3RD CAR ACTUATION DATA

The City of Phoenix collected data on the number of actuation and left turn volumes for a comparison of 3rd car and 1st car actuation. The City of Phoenix recorded the number of times the left arrow was actuated and the left turn volumes for a 24 hour period of 3rd car actuation and a 24 hour period of 1st car actuation.

The following three tables contain a summary of this data as it relates to the delay studies performed as part of this research effort. The first column of data represents the number of times the left arrow was actuated during the PM peak hour under 3rd car actuation. The second column is the City of Phoenix recorded left volumes for the same time period. The third column is the left turn volumes recorded during the delay study for 3rd car actuation. It should be noted that the 3rd car actuation studies and the 3rd car delay studies were not performed on the same days. The forth column represents the number of times the left arrow was actuated during the PM peak under 1st car actuation. The fifth column is the left turn volumes recorded by the City of Phoenix for the same time period. The sixth column is the left turn volumes recorded during the delay studies for 1st car actuation.

"Vehicles/actuation" is calculated as the quotient of "actuation volume" and "times actuated" for the PM peak hour. The "24 hour vehicles/actuation" values are those reported by the City of Phoenix for the 24 hours of recorded data.

3rd Car Versus 1st Car Actuation 35th Ave./Dunlap

	Delay Study Date	Actuation Study Date
3rd Car:	5/3/90	5/17/90
1st Car:	5/16/90	5/16/90

NORTHBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:15-4:30	4	35	53	8	30	32
4:30-4:45	7	40	54	7	34	35
4:45-5:00	8	44	50	10	46	53
5:00-5:15	5	39	49	9	51	62
TOTAL	24	158	206	34	161	182
VEHS./ACTUATION		6.6			4.7	
24 HR. VEHS./ACT.*		14.5			4.1	

SOUTHBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:15-4:30	3	28	38	9	24	26
4:30-4:45	4	28	40	8	25	24
4:45-5:00	1	20	45	7	33	30
5:00-5:15	4	23	23	8	29	32
TOTAL	12	99	146	32	111	112
VEHS./ACTUATION		8.3			3.5	
24 HR. VEHS./ACT.*		7.2			3.9	

EASTBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:15-4:30	3	36	48	8	49	30
4:30-4:45	6	50	55	10	59	51
4:45-5:00	8	58	51	10	61	66
5:00-5:15	8	60	69	10	50	54
TOTAL	25	204	223	38	219	201
VEHS./ACTUATION		8.2			5.8	

WESTBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:15-4:30	1	35	41	9	33	45
4:30-4:45	5	37	50	10	47	40
4:45-5:00	3	36	55	10	46	52
5:00-5:15	3	36	52	9	44	63
TOTAL	12	144	198	38	170	200
VEHS./ACTUATION		12.0			4.5	

3rd Car Versus 1st Car Actuation — 43rd Ave./Northern

	Delay Study Date	Actuation Study Date
3rd Car:	5/1/90	5/16/90
1st Car:	5/17/90	5/17/90

NORTHBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:30-4:45	7	47	53	8	49	51
4:45-5:00	6	37	51	10	57	53
5:00-5:15	7	51	58	10	49	55
5:15-5:30	7	48	66	9	46	62
TOTAL	27	183	228	37	201	221
VEHS./ACTUATION		6.8			5.4	

SOUTHBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:30-4:45	5	31	31	9	38	37
4:45-5:00	6	43	41	9	38	45
5:00-5:15	5	33	46	8	33	39
5:15-5:30	7	36	34	9	37	47
TOTAL	23	143	152	35	146	168
VEHS./ACTUATION		6.2			4.2	

EASTBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:30-4:45	8	42	60	8	41	44
4:45-5:00	7	45	49	9	42	44
5:00-5:15	8	41	59	10	53	67
5:15-5:30	7	31	57	10	44	53
TOTAL	30	159	225	37	180	208
VEHS./ACTUATION		5.3			4.9	
24 HR. VEHS./ACT.*		8.0			3.6	

WESTBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
4:30-4:45	3	25	37	10	40	44
4:45-5:00	5	32	46	10	34	37
5:00-5:15	7	36	29	9	27	34
5:15-5:30	7	33	50	8	27	33
TOTAL	22	126	162	37	128	148
VEHS./ACTUATION		5.7			3.5	
24 HR. VEHS./ACT.*		12.7			4.6	

3rd Car Versus 1st Car Actuation — 51st St./Elliot

	Delay Study Date	Actuation Study Date
3rd Car:	5/21/90	5/16/90
1st Car:	5/14/90	5/14/90

EASTBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
5:00-5:15	0	18	17	4	10	7
5:15-5:30	0	12	15	4	11	5
5:30-5:45	0	16	12	5	20	13
5:45-6:00	0	8	15	1	8	6
TOTAL	0	54	59	14	49	31
VEHS./ACTUATION		--			3.5	
24 HR. VEHS./ACT.		34.2			4.9	

WESTBOUND LEFT TURN

Time	3rd Car Study Times Actuated	3rd Car Study Actuation Volume	3rd Car Study Delay Volume	1st Car Study Times Actuated	1st Car Study Actuation Volume	1st Car Study Delay Volume
5:00-5:15	6	62	83	9	80	94
5:15-5:30	5	57	80	10	82	64
5:30-5:45	6	72	83	12	80	97
5:45-6:00	3	74	81	11	85	59
TOTAL	20	265	327	42	327	314
VEHS./ACTUATION		13.3			7.8	
24 HR. VEHS./ACT.		18.0			8.7	

APPENDIX E

PIMA COUNTY INTERSECTION ANALYSIS

TABLE E-1
PIMA COUNTY SIGNAL OPERATION ANALYSIS
STOP DELAY PER APPROACH VEHICLE

INTERSECTION	BEFORE: LEFT TURNS	BEFORE: THRU & RIGHT TURNS	AFTER: LEFT TURNS	AFTER: THRU & RIGHT TURNS
	DELAY PER APPROACH VEHICLE (SEC)			
AJO WAY/ALVERNON WAY				
EASTBOUND	34.99	14.23	40.86	37.72
WESTBOUND	31.00	27.97	27.50	37.05
NORTHBOUND	56.26	12.92	55.58	15.41
SOUTHBOUND	19.00	9.62	37.59	13.41
ALVERNON WAY/IRVINGTON RD.				
EASTBOUND	14.45	15.92	35.06	37.99
WESTBOOUND	15.15	15.28	17.33	19.81
NORTHBOUND	17.83	12.52	27.46	13.38
SOUTHBOUND	21.27	9.03	26.93	10.88
CAMPBELL AVE./SKYLINE DR.				
EASTBOUND	*	*	*	*
WESTBOUND	33.15	7.08	36.06	11.73
NORTHBOUND	26.17	12.57	34.72	14.16
SOUTHBOUND	29.81	16.98	36.65	17.07
FIRST AVE./ORANGE GROVE RD.				
EASTBOUND	21.64	11.02	23.79	9.63
WESTBOUND	33.62	13.28	35.76	13.42
NORTHBOUND	13.57	7.76	17.34	10.22
SOUTHBOUND	13.59	13.65	24.56	12.93
FIRST AVE./RIVER RD.				
EASTBOUND	42.81	17.21	45.49	14.88
WESTBOUND	41.32	15.21	39.72	14.98
NORTHBOUND	15.48	15.40	22.75	16.23
SOUTHBOUND	*	*	30.16	17.67
INA RD./THORNYDALE RD.				
EASTBOUND	37.15	16.89	*	*
WESTBOUND	34.57	20.10	*	*
NORTHBOUND	22.45	16.87	*	*
SOUTHBOUND	18.69	16.94	*	*
KOLB RD./VALENCIA RD.				
EASTBOUND	15.48	6.27	22.96	13.63
WESTBOUND	20.00	23.91	40.00	17.46
NORTHBOUND	34.59	14.57	41.00	23.93
SOUTHBOUND	37.38	5.09	32.69	7.96

* Indicates data not available

PIMA COUNTY SIGNAL OPERATION ANALYSIS
STOP DELAY PER APPROACH VEHICLE (CONTINUED)

	DELAY PER APPROACH VEHICLE (SEC)			
	BEFORE		AFTER	
INTERSECTION	LEFT TURNS	THRU & RIGHT TURNS	LEFT TURNS	THRU & RIGHT TURNS
PALO VERDE WAY/VALENCIA RD.				
EASTBOUND	20.37	1.74	16.81	2.57
WESTBOUND	5.00	5.27	13.12	10.75
NORTHBOUND	22.50	10.84	20.00	17.62
SOUTHBOUND	24.00	30.00	29.79	22.59

Note: Due to a free flow right turn lane on the southbound approach at Palo Verde Way and Valencia Road, the delay value is for the southbound through movement only. The right turn approach volume was not include in the computations.

APPENDIX F

TRAVEL TIME STUDY

F-1 GLENDALE COMPARISONS

F-2 TEMPE COMPARISONS

F-3 SAMPLE TRAVEL TIME OUTPUT

F-4 SCOTTSDALE TRAVEL TIME DATA

APPENDIX F-1

GLENDALE COMPARISONS

TABLE F-1

GLENDALE TRAVEL TIME STUDIES
EXISTING LEADING MINUS FORCAST LEADING

ROUTE	TIME	DELAY: WEIGHTED EXISTING LEADING (VEH-HR)	DELAY: WEIGHTED FORCAST LEADING (VEH-HR)	DELAY: EXISTING LEADING MINUS FORCAST LEADING (VEH-HR)	DELAY: $0.616 /(VEH-HR) COST	TRAVEL TIME: WEIGHTED EXISTING LEADING (VEH-HR)	TRAVEL TIME: WEIGHTED FORCAST LEADING (VEH-HR)	TRAVEL TIME: EXISTING LEADING MINUS FORCAST LEADING (VEH-HR)	TRAVEL TIME: $3.35 /(VEH-HR) COST	STOPS: WEIGHTED EXISTING LEADING (KVEH-STP)	STOPS: WEIGHTED FORCAST LEADING (KVEH-STP)	STOPS: EXISTING LEADING MINUS FORCAST LEADING (KVEH-STP)	STOPS: $41.00 /(KVEH-STP) COST	TOTAL COST
51ST AVE NB	AM	7	45	−38	($23.68)	124	162	−38	($128.80)	2.4	5.9	−3.5	($145.51)	($297.99)
51ST AVE NB	MID	20	25	−5	($3.07)	180	188	−8	($27.35)	3.3	3.3	0.0	$0.00	($30.43)
51ST AVE NB	OF	77	40	37	$22.81	475	417	58	$193.38	11.8	7.8	3.9	$160.76	$376.95
51ST AVE NB	PM	106	81	25	$15.43	413	383	30	$100.12	14.5	14.5	0.0	$0.00	$115.55
51ST AVE SB	AM	42	64	−23	($14.09)	359	382	−23	($76.61)	9.1	12.2	−3.0	($125.01)	($215.70)
51ST AVE SB	MID	24	49	−24	($15.02)	186	210	−24	($81.71)	4.9	4.9	0.0	$0.00	($96.73)
51ST AVE SB	OF	54	25	29	$17.71	466	426	40	$134.84	12.4	8.3	4.1	$169.74	$322.29
51ST AVE SB	PM	14	79	−66	($40.36)	210	285	−75	($250.83)	5.9	9.9	−4.0	($162.52)	($453.72)
59TH AVE NB	AM	6	24	−18	($11.18)	97	115	−18	($60.78)	2.5	3.8	−1.3	($51.50)	($123.45)
59TH AVE NB	MID	19	13	6	$3.70	143	131	12	$38.71	3.3	1.7	1.7	$68.22	$110.64
59TH AVE NB	OF	44	49	−4	($2.72)	345	362	−17	($55.55)	11.9	8.0	4.0	$163.18	$104.90
59TH AVE NB	PM	53	61	−8	($4.84)	225	239	−14	($48.55)	10.9	8.7	2.2	$89.13	$35.75
59TH AVE SB	AM	26	19	8	$4.63	192	182	11	$35.66	4.5	2.3	2.3	$92.41	$132.70
59TH AVE SB	MID	22	11	11	$6.59	143	134	9	$31.16	3.3	3.3	0.0	$0.00	$37.74
59TH AVE SB	OF	45	43	2	$1.26	320	323	−3	($10.28)	11.0	7.4	3.7	$150.92	$141.90
59TH AVE SB	PM	9	37	−28	($17.48)	92	131	−39	($129.26)	2.3	5.7	−3.4	($141.20)	($287.95)
NORTHERN AVE EB	AM	9	23	−14	($8.36)	92	106	−13	($43.77)	3.6	1.8	1.8	$74.17	$22.04
NORTHERN AVE EB	MID	14	35	−21	($12.79)	99	122	−22	($74.50)	3.6	5.3	−1.8	($72.94)	($160.22)
NORTHERN AVE EB	OF	12	46	−34	($20.70)	206	230	−24	($80.40)	4.3	4.3	0.0	$0.00	($101.10)
NORTHERN AVE EB	PM	28	44	−16	($9.76)	141	160	−19	($63.71)	6.8	11.4	−4.6	($187.12)	($260.59)
NORTHERN AVE WB	AM	12	48	−36	($21.91)	98	130	−33	($110.12)	3.9	3.9	0.0	$0.00	($132.03)
NORTHERN AVE WB	MID	10	10	0	$0.00	87	86	1	$4.69	3.4	1.7	1.7	$68.88	$73.57
NORTHERN AVE WB	OF	46	3	43	$26.64	274	237	37	$124.90	5.4	0.0	5.4	$220.13	$371.67
NORTHERN AVE WB	PM	63	62	1	$0.90	195	186	9	$29.46	10.6	7.9	2.6	$108.16	$138.52
OLIVE AVE EB	AM	18	22	−4	($2.52)	133	144	−11	($37.22)	4.2	4.2	0.0	$0.00	($39.74)
OLIVE AVE EB	MID	34	18	16	$9.98	176	157	19	$63.32	7.3	2.4	4.9	$199.26	$272.55
OLIVE AVE EB	OF	52	63	−11	($6.78)	334	347	−12	($41.47)	9.9	9.9	0.0	$0.00	($48.25)
OLIVE AVE EB	PM	27	55	−28	($17.12)	168	185	−17	($57.13)	6.8	6.8	0.0	$0.00	($74.25)
OLIVE AVE WB	AM	10	44	−35	($21.41)	102	150	−48	($161.09)	1.7	6.9	−5.1	($210.82)	($393.32)
OLIVE AVE WB	MID	26	17	8	$5.13	143	133	10	$33.89	4.3	2.1	2.1	$87.82	$126.84
OLIVE AVE WB	OF	39	70	−31	($18.88)	316	359	−43	($144.70)	10.0	10.0	0.0	$0.00	($163.58)
OLIVE AVE WB	PM	70	51	19	$11.91	257	213	43	$145.67	11.6	5.8	5.8	$237.72	$395.30
PEORIA AVE EB	AM	8	14	−6	($3.63)	44	50	−6	($18.49)	1.4	2.1	−0.7	($28.09)	($50.20)
PEORIA AVE EB	MID	20	7	14	$8.49	114	98	15	$51.28	3.7	1.8	1.8	$75.32	$135.09
PEORIA AVE EB	OF	28	22	6	$3.89	218	215	3	$10.57	3.8	7.6	−3.8	($155.23)	($140.77)
PEORIA AVE EB	PM	18	38	−20	($12.19)	113	130	−17	($57.77)	3.7	3.7	0.0	$0.00	($69.96)
PEORIA AVE WB	AM	8	7	0	$0.19	62	59	3	$11.24	2.2	1.1	1.1	$45.02	$56.45
PEORIA AVE WB	MID	12	11	0	$0.28	90	85	5	$15.34	3.3	1.6	1.6	$67.57	$83.19
PEORIA AVE WB	OF	36	35	1	$0.65	219	217	2	$7.06	7.6	7.6	0.0	$0.00	$7.71
PEORIA AVE WB	PM	11	31	−21	($12.77)	122	147	−24	($82.06)	4.5	4.5	0.0	$0.00	($94.82)
SAMPLE SIZE					40				40				40	40
MEAN DIFFERENCE					($4.03)				($20.27)				$19.96	($4.34)
STD DEVIATION					$13.75				$89.37				$112.24	$199.90
TEST STAT					−1.852				−1.435				1.125	−0.137
SIGNIFICANT?					N				N				N	N

TABLE F-2

GLENDALE TRAVEL TIME STUDIES
EXISTING LEADING MINUS FORCAST LAGGING

ROUTE	TIME	DELAY: WEIGHTED EXISTING LEADING (VDH-HR)	DELAY: WEIGHTED FORCAST LAGGING (VDH-HR)	DELAY: EXISTING LEADING MINUS FORCAST LAGGING (VDH-HR)	DELAY: $0.616 /(VDH-HR) COST	TRAVEL TIME: WEIGHTED EXISTING LEADING (VDH-HR)	TRAVEL TIME: WEIGHTED FORCAST LAGGING (VDH-HR)	TRAVEL TIME: EXISTING LEADING MINUS FORCAST LAGGING (VDH-HR)	TRAVEL TIME: $3.35 /(VDH-HR) COST	STOPS: WEIGHTED EXISTING LEADING (KVDH-STP)	STOPS: WEIGHTED FORCAST LAGGING (KVDH-STP)	STOPS: EXISTING LEADING MINUS FORCAST LAGGING (KVDH-STP)	STOPS: $41.00 /(KVDH-STP) COST	TOTAL COST
51ST AVE NB	AM	7	26	−20	($12.15)	124	143	−19	($62.75)	2.4	5.9	−3.5	($145.51)	($220.40)
51ST AVE NB	MD	20	25	−5	($3.07)	180	186	−7	($22.79)	3.3	8.2	−4.9	($200.86)	($226.73)
51ST AVE NB	OF	77	44	34	$20.80	475	408	66	$222.57	11.8	7.8	3.9	$160.76	$404.13
51ST AVE NB	PM	106	78	27	$16.92	413	380	32	$108.24	14.5	11.6	2.9	$119.23	$244.39
51ST AVE SB	AM	42	39	3	$1.57	359	351	8	$25.54	9.1	9.1	0.0	$0.00	$27.10
51ST AVE SB	MD	24	27	−2	($1.39)	186	185	1	$3.03	4.9	4.9	0.0	$0.00	$1.64
51ST AVE SB	OF	54	22	32	$19.84	466	409	56	$188.77	12.4	8.3	4.1	$169.74	$378.35
51ST AVE SB	PM	14	90	−77	($47.14)	210	289	−79	($263.74)	5.9	11.9	−5.9	($243.79)	($554.67)
59TH AVE NB	AM	6	19	−13	($8.17)	97	105	−9	($29.22)	2.5	3.8	−1.3	($51.50)	($88.88)
59TH AVE NB	MD	19	17	2	$1.42	143	135	7	$24.78	3.3	3.3	0.0	$0.00	$26.20
59TH AVE NB	OF	44	44	0	$0.00	345	333	12	$40.74	11.9	8.0	4.0	$163.18	$203.92
59TH AVE NB	PM	53	37	16	$10.04	225	195	30	$99.13	10.9	4.3	6.5	$267.40	$376.57
59TH AVE SB	AM	26	39	−13	($7.71)	192	206	−14	($46.14)	4.5	6.8	−2.3	($92.41)	($146.27)
59TH AVE SB	MD	22	25	−3	($2.01)	143	153	−10	($34.27)	3.3	5.0	−1.7	($68.63)	($104.91)
59TH AVE SB	OF	45	24	21	$13.23	320	281	39	$130.16	11.0	7.4	3.7	$150.92	$294.31
59TH AVE SB	PM	9	38	−29	($17.88)	92	127	−35	($116.44)	2.3	5.7	−3.4	($141.20)	($275.52)
NORTHERN AVE EB	AM	9	16	−7	($4.33)	92	97	−5	($16.83)	3.6	3.6	0.0	$0.00	($21.17)
NORTHERN AVE EB	MD	14	24	−10	($6.09)	99	105	−5	($18.21)	3.6	3.6	0.0	$0.00	($24.30)
NORTHERN AVE EB	OF	12	20	−8	($5.17)	206	199	7	$24.12	4.3	4.3	0.0	$0.00	$18.95
NORTHERN AVE EB	PM	28	56	−29	($17.57)	141	174	−34	($112.55)	6.8	11.4	−4.6	($187.12)	($317.24)
NORTHERN AVE WB	AM	12	30	−18	($10.95)	98	113	−16	($52.35)	3.9	3.9	0.0	$0.00	($63.31)
NORTHERN AVE WB	MD	10	9	1	$0.57	87	86	1	$4.69	3.4	3.4	0.0	$0.00	$5.26
NORTHERN AVE WB	OF	46	25	21	$12.86	274	258	16	$54.96	5.4	5.4	0.0	$0.00	$67.82
NORTHERN AVE WB	PM	63	29	34	$20.76	195	146	49	$164.47	10.6	2.6	7.9	$324.47	$509.71
OLIVE AVE EB	AM	18	23	−5	($3.24)	133	140	−7	($23.51)	4.2	6.3	−2.1	($86.31)	($113.05)
OLIVE AVE EB	MD	34	3	31	$19.13	176	138	38	$126.63	7.3	2.4	4.9	$199.26	$345.02
OLIVE AVE EB	OF	52	15	37	$22.88	334	286	48	$161.28	9.9	5.0	5.0	$203.03	$387.19
OLIVE AVE EB	PM	27	50	−23	($14.01)	168	176	−8	($25.39)	6.8	6.8	0.0	$0.00	($39.40)
OLIVE AVE WB	AM	10	29	−19	($11.73)	102	119	−17	($55.82)	1.7	3.4	−1.7	($70.27)	($137.83)
OLIVE AVE WB	MD	26	5	21	$12.83	143	120	23	$75.74	4.3	2.1	2.1	$87.82	$176.39
OLIVE AVE WB	OF	39	35	4	$2.57	316	305	11	$37.34	10.0	10.0	0.0	$0.00	$39.92
OLIVE AVE WB	PM	70	60	10	$5.95	257	226	31	$102.51	11.6	11.6	0.0	$0.00	$108.46
PEORIA AVE EB	AM	8	9	−1	($0.70)	44	44	1	$1.91	1.4	2.1	−0.7	($28.09)	($26.88)
PEORIA AVE EB	MD	20	12	9	$5.34	114	100	14	$47.86	3.7	1.8	1.8	$75.32	$128.52
PEORIA AVE EB	OF	28	36	−7	($4.53)	218	222	−4	($14.09)	3.8	7.6	−3.8	($155.23)	($173.85)
PEORIA AVE EB	PM	18	39	−20	($12.50)	113	137	−24	($79.86)	3.7	5.5	−1.8	($74.87)	($167.23)
PEORIA AVE WB	AM	8	9	−2	($1.13)	62	63	−0	($1.02)	2.2	2.2	0.0	$0.00	($2.15)
PEORIA AVE WB	MD	12	8	4	$2.26	90	86	4	$12.27	3.3	3.3	0.0	$0.00	$14.52
PEORIA AVE WB	OF	36	33	3	$1.95	219	212	7	$24.70	7.6	7.6	0.0	$0.00	$26.65
PEORIA AVE WB	PM	11	48	−38	($23.21)	122	161	−39	($130.45)	4.5	6.8	−2.3	($92.70)	($246.36)
SAMPLE SIZE					40				40				40	40
MEAN DIFFERENCE					($0.59)				$14.40				$7.07	$20.87
STD DEVIATION					$13.88				$94.38				$127.16	$226.36
TEST STAT					−0.271				0.965				0.351	0.583
SIGNIFICANT?					N				N				N	N

TABLE F-3

GLENDALE TRAVEL TIME STUDIES
EXISTING LEADING MINUS FORCAST COMBINATION

ROUTE	TIME	DELAY: WEIGHTED EXISTING LEADING (VEH-HR)	DELAY: WEIGHTED FORCAST COMBINATION (VEH-HR)	DELAY: EXISTING LEADING MINUS FORCAST COMBINATION (VEH-HR)	DELAY: $0.616 /(VEH-HR) COST	TRAVEL TIME: WEIGHTED EXISTING LEADING (VEH-HR)	TRAVEL TIME: WEIGHTED FORCAST COMBINATION (VEH-HR)	TRAVEL TIME: EXISTING LEADING MINUS FORCAST COMBINATION (VEH-HR)	TRAVEL TIME: $3.35 /(VEH-HR) COST	STOPS: WEIGHTED EXISTING LEADING (KVEH-STP)	STOPS: WEIGHTED FORCAST COMBINATION (KVEH-STP)	STOPS: EXISTING LEADING MINUS FORCAST COMBINATION (KVEH-STP)	STOPS: $41.00 /(KVEH-STP) COST	TOTAL COST
51ST AVE NB	AM	7	42	-36	($22.06)	124	157	-33	($111.19)	2.4	4.7	-2.4	($97.01)	($230.26)
51ST AVE NB	MD	20	12	8	$4.75	180	169	10	$34.95	3.3	3.3	0.0	$0.00	$39.70
51ST AVE NB	OF	77	34	44	$26.84	475	405	70	$233.52	11.8	7.8	3.9	$160.76	$421.12
51ST AVE NB	PM	106	25	81	$49.76	413	317	95	$319.31	14.5	5.8	8.7	$357.68	$726.76
51ST AVE SB	AM	42	82	-41	($25.04)	359	385	-26	($87.96)	9.1	12.2	-3.0	($125.01)	($238.01)
51ST AVE SB	MD	24	15	9	$5.56	186	170	15	$51.44	4.9	4.9	0.0	$0.00	$57.01
51ST AVE SB	OF	54	38	16	$9.92	466	446	20	$65.49	12.4	8.3	4.1	$169.74	$245.15
51ST AVE SB	PM	14	84	-70	($43.07)	210	277	-67	($225.01)	5.9	9.9	-4.0	($162.52)	($430.61)
59TH AVE NB	AM	6	29	-23	($14.18)	97	117	-21	($68.96)	2.5	5.0	-2.5	($102.99)	($186.13)
59TH AVE NB	MD	19	14	5	$2.85	143	135	8	$27.87	3.3	3.3	0.0	$0.00	$30.72
59TH AVE NB	OF	44	55	-11	($6.81)	345	349	-4	($14.81)	11.9	11.9	0.0	$0.00	($21.62)
59TH AVE NB	PM	53	33	21	$12.65	225	192	33	$109.24	10.9	4.3	6.5	$267.40	$389.29
59TH AVE SB	AM	26	21	6	$3.47	192	185	7	$23.07	4.5	4.5	0.0	$0.00	$26.54
59TH AVE SB	MD	22	9	13	$7.73	143	130	13	$43.62	3.3	3.3	0.0	$0.00	$51.35
59TH AVE SB	OF	45	28	17	$10.71	320	297	24	$78.78	11.0	11.0	0.0	$0.00	$89.49
59TH AVE SB	PM	9	37	-28	($17.48)	92	123	-30	($101.49)	2.3	5.7	-3.4	($141.20)	($260.17)
NORTHERN AVE EB	AM	9	6	4	$2.17	92	82	10	$33.67	3.6	1.8	1.8	$74.17	$110.00
NORTHERN AVE EB	MD	14	16	-2	($1.22)	99	98	1	$4.97	3.6	3.6	0.0	$0.00	$3.75
NORTHERN AVE EB	OF	12	29	-17	($10.35)	206	209	-2	($8.04)	4.3	4.3	0.0	$0.00	($18.39)
NORTHERN AVE EB	PM	28	48	-20	($12.10)	141	159	-18	($61.58)	6.8	9.1	-2.3	($93.56)	($167.25)
NORTHERN AVE WB	AM	12	37	-24	($14.94)	98	119	-22	($72.21)	3.9	5.8	-1.9	($79.54)	($166.69)
NORTHERN AVE WB	MD	10	10	0	$0.29	87	85	2	$7.82	3.4	3.4	0.0	$0.00	$8.10
NORTHERN AVE WB	OF	46	12	34	$21.13	274	243	31	$104.92	5.4	5.4	0.0	$0.00	$126.05
NORTHERN AVE WB	PM	63	24	39	$23.92	195	144	51	$169.38	10.6	5.3	5.3	$216.32	$409.62
OLIVE AVE EB	AM	18	58	-40	($24.85)	133	188	-54	($182.17)	4.2	8.4	-4.2	($172.61)	($379.63)
OLIVE AVE EB	MD	34	7	27	$16.63	176	136	39	$131.15	7.3	2.4	4.9	$199.26	$347.04
OLIVE AVE EB	OF	52	8	44	$27.11	334	274	61	$202.76	9.9	5.0	5.0	$203.03	$432.90
OLIVE AVE EB	PM	27	61	-34	($21.01)	168	195	-27	($88.88)	6.8	9.1	-2.3	($93.23)	($203.12)
OLIVE AVE WB	AM	10	26	-16	($9.97)	102	118	-16	($54.23)	1.7	1.7	0.0	$0.00	($64.20)
OLIVE AVE WB	MD	26	7	18	$11.36	143	123	20	$67.77	4.3	4.3	0.0	$0.00	$79.13
OLIVE AVE WB	OF	39	36	3	$1.72	316	308	8	$28.01	10.0	10.0	0.0	$0.00	$29.72
OLIVE AVE WB	PM	70	34	36	$22.32	257	194	63	$210.42	11.6	5.8	5.8	$237.72	$470.46
PEORIA AVE EB	AM	8	10	-2	($1.29)	44	44	0	$1.27	1.4	1.4	0.0	$0.00	($0.01)
PEORIA AVE EB	MD	20	8	13	$7.86	114	99	15	$49.57	3.7	1.8	1.8	$75.32	$132.75
PEORIA AVE EB	OF	28	22	6	$3.89	218	210	7	$24.66	3.8	7.6	-3.8	($155.23)	($126.68)
PEORIA AVE EB	PM	18	37	-19	($11.56)	113	130	-17	($57.77)	3.7	3.7	0.0	$0.00	($69.33)
PEORIA AVE WB	AM	8	12	-5	($2.82)	62	69	-6	($21.46)	2.2	3.3	-1.1	($45.02)	($69.29)
PEORIA AVE WB	MD	12	5	7	$4.51	90	81	9	$30.67	3.3	1.6	1.6	$67.57	$102.75
PEORIA AVE WB	OF	36	31	5	$3.24	219	211	8	$28.23	7.6	3.8	3.8	$155.47	$186.95
PEORIA AVE WB	PM	11	46	-35	($21.67)	122	161	-39	($130.45)	4.5	4.5	0.0	$0.00	($152.11)
SAMPLE SIZE					40				40				40	40
MEAN DIFFERENCE					$0.50				$19.91				$22.91	$43.32
STD DEVIATION					$17.71				$111.37				$125.98	$245.30
TEST STAT					0.178				1.131				1.150	1.117
SIGNIFICANT?					N				N				N	N

TABLE F-4

GLENDALE TRAVEL TIME STUDIES
FORCAST LEADING MINUS FORCAST LAGGING

ROUTE	TIME	DELAY: WEIGHTED FORCAST LEADING (VEH-HR)	DELAY: WEIGHTED FORCAST LAGGING (VEH-HR)	DELAY: FORCAST LEADING MINUS FORCAST LAGGING (VEH-HR)	DELAY: $0.616 /(VEH-HR) COST	TRAVEL TIME: WEIGHTED FORCAST LEADING (VEH-HR)	TRAVEL TIME: WEIGHTED FORCAST LAGGING (VEH-HR)	TRAVEL TIME: FORCAST LEADING MINUS FORCAST LAGGING (VEH-HR)	TRAVEL TIME: $3.35 /(VEH-HR) COST	STOPS: WEIGHTED FORCAST LEADING (KVEH-STP)	STOPS: WEIGHTED FORCAST LAGGING (KVEH-STP)	STOPS: FORCAST LEADING MINUS FORCAST LAGGING (KVEH-STP)	STOPS: $41.00 /(KVEH-STP) COST	TOTAL COST
51ST AVE NB	AM	45	26	19	$11.54	162	143	20	$66.05	5.9	5.9	0.0	$0.00	$77.59
51ST AVE NB	MD	25	25	0	$0.00	188	186	1	$4.56	3.3	8.2	-4.9	($200.86)	($196.30)
51ST AVE NB	OF	40	44	-3	($2.01)	417	408	9	$29.19	7.8	7.8	0.0	$0.00	$27.18
51ST AVE NB	PM	81	78	2	$1.49	383	380	2	$8.12	14.5	11.6	2.9	$119.23	$128.84
51ST AVE SB	AM	64	39	25	$15.65	382	351	30	$102.14	12.2	9.1	3.0	$125.01	$242.80
51ST AVE SB	MD	49	27	22	$13.63	210	185	25	$84.73	4.9	4.9	0.0	$0.00	$98.37
51ST AVE SB	OF	25	22	3	$2.13	426	409	16	$53.94	8.3	8.3	0.0	$0.00	$56.06
51ST AVE SB	PM	79	90	-11	($6.78)	285	289	-4	($12.91)	9.9	11.9	-2.0	($81.26)	($100.96)
59TH AVE NB	AM	24	19	5	$3.01	115	105	9	$31.56	3.8	3.8	0.0	$0.00	$34.57
59TH AVE NB	MD	13	17	-4	($2.28)	131	135	-4	($13.94)	1.7	3.3	-1.7	($68.22)	($84.44)
59TH AVE NB	OF	49	44	4	$2.72	362	333	29	$96.29	8.0	8.0	0.0	$0.00	$99.02
59TH AVE NB	PM	61	37	24	$14.88	239	195	44	$147.68	8.7	4.3	4.3	$178.27	$340.83
59TH AVE SB	AM	19	39	-20	($12.34)	182	206	-24	($81.80)	2.3	6.8	-4.5	($184.83)	($278.97)
59TH AVE SB	MD	11	25	-14	($8.59)	134	153	-20	($65.43)	3.3	5.0	-1.7	($68.63)	($142.65)
59TH AVE SB	OF	43	24	19	$11.97	323	281	42	$140.44	7.4	7.4	0.0	$0.00	$152.41
59TH AVE SB	PM	37	38	-1	($0.39)	131	127	4	$12.82	5.7	5.7	0.0	$0.00	$12.43
NORTHERN AVE EB	AM	23	16	7	$4.02	106	97	8	$26.93	1.8	3.6	-1.8	($74.17)	($43.21)
NORTHERN AVE EB	MD	35	24	11	$6.70	122	105	17	$56.29	5.3	3.6	1.8	$72.94	$135.92
NORTHERN AVE EB	OF	46	20	25	$15.52	230	199	31	$104.52	4.3	4.3	0.0	$0.00	$120.04
NORTHERN AVE EB	PM	44	56	-13	($7.81)	160	174	-15	($48.84)	11.4	11.4	0.0	$0.00	($56.65)
NORTHERN AVE WB	AM	48	30	18	$10.95	130	113	17	$57.77	3.9	3.9	0.0	$0.00	$68.72
NORTHERN AVE WB	MD	10	9	1	$0.57	86	86	0	$0.00	1.7	3.4	-1.7	($68.88)	($68.31)
NORTHERN AVE WB	OF	3	25	-22	($13.78)	237	258	-21	($69.95)	0.0	5.4	-5.4	($220.13)	($303.86)
NORTHERN AVE WB	PM	62	29	32	$19.86	186	146	40	$135.01	7.9	2.6	5.3	$216.32	$371.19
OLIVE AVE EB	AM	22	23	-1	($0.72)	144	140	4	$13.71	4.2	6.3	-2.1	($86.31)	($73.31)
OLIVE AVE EB	MD	18	3	15	$9.15	157	138	19	$63.32	2.4	2.4	0.0	$0.00	$72.46
OLIVE AVE EB	OF	63	15	48	$29.66	347	286	61	$202.76	9.9	5.0	5.0	$203.03	$435.45
OLIVE AVE EB	PM	55	50	5	$3.11	185	176	9	$31.74	6.8	6.8	0.0	$0.00	$34.85
OLIVE AVE WB	AM	44	29	16	$9.68	150	119	31	$105.27	6.9	3.4	3.4	$140.55	$255.49
OLIVE AVE WB	MD	17	5	12	$7.70	133	120	12	$41.86	2.1	2.1	0.0	$0.00	$49.56
OLIVE AVE WB	OF	70	35	35	$21.46	359	305	54	$182.04	10.0	10.0	0.0	$0.00	$203.50
OLIVE AVE WB	PM	51	60	-10	($5.95)	213	226	-13	($43.16)	5.8	11.6	-5.8	($237.72)	($286.83)
PEORIA AVE EB	AM	14	9	5	$2.93	50	44	6	$20.40	2.1	2.1	0.0	$0.00	$23.33
PEORIA AVE EB	MD	7	12	-5	($3.14)	98	100	-1	($3.42)	1.8	1.8	0.0	$0.00	($6.56)
PEORIA AVE EB	OF	22	36	-14	($8.42)	215	222	-7	($24.66)	7.6	7.6	0.0	$0.00	($33.08)
PEORIA AVE EB	PM	38	39	-1	($0.31)	130	137	-7	($22.09)	3.7	5.5	-1.8	($74.87)	($97.27)
PEORIA AVE WB	AM	7	9	-2	($1.32)	59	63	-4	($12.26)	1.1	2.2	-1.1	($45.02)	($58.59)
PEORIA AVE WB	MD	11	8	3	$1.97	85	86	-1	($3.07)	1.6	3.3	-1.6	($67.57)	($68.66)
PEORIA AVE WB	OF	35	33	2	$1.30	217	212	5	$17.64	7.6	7.6	0.0	$0.00	$18.94
PEORIA AVE WB	PM	31	48	-17	($10.45)	147	161	-14	($48.39)	4.5	6.8	-2.3	($92.70)	($151.54)
SAMPLE SIZE					40				40				40	40
MEAN DIFFERENCE					$3.43				$34.67				($12.90)	$25.21
STD DEVIATION					$9.68				$68.54				$101.52	$166.97
TEST STAT					2.242				3.200				-0.803	0.955
SIGNIFICANT?					Y				Y				N	N

TABLE F-5

GLENDALE TRAVEL TIME STUDIES
FORCAST LEADING MINUS FORCAST COMBINATION

ROUTE	TIME	DELAY: WEIGHTED FORCAST LEADING (VDH-HR)	DELAY: WEIGHTED FORCAST COMBINATION (VDH-HR)	DELAY: FORCAST LEADING MINUS FORCAST COMBINATION (VDH-HR)	DELAY: $0.616 /(VDH-HR) COST	TRAVEL TIME: WEIGHTED FORCAST LEADING (VDH-HR)	TRAVEL TIME: WEIGHTED FORCAST COMBINATION (VDH-HR)	TRAVEL TIME: FORCAST LEADING MINUS FORCAST COMBINATION (VDH-HR)	TRAVEL TIME: $3.35 /(VDH-HR) COST	STOPS: WEIGHTED FORCAST LEADING (KVDH-STP)	STOPS: WEIGHTED FORCAST COMBINATION (KVDH-STP)	STOPS: FORCAST LEADING MINUS FORCAST COMBINATION (KVDH-STP)	STOPS: $41.00 /(KVDH-STP) COST	TOTAL COST
51ST AVE NB	AM	45	42	3	$1.62	162	157	5	$17.61	5.9	4.7	1.2	$48.50	$67.74
51ST AVE NB	MD	25	12	13	$7.82	188	169	19	$62.30	3.3	3.3	0.0	$0.00	$70.13
51ST AVE NB	OF	40	34	7	$4.03	417	405	12	$40.14	7.8	7.8	0.0	$0.00	$44.16
51ST AVE NB	PM	81	25	56	$34.33	383	317	65	$219.19	14.5	5.8	8.7	$357.68	$611.21
51ST AVE SB	AM	64	82	−18	($10.96)	382	385	−3	($11.35)	12.2	12.2	0.0	$0.00	($22.31)
51ST AVE SB	MD	49	15	33	$20.59	210	170	40	$133.15	4.9	4.9	0.0	$0.00	$153.74
51ST AVE SB	OF	25	38	−13	($7.79)	426	446	−21	($69.34)	8.3	8.3	0.0	$0.00	($77.14)
51ST AVE SB	PM	79	84	−4	($2.71)	285	277	8	$25.82	9.9	9.9	0.0	$0.00	$23.11
59TH AVE NB	AM	24	29	−5	($3.01)	115	117	−2	($8.18)	3.8	5.0	−1.3	($51.50)	($62.69)
59TH AVE NB	MD	13	14	−1	($0.85)	131	135	−3	($10.84)	1.7	3.3	−1.7	($68.22)	($79.92)
59TH AVE NB	OF	49	55	−7	($4.09)	362	349	12	$40.74	8.0	11.9	−4.0	($163.18)	($126.53)
59TH AVE NB	PM	61	33	28	$17.48	239	192	47	$157.80	8.7	4.3	4.3	$178.27	$353.55
59TH AVE SB	AM	19	21	−2	($1.16)	182	185	−4	($12.58)	2.3	4.5	−2.3	($92.41)	($106.16)
59TH AVE SB	MD	11	9	2	$1.15	134	130	4	$12.46	3.3	3.3	0.0	$0.00	$13.61
59TH AVE SB	OF	43	28	15	$9.45	323	297	27	$89.06	7.4	11.0	−3.7	($150.92)	($52.41)
59TH AVE SB	PM	37	37	0	$0.00	131	123	8	$27.78	5.7	5.7	0.0	$0.00	$27.78
NORTHERN AVE EB	AM	23	6	17	$10.52	106	82	23	$77.44	1.8	1.8	0.0	$0.00	$87.96
NORTHERN AVE EB	MD	35	16	19	$11.57	122	98	24	$79.46	5.3	3.6	1.8	$72.94	$163.97
NORTHERN AVE EB	OF	46	29	17	$10.35	230	209	22	$72.36	4.3	4.3	0.0	$0.00	$82.71
NORTHERN AVE EB	PM	44	48	−4	($2.34)	160	159	1	$2.12	11.4	9.1	2.3	$93.56	$93.34
NORTHERN AVE WB	AM	48	37	11	$6.97	130	119	11	$37.91	3.9	5.8	−1.9	($79.54)	($34.66)
NORTHERN AVE WB	MD	10	10	0	$0.29	86	85	1	$3.13	1.7	3.4	−1.7	($68.88)	($65.47)
NORTHERN AVE WB	OF	3	12	−9	($5.51)	237	243	−6	($19.98)	0.0	5.4	−5.4	($220.13)	($245.63)
NORTHERN AVE WB	PM	62	24	37	$23.02	186	144	42	$139.92	7.9	5.3	2.6	$108.16	$271.10
OLIVE AVE EB	AM	22	58	−36	($22.33)	144	188	−43	($144.95)	4.2	8.4	−4.2	($172.61)	($339.89)
OLIVE AVE EB	MD	18	7	11	$6.65	157	136	20	$67.84	2.4	2.4	0.0	$0.00	$74.49
OLIVE AVE EB	OF	63	8	55	$33.89	347	274	73	$244.23	9.9	5.0	5.0	$203.03	$481.16
OLIVE AVE EB	PM	55	61	−6	($3.89)	185	195	−9	($31.74)	6.8	9.1	−2.3	($93.23)	($128.87)
OLIVE AVE WB	AM	44	26	19	$11.44	150	118	32	$106.86	6.9	1.7	5.1	$210.82	$329.12
OLIVE AVE WB	MD	17	7	10	$6.23	133	123	10	$33.89	2.1	4.3	−2.1	($87.82)	($47.71)
OLIVE AVE WB	OF	70	36	33	$20.60	359	308	52	$172.70	10.0	10.0	0.0	$0.00	$193.30
OLIVE AVE WB	PM	51	34	17	$10.42	213	194	19	$64.74	5.8	5.8	0.0	$0.00	$75.16
PEORIA AVE EB	AM	14	10	4	$2.34	50	44	6	$19.76	2.1	1.4	0.7	$28.09	$50.19
PEORIA AVE EB	MD	7	8	−1	($0.63)	98	99	−1	($1.71)	1.8	1.8	0.0	$0.00	($2.34)
PEORIA AVE EB	OF	22	22	0	$0.00	215	210	4	$14.09	7.6	7.6	0.0	$0.00	$14.09
PEORIA AVE EB	PM	38	37	1	$0.62	130	130	0	$0.00	3.7	3.7	0.0	$0.00	$0.62
PEORIA AVE WB	AM	7	12	−5	($3.01)	59	69	−10	($32.70)	1.1	3.3	−2.2	($90.04)	($125.74)
PEORIA AVE WB	MD	11	5	7	$4.23	85	81	5	$15.34	1.6	1.6	0.0	$0.00	$19.57
PEORIA AVE WB	OF	35	31	4	$2.60	217	211	6	$21.17	7.6	3.8	3.8	$155.47	$179.24
PEORIA AVE WB	PM	31	46	−14	($8.90)	147	161	−14	($48.39)	4.5	4.5	0.0	$0.00	($57.29)
SAMPLE SIZE					40				40				40	40
MEAN DIFFERENCE					$4.53				$40.18				$2.95	$47.66
STD DEVIATION					$11.29				$75.93				$110.62	$179.58
TEST STAT					2.536				3.347				0.169	1.678
SIGNIFICANT?					Y				Y				N	N

TABLE F-6

GLENDALE TRAVEL TIME STUDIES
FORCAST LAGGING MINUS FORCAST COMBINATION

		DELAY				TRAVEL TIME				STOPS				TOTAL
ROUTE	TIME	WEIGHTED FORCAST LAGGING (VEH-HR)	WEIGHTED FORCAST COMBINATION (VEH-HR)	FORCAST LAGGING MINUS FORCAST COMBINATION (VEH-HR)	$0.616 /(VEH-HR) COST	WEIGHTED FORCAST LAGGING (VEH-HR)	WEIGHTED FORCAST COMBINATION (VEH-HR)	FORCAST LAGGING MINUS FORCAST COMBINATION (VEH-HR)	$3.35 /(VEH-HR) COST	WEIGHTED FORCAST LAGGING (KVEH-STP)	WEIGHTED FORCAST COMBINATION (KVEH-STP)	FORCAST LAGGING MINUS FORCAST COMBINATION (KVEH-STP)	$41.00 /(KVEH-STP) COST	COST
51ST AVE NB	AM	26	42	-16	($9.92)	143	157	-14	($48.44)	5.9	4.7	1.2	$48.50	($9.85)
51ST AVE NB	MD	25	12	13	$7.82	186	169	17	$57.74	8.2	3.3	4.9	$200.86	$266.43
51ST AVE NB	OF	44	34	10	$6.04	408	405	3	$10.95	7.8	7.8	0.0	$0.00	$16.98
51ST AVE NB	PM	78	25	53	$32.84	380	317	63	$211.07	11.6	5.8	5.8	$238.46	$482.37
51ST AVE SB	AM	39	82	-43	($26.61)	351	385	-34	($113.49)	9.1	12.2	-3.0	($125.01)	($265.11)
51ST AVE SB	MD	27	15	11	$6.96	185	170	14	$48.42	4.9	4.9	0.0	$0.00	$55.37
51ST AVE SB	OF	22	38	-16	($9.92)	409	446	-37	($123.28)	8.3	8.3	0.0	$0.00	($133.20)
51ST AVE SB	PM	90	84	7	$4.07	289	277	12	$38.73	11.9	9.9	2.0	$81.26	$124.06
59TH AVE NB	AM	19	29	-10	($6.02)	105	117	-12	($39.74)	3.8	5.0	-1.3	($51.50)	($97.25)
59TH AVE NB	MD	17	14	2	$1.42	135	135	1	$3.10	3.3	3.3	0.0	$0.00	$4.52
59TH AVE NB	OF	44	55	-11	($6.81)	333	349	-17	($55.55)	8.0	11.9	-4.0	($163.18)	($225.54)
59TH AVE NB	PM	37	33	4	$2.60	195	192	3	$10.12	4.3	4.3	0.0	$0.00	$12.72
59TH AVE SB	AM	39	21	18	$11.18	206	185	21	$69.22	6.8	4.5	2.3	$92.41	$172.82
59TH AVE SB	MD	25	9	16	$9.74	153	130	23	$77.89	5.0	3.3	1.7	$68.63	$156.26
59TH AVE SB	OF	24	28	-4	($2.52)	281	297	-15	($51.38)	7.4	11.0	-3.7	($150.92)	($204.82)
59TH AVE SB	PM	38	37	1	$0.39	127	123	4	$14.96	5.7	5.7	0.0	$0.00	$15.35
NORTHERN AVE EB	AM	16	6	11	$6.50	97	82	15	$50.50	3.6	1.8	1.8	$74.17	$131.17
NORTHERN AVE EB	MD	24	16	8	$4.87	105	98	7	$23.18	3.6	3.6	0.0	$0.00	$28.05
NORTHERN AVE EB	OF	20	29	-8	($5.17)	199	209	-10	($32.16)	4.3	4.3	0.0	$0.00	($37.33)
NORTHERN AVE EB	PM	56	48	9	$5.47	174	159	15	$50.96	11.4	9.1	2.3	$93.56	$149.99
NORTHERN AVE WB	AM	30	37	-6	($3.98)	113	119	-6	($19.86)	3.9	5.8	-1.9	($79.54)	($103.38)
NORTHERN AVE WB	MD	9	10	-0	($0.29)	86	85	1	$3.13	3.4	3.4	0.0	$0.00	$2.84
NORTHERN AVE WB	OF	25	12	13	$8.27	258	243	15	$49.96	5.4	5.4	0.0	$0.00	$58.23
NORTHERN AVE WB	PM	29	24	5	$3.16	146	144	1	$4.91	2.6	5.3	-2.6	($108.16)	($100.09)
OLIVE AVE EB	AM	23	58	-35	($21.61)	140	188	-47	($158.66)	6.3	8.4	-2.1	($86.30)	($266.58)
OLIVE AVE EB	MD	3	7	-4	($2.49)	138	136	1	$4.52	2.4	2.4	0.0	$0.00	$2.03
OLIVE AVE EB	OF	15	8	7	$4.24	286	274	12	$41.47	5.0	5.0	0.0	$0.00	$45.71
OLIVE AVE EB	PM	50	61	-11	($7.00)	176	195	-19	($63.48)	6.8	9.1	-2.3	($93.23)	($163.72)
OLIVE AVE WB	AM	29	26	3	$1.76	119	118	0	$1.59	3.4	1.7	1.7	$70.27	$73.63
OLIVE AVE WB	MD	5	7	-2	($1.47)	120	123	-2	($7.97)	2.1	4.3	-2.1	($87.82)	($97.26)
OLIVE AVE WB	OF	35	36	-1	($0.86)	305	308	-3	($9.34)	10.0	10.0	0.0	$0.00	($10.19)
OLIVE AVE WB	PM	60	34	27	$16.37	226	194	32	$107.91	11.6	5.8	5.8	$237.72	$361.99
PEORIA AVE EB	AM	9	10	-1	($0.59)	44	44	-0	($0.64)	2.1	1.4	0.7	$28.09	$26.86
PEORIA AVE EB	MD	12	8	4	$2.51	100	99	1	$1.71	1.8	1.8	0.0	$0.00	$4.22
PEORIA AVE EB	OF	36	22	14	$8.42	222	210	12	$38.75	7.6	7.6	0.0	$0.00	$47.18
PEORIA AVE EB	PM	39	37	2	$0.94	137	130	7	$22.09	5.5	3.7	1.8	$74.87	$97.89
PEORIA AVE WB	AM	9	12	-3	($1.69)	63	69	-6	($20.44)	2.2	3.3	-1.1	($45.02)	($67.14)
PEORIA AVE WB	MD	8	5	4	$2.26	86	81	5	$18.40	3.3	1.6	1.6	$67.57	$88.23
PEORIA AVE WB	OF	33	31	2	$1.30	212	211	1	$3.53	7.6	3.8	3.8	$155.47	$160.30
PEORIA AVE WB	PM	48	46	3	$1.55	161	161	0	$0.00	6.8	4.5	2.3	$92.70	$94.25
SAMPLE SIZE					40				40				40	40
MEAN DIFFERENCE					$1.09				$5.51				$15.85	$22.45
STD DEVIATION					$9.52				$62.98				$93.42	$153.48
TEST STAT					0.726				0.553				1.073	0.925
SIGNIFICANT?					N				N				N	N

APPENDIX F-2

TEMPE COMPARISONS

TABLE F-7

TEMPE TRAVEL TIME STUDIES
EXISTING LEADING MINUS FORCAST LEADING

		DELAY				TRAVEL TIME				STOPS				TOTAL
ROUTE	TIME	WEIGHTED EXISTING LEADING (VEH-HR)	WEIGHTED FORCAST LEADING (VEH-HR)	EXISTING LEADING MINUS FORCAST LEADING (VEH-HR)	$0.616 /(VEH-HR) COST	WEIGHTED EXISTING LEADING (VEH-HR)	WEIGHTED FORCAST LEADING (VEH-HR)	EXISTING LEADING MINUS FORCAST LEADING (VEH-HR)	$3.35 /(VEH-HR) COST	WEIGHTED EXISTING LEADING (KVEH-STP)	WEIGHTED FORCAST LEADING (KVEH-STP)	EXISTING LEADING MINUS FORCAST LEADING (KVEH-STP)	$41.00 /(KVEH-STP) COST	COST
48TH ST. NB	AM	17	4	14	$8.33	117	101	16	$55.04	3.5	3.5	0.0	$0.00	$63.37
48TH ST. NB	MID	28	4	24	$15.01	164	141	23	$77.07	4.9	4.9	0.0	$0.00	$92.08
48TH ST. NB	PM	25	40	-15	($9.04)	96	112	-16	($53.83)	2.5	2.5	0.0	$0.00	($62.87)
48TH ST. SB	AM	5	36	-31	($19.33)	48	79	-31	($105.11)	1.5	2.9	-1.5	($60.15)	($184.59)
48TH ST. SB	MID	49	14	35	$21.35	246	191	55	$185.78	6.2	0.0	6.2	$255.80	$462.93
48TH ST. SB	PM	44	11	33	$20.55	202	157	44	$147.55	14.4	4.8	9.6	$394.01	$562.12
SOUTHERN AVE EB	AM	12	16	-4	($2.28)	55	54	1	$2.87	3.1	2.1	1.0	$42.11	$42.69
SOUTHERN AVE EB	MID													
SOUTHERN AVE EB	PM	54	63	-8	($5.21)	241	237	4	$12.15	13.1	8.7	4.4	$178.43	$185.37
SOUTHERN AVE WB	AM	15	58	-43	($26.68)	149	198	-49	($164.04)	3.4	10.2	-6.8	($277.98)	($468.70)
SOUTHERN AVE WB	MID													
SOUTHERN AVE WB	PM	23	17	5	$3.20	103	96	7	$23.22	6.2	2.1	4.2	$170.48	$196.90
BROADWAY RD EB	AM	6	15	-9	($5.68)	104	110	-6	($19.86)	2.4	2.4	0.0	$0.00	($25.54)
BROADWAY RD EB	MID	49	106	-57	($35.41)	483	540	-57	($192.59)	20.7	10.3	10.3	$424.27	$196.27
BROADWAY RD EB	PM	22	117	-95	($58.25)	271	358	-87	($290.40)	5.7	17.0	-11.3	($465.27)	($813.92)
BROADWAY RD WB	AM	30	52	-22	($13.43)	202	208	-6	($19.22)	12.4	4.1	8.3	$338.74	$306.09
BROADWAY RD WB	MID	117	72	46	$28.23	550	481	69	$230.28	20.6	10.3	10.3	$422.75	$681.26
BROADWAY RD WB	PM	100	43	57	$35.26	283	201	82	$274.44	12.1	4.0	8.1	$331.28	$640.98
PREST RD. NB	AM	9	31	-21	($13.22)	79	98	-19	($64.19)	5.5	2.8	2.8	$113.12	$35.72
PREST RD. NB	MID	17	6	11	$6.68	161	149	12	$41.50	5.6	5.6	0.0	$0.00	$48.17
PREST RD. NB	PM	8	21	-14	($8.46)	74	84	-11	($36.82)	2.5	2.5	0.0	$0.00	($45.28)
PREST RD. SB	AM	1	4	-2	($1.33)	34	35	-1	($4.84)	1.3	0.0	1.3	$53.30	$47.13
PREST RD. SB	MID	1	1	0	$0.00	120	130	-10	($32.33)	0.0	0.0	0.0	$0.00	($32.33)
PREST RD. SB	PM	19	11	9	$5.32	113	99	14	$48.23	3.5	3.5	0.0	$0.00	$53.55
SAMPLE SIZE					22				22				22	22
MEAN DIFFERENCE					($2.47)				$5.22				$87.31	$90.06
STD DEVIATION					$21.23				$132.26				$219.51	$336.58
TEST STAT					-0.546				0.185				1.866	1.255
SIGNIFICANT?					N				N				N	N

TABLE F-8

TEMPE TRAVEL TIME STUDIES
EXISTING LEADING MINUS FORCAST LAGGING

ROUTE	TIME	DELAY: WEIGHTED EXISTING LEADING (VEH-HR)	DELAY: WEIGHTED FORCAST LAGGING (VEH-HR)	DELAY: EXISTING LEADING MINUS FORCAST LAGGING (VEH-HR)	DELAY: $0.616 /(VEH-HR) COST	TRAVEL TIME: WEIGHTED EXISTING LEADING (VEH-HR)	TRAVEL TIME: WEIGHTED FORCAST LAGGING (VEH-HR)	TRAVEL TIME: EXISTING LEADING MINUS FORCAST LAGGING (VEH-HR)	TRAVEL TIME: $5.55 /(VEH-HR) COST	STOPS: WEIGHTED EXISTING LEADING (KVEH-STP)	STOPS: WEIGHTED FORCAST LAGGING (KVEH-STP)	STOPS: EXISTING LEADING MINUS FORCAST LAGGING (KVEH-STP)	STOPS: $41.00 /(KVEH-STP) COST	TOTAL COST
48TH STREET NB	AM	17	3	14	$8.93	117	94	23	$77.70	3.5	3.5	0.0	$0.00	$86.63
48TH STREET NB	MID	28	8	20	$12.50	164	142	22	$72.54	4.9	4.9	0.0	$0.00	$85.04
48TH STREET NB	PM	25	44	-19	($11.62)	96	117	-21	($70.21)	2.5	2.5	0.0	$0.00	($81.83)
48TH STREET SB	AM	5	22	-18	($10.79)	48	66	-18	($58.70)	1.5	2.9	-1.5	($60.15)	($129.64)
48TH STREET SB	MID	49	68	-19	($11.74)	246	250	-3	($11.61)	6.2	12.5	-6.2	($255.80)	($279.15)
48TH STREET SB	PM	44	87	-43	($26.31)	202	239	-37	($125.20)	14.4	14.4	0.0	$0.00	($151.51)
SOUTHERN AVE EB	AM	12	12	-0	($0.18)	55	50	5	$17.20	3.1	2.1	1.0	$42.11	$59.13
SOUTHERN AVE EB	MID	26	18	7	$4.53	209	197	12	$41.03	8.8	4.4	4.4	$180.77	$226.32
SOUTHERN AVE EB	PM	54	21	34	$20.85	241	199	41	$137.69	13.1	4.4	8.7	$356.86	$515.41
SOUTHERN AVE WB	AM	15	42	-27	($16.82)	149	181	-32	($107.26)	3.4	10.2	-6.8	($277.98)	($402.06)
SOUTHERN AVE WB	MID	11	47	-37	($22.73)	187	229	-42	($141.27)	0.0	9.5	-9.5	($389.01)	($553.00)
SOUTHERN AVE WB	PM	23	27	-5	($2.85)	103	104	-1	($1.93)	6.2	2.1	4.2	$170.48	$165.70
BROADWAY RD. EB	AM	6	14	-8	($4.87)	104	109	-5	($15.44)	2.4	2.4	0.0	$0.00	($20.31)
BROADWAY RD. EB	MID	49	167	-118	($72.60)	483	598	-115	($385.18)	20.7	20.7	0.0	$0.00	($457.77)
BROADWAY RD. EB	PM	22	68	-46	($28.16)	271	310	-39	($132.00)	5.7	17.0	-11.3	($465.27)	($625.42)
BROADWAY RD. WB	AM	30	34	-5	($2.83)	202	205	-3	($11.53)	12.4	8.3	4.1	$169.37	$155.01
BROADWAY RD. WB	MID	117	52	66	$40.58	550	458	92	$307.04	20.6	10.3	10.3	$422.75	$770.37
BROADWAY RD. WB	PM	100	137	-37	($22.81)	283	301	-18	($60.15)	12.1	12.1	0.0	$0.00	($82.96)
PRIEST RD. NB	AM	9	11	-2	($1.42)	79	83	-4	($12.84)	5.5	2.8	2.8	$113.12	$98.87
PRIEST RD. NB	MID	17	3	14	$8.58	161	150	11	$36.31	5.6	5.6	0.0	$0.00	$44.89
PRIEST RD. NB	PM	8	16	-9	($5.50)	74	82	-8	($27.62)	2.5	2.5	0.0	$0.00	($33.12)
PRIEST RD. SB	AM	1	22	-20	($12.46)	34	56	-22	($73.79)	1.3	1.3	0.0	$0.00	($86.25)
PRIEST RD. SB	MID	1	1	0	$0.00	120	124	-4	($13.86)	0.0	0.0	0.0	$0.00	($13.86)
PRIEST RD. SB	PM	19	6	13	$8.28	113	99	14	$48.23	3.5	3.5	0.0	$0.00	$56.50
SAMPLE SIZE					24				24				24	24
MEAN DIFFERENCE					($6.23)				($21.29)				$0.30	($27.21)
STD DEVIATION					$21.10				$123.58				$200.56	$308.28
TEST STAT					-1.445				-0.844				0.007	-0.432
SIGNIFICANT?					N				N				N	N

TABLE F-9

TEMPE TRAVEL TIME STUDIES
EXISTING LEADING MINUS FORCAST COMBINATION

ROUTE	TIME	DELAY: WEIGHTED EXISTING LEADING (VEH-HR)	DELAY: WEIGHTED FORCAST COMBINATION (VEH-HR)	DELAY: EXISTING LEADING MINUS FORCAST COMBINATION (VEH-HR)	DELAY: $0.616 /(VEH-HR) COST	TRAVEL TIME: WEIGHTED EXISTING LEADING (VEH-HR)	TRAVEL TIME: WEIGHTED FORCAST COMBINATION (VEH-HR)	TRAVEL TIME: EXISTING LEADING MINUS FORCAST COMBINATION (VEH-HR)	TRAVEL TIME: $3.35 /(VEH-HR) COST	STOPS: WEIGHTED EXISTING LEADING (KVEH-STP)	STOPS: WEIGHTED FORCAST COMBINATIO (KVEH-STP)	STOPS: EXISTING LEADING MINUS FORCAST COMBINATION (KVEH-STP)	STOPS: $41.00 /(KVEH-STP) COST	TOTAL COST
48TH ST. NB	AM	17	15	2	$1.19	117	117	0	$0.00	3.5	7.0	−3.5	($142.64)	($141.45)
48TH ST. NB	MD	28	38	−9	($5.84)	164	176	−12	($40.80)	4.9	4.9	0.0	$0.00	($46.64)
48TH ST. NB	PM	25	42	−17	($10.33)	96	115	−19	($63.19)	2.5	2.5	0.0	$0.00	($73.52)
48TH ST. SB	AM	5	14	−9	($5.52)	48	58	−10	($34.13)	1.5	4.4	−2.9	($120.29)	($159.94)
48TH ST. SB	MD	49	29	19	$11.74	246	203	43	$145.14	6.2	6.2	0.0	$0.00	$156.89
48TH ST. SB	PM	44	48	−4	($2.47)	202	184	17	$58.13	14.4	4.8	9.6	$394.01	$449.67
SOUTHERN AVE EB	AM	12	9	3	$1.93	55	47	9	$28.67	3.1	2.1	1.0	$42.11	$72.71
SOUTHERN AVE EB	MD	26	6	20	$12.07	209	168	42	$139.50	8.8	4.4	4.4	$180.77	$332.34
SOUTHERN AVE EB	PM	54	5	50	$30.53	241	167	74	$247.04	13.1	4.4	8.7	$356.86	$634.43
SOUTHERN AVE WB	AM	15	19	−4	($2.32)	149	151	−2	($6.31)	3.4	3.4	0.0	$0.00	($8.63)
SOUTHERN AVE WB	MD	11	33	−22	($13.80)	187	207	−20	($66.22)	0.0	4.7	−4.7	($194.50)	($274.52)
SOUTHERN AVE WB	PM	23	13	10	$6.05	103	89	14	$48.37	6.2	4.2	2.1	$85.24	$139.65
BROADWAY RD EB	AM	6	23	−17	($10.55)	104	119	−15	($50.75)	2.4	7.1	−4.7	($194.42)	($255.72)
BROADWAY RD EB	MD	49	49	0	$0.00	483	466	17	$57.78	20.7	10.3	10.3	$424.27	$482.04
BROADWAY RD EB	PM	22	120	−98	($60.19)	271	359	−88	($295.68)	5.7	17.0	−11.3	($465.27)	($821.14)
BROADWAY RD WB	AM	30	38	−8	($4.95)	202	204	−2	($7.69)	12.4	8.3	4.1	$169.37	$156.73
BROADWAY RD WB	MD	117	57	60	$37.05	550	467	83	$278.25	20.6	10.3	10.3	$422.75	$738.06
BROADWAY RD WB	PM	100	79	21	$13.13	283	236	47	$157.90	12.1	8.1	4.0	$165.64	$336.67
PREST RD. NB	AM	9	14	−5	($2.83)	79	85	−6	($20.54)	5.5	2.8	2.8	$113.12	$89.75
PREST RD. NB	MD	17	31	−14	($8.58)	161	170	−9	($31.12)	5.6	11.1	−5.6	($228.53)	($268.24)
PREST RD. NB	PM	8	25	−17	($10.58)	74	88	−14	($48.33)	2.5	2.5	0.0	$0.00	($58.91)
PREST RD. SB	AM	1	21	−20	($12.23)	34	53	−20	($66.53)	1.3	1.3	0.0	$0.00	($78.77)
PREST RD. SB	MD	1	1	0	$0.00	120	116	4	$13.86	0.0	0.0	0.0	$0.00	$13.86
PREST RD. SB	PM	19	16	3	$1.77	113	107	6	$19.29	3.5	3.5	0.0	$0.00	$21.06
SAMPLE SIZE					24				24				24	24
MEAN DIFFERENCE					($1.45)				$19.28				$42.02	$59.85
STD DEVIATION					$17.73				$117.01				$217.57	$333.11
TEST STAT					−0.400				0.807				0.946	0.880
SIGNIFICANT?					N				N				N	N

TABLE F-10

TEMPE TRAVEL TIME STUDIES
FORCAST LEADING MINUS FORCAST LAGGING

ROUTE	TIME	DELAY: WEIGHTED FORCAST LEADING (VEH-HR)	DELAY: WEIGHTED FORCAST LAGGING (VEH-HR)	DELAY: FORCAST LEADING MINUS FORCAST LAGGING (VEH-HR)	DELAY: $0.616 /(VEH-HR) COST	TRAVEL TIME: WEIGHTED FORCAST LEADING (VEH-HR)	TRAVEL TIME: WEIGHTED FORCAST LAGGING (VEH-HR)	TRAVEL TIME: FORCAST LEADING MINUS FORCAST LAGGING (VEH-HR)	TRAVEL TIME: $3.35 /(VEH-HR) COST	STOPS: WEIGHTED FORCAST LEADING (KVEH-STP)	STOPS: WEIGHTED FORCAST LAGGING (KVEH-STP)	STOPS: FORCAST LEADING MINUS FORCAST LAGGING (KVEH-STP)	STOPS: $41.00 /(KVEH-STP) COST	TOTAL COST
48TH ST. NB	AM	4	3	1	$0.60	101	94	7	$22.66	3.5	3.5	0.0	$0.00	$23.26
48TH ST. NB	MD	4	8	-4	($2.50)	141	142	-1	($4.53)	4.9	4.9	0.0	$0.00	($7.03)
48TH ST. NB	PM	40	44	-4	($2.58)	112	117	-5	($16.38)	2.5	2.5	0.0	$0.00	($18.96)
48TH ST. SB	AM	36	22	14	$8.53	79	66	14	$46.41	2.9	2.9	0.0	$0.00	$54.95
48TH ST. SB	MD	14	68	-54	($33.09)	191	250	-59	($197.40)	0.0	12.5	-12.5	($511.60)	($742.09)
48TH ST. SB	PM	11	87	-76	($46.86)	157	239	-81	($272.75)	4.8	14.4	-9.6	($394.01)	($713.63)
SOUTHERN AVE EB	AM	16	12	3	$2.11	54	50	4	$14.34	2.1	2.1	0.0	$0.00	$16.44
SOUTHERN AVE EB	MD													
SOUTHERN AVE EB	PM	63	21	42	$26.06	237	199	37	$125.54	8.7	4.4	4.4	$178.43	$330.04
SOUTHERN AVE WB	AM	58	42	16	$9.86	198	181	17	$56.78	10.2	10.2	0.0	$0.00	$66.64
SOUTHERN AVE WB	MD													
SOUTHERN AVE WB	PM	17	27	-10	($6.05)	96	104	-8	($25.15)	2.1	2.1	0.0	$0.00	($31.20)
BROADWAY RD EB	AM	15	14	1	$0.81	110	109	1	$4.41	2.4	2.4	0.0	$0.00	$5.22
BROADWAY RD EB	MD	106	167	-60	($37.18)	540	598	-57	($192.59)	10.3	20.7	-10.3	($424.27)	($654.04)
BROADWAY RD EB	PM	117	68	49	$30.10	358	310	47	$158.40	17.0	17.0	0.0	$0.00	$188.50
BROADWAY RD WB	AM	52	34	17	$10.60	208	205	2	$7.69	4.1	8.3	-4.1	($169.37)	($151.08)
BROADWAY RD WB	MD	72	52	20	$12.35	481	458	23	$76.76	10.3	10.3	0.0	$0.00	$89.11
BROADWAY RD WB	PM	43	137	-94	($58.07)	201	301	-100	($334.59)	4.0	12.1	-8.1	($331.28)	($723.94)
PRIEST RD. NB	AM	31	11	19	$11.80	98	83	15	$51.35	2.8	2.8	0.0	$0.00	$63.15
PRIEST RD. NB	MD	6	3	3	$1.91	149	150	-2	($5.19)	5.6	5.6	0.0	$0.00	($3.28)
PRIEST RD. NB	PM	21	16	5	$2.96	84	82	3	$9.21	2.5	2.5	0.0	$0.00	$12.17
PRIEST RD. SB	AM	4	22	-18	($11.12)	35	56	-21	($68.95)	0.0	1.3	-1.3	($53.30)	($133.38)
PRIEST RD. SB	MD	1	1	0	$0.00	130	124	6	$18.48	0.0	0.0	0.0	$0.00	$18.48
PRIEST RD. SB	PM	11	6	5	$2.96	99	99	0	$0.00	3.5	3.5	0.0	$0.00	$2.96
SAMPLE SIZE					22				22				22	22
MEAN DIFFERENCE					($3.49)				($23.89)				($77.52)	($104.90)
STD DEVIATION					$21.97				$121.80				$174.29	$306.82
TEST STAT					-0.745				-0.920				-2.086	-1.604
SIGNFICANT?					N				N				Y	N

Y
2.064

TABLE F-11

TEMPE TRAVEL TIME STUDIES

FORÇAST LEADING MINUS FORCAST COMBINATION

ROUTE	TIME	DELAY: WEIGHTED FORCAST LEADING (VEH-HRS)	DELAY: WEIGHTED FORCAST COMBINATIO (VEH-HRS)	DELAY: FORCAST LEADING MINUS FORCAST COMBINATIO (VEH-HRS)	TRAVEL TIME: WEIGHTED FORCAST LEADING (VEH-HRS)	TRAVEL TIME: WEIGHTED FORCAST COMBINATIO (VEH-HRS)	TRAVEL TIME: FORCAST LEADING MINUS FORCAST COMBINATIO (VEH-HRS)	STOPS: WEIGHTED FORCAST LEADING (KVEH-STPS)	STOPS: WEIGHTED FORCAST COMBINATION (KVEH-STPS)	STOPS: FORCAST LEADING MINUS FORCAST COMBINATION (KVEH-STPS)
48TH STREET NB	AM	4	15	-12	101	117	-16	3	7	-3.5
48TH STREET NB	MID	4	38	-34	141	176	-35	5	5	0.0
48TH STREET NB	PM	40	42	-2	112	115	-3	3	3	0.0
48TH STREET SB	AM	36	14	22	79	58	21	3	4	-1.5
48TH STREET SB	MID	14	29	-16	191	203	-12	0	6	-6.2
48TH STREET SB	PM	11	48	-37	157	184	-27	5	5	0.0
SOUTHERN AVE EB	AM	16	9	7	54	47	8	2	2	0.0
SOUTHERN AVE EB	MID									
SOUTHERN AVE EB	PM	63	5	58	237	167	70	9	4	4.4
SOUTHERN AVE WB	AM	58	19	40	198	151	47	10	3	6.8
SOUTHERN AVE WB	MID									
SOUTHERN AVE WB	PM	17	13	5	96	89	8	2	4	-2.1
BROADWAY RD. EB	AM	15	23	-8	110	119	-9	2	7	-4.7
BROADWAY RD. EB	MID	106	49	57	540	466	75	10	10	0.0
BROADWAY RD. EB	PM	117	120	-3	358	359	-2	17	17	0.0
BROADWAY RD. WB	AM	52	38	14	208	204	3	4	8	-4.1
BROADWAY RD. WB	MID	72	57	14	481	467	14	10	10	0.0
BROADWAY RD. WB	PM	43	79	-36	201	236	-35	4	8	-4.0
PRIEST RD. NB	AM	31	14	17	98	85	13	3	3	0.0
PRIEST RD. NB	MID	6	31	-25	149	170	-22	6	11	-5.6
PRIEST RD. NB	PM	21	25	-3	84	88	-3	2	2	0.0
PRIEST RD. SB	AM	4	21	-18	35	53	-18	0	1	-1.3
PRIEST RD. SB	MID	1	1	0	130	116	14	0	0	0.0
PRIEST RD. SB	PM	11	16	-6	99	107	-9	3	3	0.0
		SAMPLE SIZE		22	SAMPLE SIZE		22	SAMPLE SIZE		22
		MEAN DIFFERENCE		1.581	MEAN DIFFERENCE		3.724	MEAN DIFFERENCE		-0.996
		STD DEVIATION		26.447	STD DEVIATION		29.377	STD DEVIATION		3.002
		TEST STAT		0.280	TEST STAT		0.595	TEST STAT		-1.557
		SIGNIFICANT?		N	SIGNIFICANT?		N	SIGNIFICANT?		N

TABLE F-12

TEMPE TRAVEL TIME STUDIES
FORCAST LAGGING MINUS FORCAST COMBINATION

		DELAY			TRAVEL TIME			STOPS		
ROUTE	TIME	WEIGHTED FORCAST LAGGING (VEH-HRS)	WEIGHTED FORCAST COMBINATIO (VEH-HRS)	FORCAST LAGGING MINUS FORCAST COMBINATIO (VEH-HRS)	WEIGHTED FORCAST LAGGING (VEH-HRS)	WEIGHTED FORCAST COMBINATIO (VEH-HRS)	FORCAST LAGGING MINUS FORCAST COMBINATIO (VEH-HRS)	WEIGHTED FORCAST LAGGING (KVEH-STPS)	WEIGHTED FORCAST COMBINATION (KVEH-STPS)	FORCAST LAGGING MINUS FORCAST COMBINATION (KVEH-STPS)
48TH STREET NB	AM	3	15	-13	94	117	-23	3	7	-3.5
48TH STREET NB	MID	8	38	-30	142	176	-34	5	5	0.0
48TH STREET NB	PM	44	42	2	117	115	2	3	3	0.0
48TH STREET SB	AM	22	14	9	66	58	7	3	4	-1.5
48TH STREET SB	MID	68	29	38	250	203	47	12	6	6.2
48TH STREET SB	PM	87	48	39	239	184	55	14	5	9.6
SOUTHERN AVE EB	AM	12	9	3	50	47	3	2	2	0.0
SOUTHERN AVE EB	MID	18	6	12	197	168	29	4	4	0.0
SOUTHERN AVE EB	PM	21	5	16	199	167	33	4	4	0.0
SOUTHERN AVE WB	AM	42	19	24	181	151	30	10	3	6.8
SOUTHERN AVE WB	MID	47	33	14	229	207	22	9	5	4.7
SOUTHERN AVE WB	PM	27	13	14	104	89	15	2	4	-2.1
BROADWAY RD. EB	AM	14	23	-9	109	119	-11	2	7	-4.7
BROADWAY RD. EB	MID	167	49	118	598	466	132	21	10	10.3
BROADWAY RD. EB	PM	68	120	-52	310	359	-49	17	17	0.0
BROADWAY RD. WB	AM	34	38	-3	205	204	1	8	8	0.0
BROADWAY RD. WB	MID	52	57	-6	458	467	-9	10	10	0.0
BROADWAY RD. WB	PM	137	79	58	301	236	65	12	8	4.0
PRIEST RD. NB	AM	11	14	-2	83	85	-2	3	3	0.0
PRIEST RD. NB	MID	3	31	-28	150	170	-20	6	11	-5.6
PRIEST RD. NB	PM	16	25	-8	82	88	-6	2	2	0.0
PRIEST RD. SB	AM	22	21	0	56	53	2	1	1	0.0
PRIEST RD. SB	MID	1	1	0	124	116	8	0	0	0.0
PRIEST RD. SB	PM	6	16	-11	99	107	-9	3	3	0.0
		SAMPLE SIZE		24	SAMPLE SIZE		24	SAMPLE SIZE		24
		MEAN DIFFERENCE		7.759	MEAN DIFFERENCE		12.108	MEAN DIFFERENCE		1.018
		STD DEVIATION		33.036	STD DEVIATION		37.226	STD DEVIATION		4.013
		TEST STAT		1.151	TEST STAT		1.593	TEST STAT		1.242
		SIGNIFICANT?		N	SIGNIFICANT?		N	SIGNIFICANT?		N

TABLE F-13
(LEAD - LAG) X VOLUME

			Delay	Stops	Travel Time
68TH ST		AM	57800	3400	45900
		MD	296000	16000	192000
	NB	PM	-53200	0	-67200
		AM	-23400	3600	93600
	SB	MD	168000	16000	120000
		PM	-15000	3000	9000
SCOTTSDALE RD.		AM	69300	4200	79800
		MD	N/A	N/A	N/A
	NB	PM	235600	15200	277400
		AM	180000	12000	192000
	SB	MD	N/A	N/A	N/A
		PM	36000	8000	-252000
HAYDEN RD.		AM	228250	11000	214500
		MD	672000	42000	1176000
	NB	PM	172000	12000	144000
		AM	193200	2800	-11200
	SB	MD	-1201500	13500	-958500
		PM	249000	12000	78000
CAMELBACK RD.		AM	92400	1200	91200
		MD	209000	11000	198000
	EB	PM	195200	3200	169600
		AM	119000	3400	175100
	WB	MD	-20000	0	-40000
		PM	-35000	0	0
INDIAN SCHOOL RD.		AM	60000	3000	49500
		MD	1389200	46000	1499600
	EB	PM	429000	6600	462000
		AM	24000	0	10000
	WB	MD	170000	25500	187000
		PM	134400	9600	148800
THOMAS RD.		AM	11200	3200	-6400
		MD	-270000	-10000	10000
	EB	PM	264000	4000	172000
		AM	10000	0	-2000
	WB	MD	360000	10000	510000
		PM	33600	2400	28800
MCDOWELL RD.		AM	12600	3600	-7200
		MD	-337500	-12500	12500
	EB	PM	280500	4250	182750
		AM	14000	0	-2800
	WB	MD	414000	11500	586500
		PM	44100	3150	37800
SAMPLE SIZE=			40	40	40
MEAN DIFF. =			115898.809524	7233.33333	138239.286
STD. DEV. =			336401.90403	10802.5046	353469.859
VARIANCE =			113166241035	116694106	1.2494E+11
TEST STAT. =			2.17896636022	4.23490834	2.47348391
Significant ?			Y	Y	Y

TABLE F-14

SCOTTSDALE TRAVEL TIME STUDIES
LEADING LEFT TURNS MINUS LAGGING LEFT TURNS

		DELAY			TRAVEL TIME			STOPS		
ROUTE	TIME	WEIGHTED LEADING (VEH-HRS)	WEIGHTED LAGGING (VEH-HRS)	LEADING MINUS LAGGING (VEH-HRS)	WEIGHTED LEADING (VEH-HRS)	WEIGHTED LAGGING (VEH-HRS)	LEADING MINUS LAGGING (VEH-HRS)	WEIGHTED LEADING (KVEH-STPS)	WEIGHTED LAGGING (KVEH-STPS)	LEADING MINUS LAGGING (KVEH-STPS)
68TH ST NB	AM	27	14	13	161	151	9	5.1	3.4	1.7
68TH ST NB	MID									
68TH ST NB	PM	52	123	-71	294	353	-59	11.2	11.2	0.0
68TH ST SB	AM	53	22	31	195	155	41	7.2	1.8	5.4
68TH ST SB	MID									
68TH ST SB	PM	81	103	-23	312	338	-26	6.0	12.0	-6.0
SCOTTSDALE RD NB	AM	50	40	11	303	361	-58	10.5	8.4	2.1
SCOTTSDALE RD NB	MID									
SCOTTSDALE RD NB	PM	179	97	82	765	676	90	26.6	19.0	7.6
SCOTTSDALE RD SB	AM	121	71	50	552	498	53	21.0	9.0	12.0
SCOTTSDALE RD SB	MID									
SCOTTSDALE RD SB	PM	259	246	13	873	891	-18	28.0	32.0	-4.0
HAYDEN RD NB	AM	83	53	29	378	380	-2	13.8	11.0	2.8
HAYDEN RD NB	MID	191	4	187	1672	1342	331	42.0	0.0	42.0
HAYDEN RD NB	PM	92	0	92	523	449	74	12.0	0.0	12.0
HAYDEN RD SB	AM	110	23	86	404	325	79	14.0	5.6	8.4
HAYDEN RD SB	MID	323	446	-124	1778	2055	-278	67.5	54.0	13.5
HAYDEN RD SB	PM	111	48	63	432	448	-17	27.0	15.0	12.0
CAMELBACK RD EB	AM	33	6	27	97	72	25	3.6	2.4	1.2
CAMELBACK RD EB	MID	382	235	147	1042	895	147	44.0	33.0	11.0
CAMELBACK RD EB	PM	85	27	59	273	233	40	6.4	9.6	-3.2
CAMELBACK RD WB	AM	70	17	52	177	107	70	6.8	3.4	3.4
CAMELBACK RD WB	MID	119	161	-42	719	739	-19	30.0	40.0	-10.0
CAMELBACK RD WB	PM	27	24	3	183	170	13	7.5	12.5	-5.0
INDIAN SCHOOL RD EB	AM	21	2	19	108	93	15	4.5	1.5	3.0
INDIAN SCHOOL RD EB	MID	399	13	386	971	555	417	46.0	0.0	46.0
INDIAN SCHOOL RD EB	PM	147	39	107	358	248	110	13.2	13.2	0.0
INDIAN SCHOOL RD WB	AM	14	2	12	132	123	8	2.0	2.0	0.0
INDIAN SCHOOL RD WB	MID	76	28	47	619	567	52	34.0	8.5	25.5
INDIAN SCHOOL RD WB	PM	36	10	26	198	161	37	14.4	2.4	12.0
THOMAS RD EB	AM	20	17	3	129	131	-2	6.4	3.2	3.2
THOMAS RD EB	MID	47	136	-89	706	858	-153	10.0	40.0	-30.0
THOMAS RD EB	PM	120	37	83	396	279	117	16.0	8.0	8.0
THOMAS RD WB	AM	11	4	6	148	145	3	2.0	2.0	0.0
THOMAS RD WB	MID	83	19	64	778	681	97	10.0	10.0	0.0
THOMAS RD WB	PM	39	29	9	207	202	5	7.2	7.2	0.0
MCDOWELL RD EB	AM	53	30	23	186	168	18	7.2	10.8	-3.6
MCDOWELL RD EB	MID	149	219	-69	1052	1170	-118	37.5	37.5	0.0
MCDOWELL RD EB	PM	45	129	-84	348	471	-123	8.5	25.5	-17.0
MCDOWELL RD WB	AM	117	26	91	345	244	100	16.8	8.4	8.4
MCDOWELL RD WB	MID	438	281	157	1319	1118	201	46.0	69.0	-23.0
MCDOWELL RD WB	PM	116	93	23	344	335	9	18.9	18.9	0.0
		SAMPLE SIZE		38	SAMPLE SIZE		38	SAMPLE SIZE		38
		MEAN DIFFERENCE		39.50	MEAN DIFFERENCE		33.94	MEAN DIFFERENCE		3.67
		STD DEVIATION		88.01	STD DEVIATION		118.62	STD DEVIATION		13.91
		TEST STAT		2.77	TEST STAT		1.76	TEST STAT		1.63
		SIGNIFICANT?		Y	SIGNIFICANT?		N	SIGNIFICANT?		N

APPENDIX F-3

SAMPLE TRAVEL TIME OUTPUT

Lee Engineering
SPEED AND DELAY STUDY
ROUTE SUMMARY
Route #8, AM Peak

ROUTE # 8 RUNS: 6 DIRECTION: WESTBOUND START: 05:00 END: 09:00
STREET:Southern Ave. FROM:Hardy Dr. TO:48th St.

NAME OF LINK	DISTANCE	TRAVEL TIME H:MM:SS	AVERAGE SPEED (MPH)	CRUISE SPEED (MPH)	DELAY TIME MM:SS	TOTAL DELAYS
Priest Dr.	2664	0:01:30	20	43	00:39	1
Potter Rd.	3488	0:01:04	37	48	00:06	1
48th St.	1840	0:00:38	33	41	00:00	0
TOTALS	7992	0:03:12	28	46	0:00:45	3

FUEL CONSUMPTION AND EMISSIONS SUMMARY
(PER 1000 VEHICLES)

NAME OF LINK	FUEL CONSUMPTION (GAL.)	CARBON MONOXIDE (KGS)	HYDRO CARBONS (KGS)	NITROUS OXIDES (KGS)
Priest Dr.	36.499	28.03	3.03	1.84
Potter Rd.	38.659	22.40	2.74	2.74
48th St.	21.088	12.91	1.54	1.38
TOTALS	96.246	63.34	7.31	5.97

APPENDIX F-4

SCOTTSDALE TRAVEL TIME DATA

SCOTTSDALE TRAVEL TIME STUDY RESULTS

In the spring of 1988, the signals within the City of Scottsdale was optimized. Optimization was performed using the city's FORCAST computer program. At that time, several travel time runs were performed using the TIMELAPSE travelog data collector to determine stops, delay, and travel time. These travel time runs were performed by members of the City Council and city staff.

Once the city converted all their signals to lagging phasing, city staff performed another retiming of their signal system in the Spring of 1990. Assuming the volumes along the streets stayed relatively constant over this two year period, another set of travel time runs were performed by city staff.

A comparison of the results of these two signal timing efforts is shown in TABLE F-14.

The delay column shows the difference in vehicle-seconds of delay for the route, the stops column shows vehicle-stops for the route, and the travel time column shows the vehicle-seconds of travel time for the route. At the bottom of each column are the statistics for the paired comparison test. Using a sample size of 40, the results are significant if the test statistic is greater than 2.021 or less than -2.021.

There were several conditions which changed between the before and after periods:

- o Before period travel time runs were 4 to 6 PM; after prior runs were 4 to 5 PM.
- o 2 year time period between studies.
- o City conversion of numerous intersections from protected to permissive - protected between the two studies.
- o Reduction of extension time throughout the system.
- o Refinement of leading timing patterns after before studies.
- o More recent traffic counts in timing lagging condition.
- o Different drivers in before and after studies.

In the interest of making the studies more compatible a comparison was made of PM peak runs only during the 4 to 5 PM period. This data is shown in TABLE F-13 and FIGURE F-1.

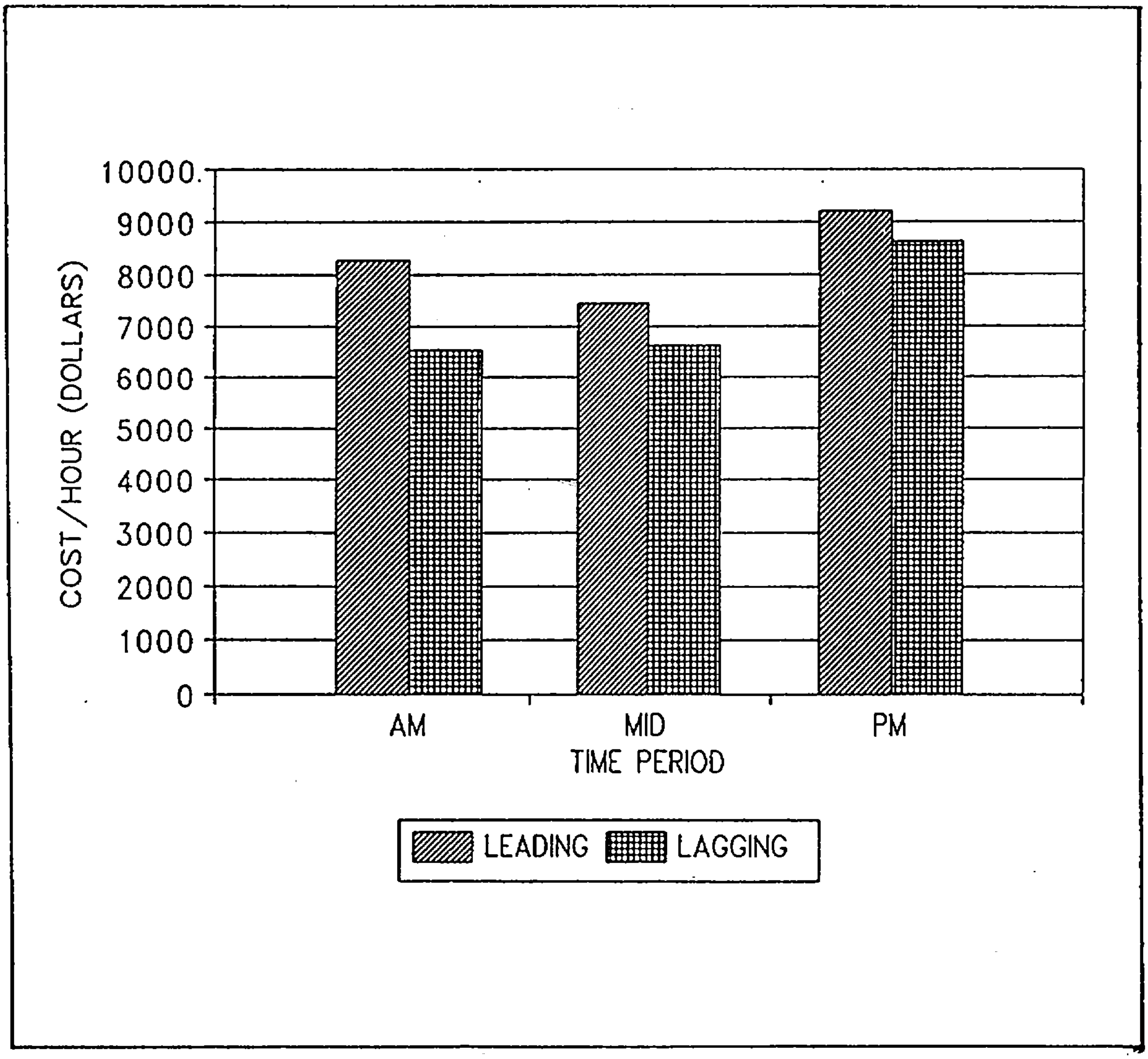


Figure F-1. Scottsdale Travel Time Study Cost/Hour

APPENDIX G

DALLAS AREA SPECIAL PHASING SEQUENCE

TYPICAL PHASING

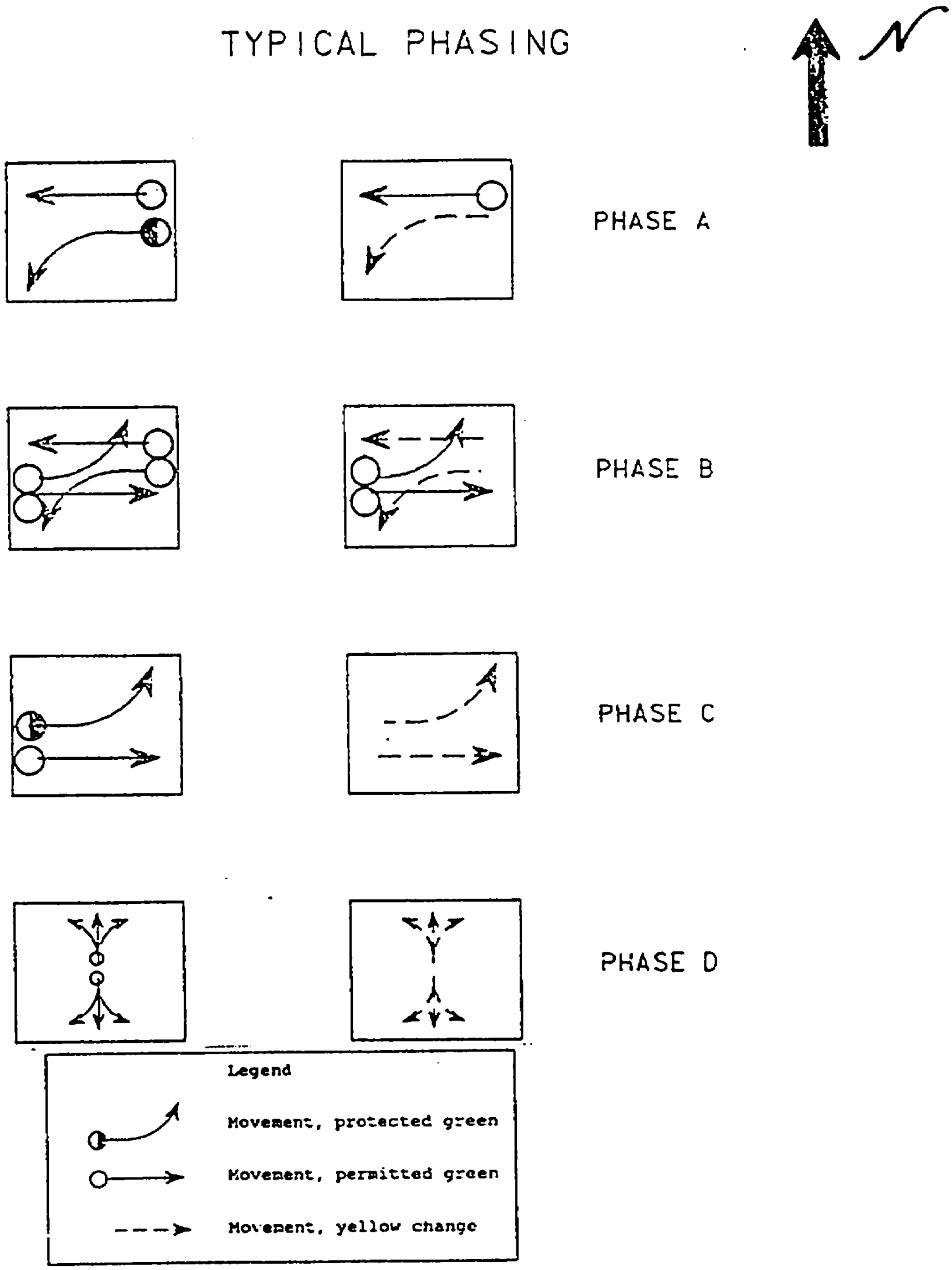


Figure 1. Typical Phasing.

Source: Reference 14

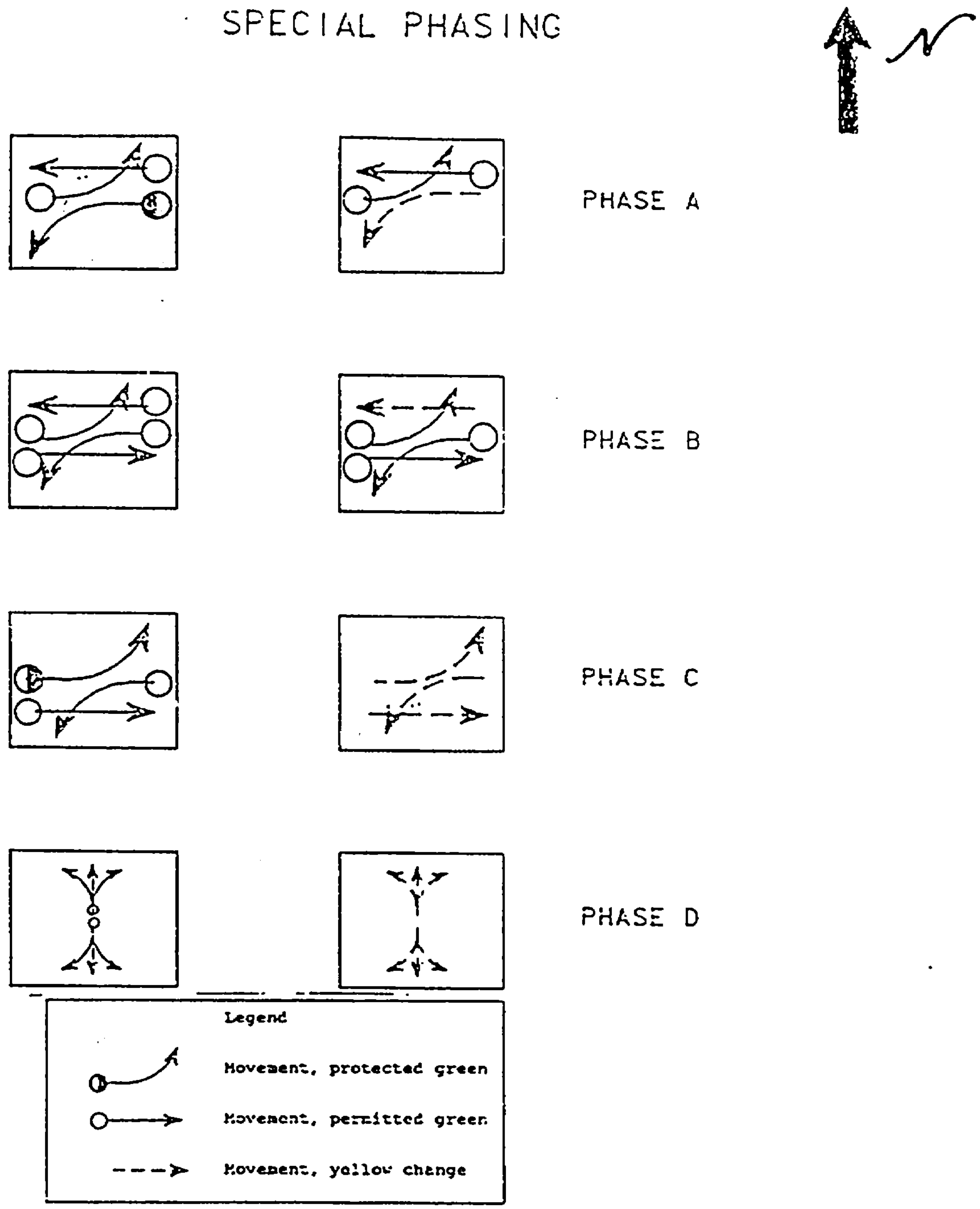


Figure 2. Special Phasing.

Source: Reference 14