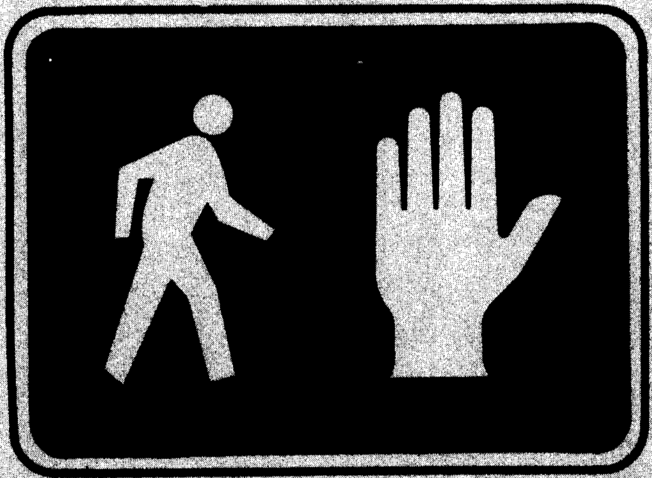
Report No. FHWA-RD-77-144

# URBAN INTERSECTION IMPROVEMENTS FOR PEDESTRIAN SAFETY

## Vol. III. Signal Timing for the Pedestrian

DEPARTMENT OF TRANSPORTATION
UNITED STATES OF AMERICA



**December 1977**

**Final Report**

**Prepared for**

**FEDERAL HIGHWAY ADMINISTRATION**
**Offices of Research & Development**
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# FOREWORD

This five-volume report describes pedestrian problems at urban intersections and timing and display improvements for pedestrian signals. This report will be of interest to traffic engineers and others responsible for pedestrian safety.

The five volumes are:

Vol. I - Executive Summary
Vol. II - Identification of Safety and Operational Problems at Intersections
Vol. III - Signal Timing for the Pedestrian
Vol. IV - Pedestrian Signal Displays and Operation
Vol. V - Evaluation of Alternatives to Full Signalization at Pedestrian Crossings

Sufficient copies of the five volumes are being distributed to provide a minimum of one copy to each FHWA Regional and Division office. Additional copies of the Executive Summary have also been provided to allow wider distribution of this report. Copies sent direct to the Division Offices should be distributed to the State highway agency, Governor's Representative for Highway Safety, and to major metropolitan areas.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

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**16. Abstract**

This report presents the final results of the signal timing element of the FHWA research project "Urban Intersection Improvements for Pedestrian Safety." The report discusses the research which was conducted, and recommends improvements in the timing of pedestrian traffic signals both to maximize safety and to minimize delay.

Specifically, the elements covered in this volume include the following:

Chapter II - Timing for a Combined Pedestrian - Vehicular Interval. This includes discussions of minimum WALK time, minimum clearance interval, and allocation of excess pedestrian time.

Chapter III - Alternative Phasing Schemes. This includes an analysis of early and late release of pedestrians with respect to vehicles, scramble pedestrian timing, and signal phasing for the partial crossing of wide, channelized streets.

Chapter IV - Other Areas of Pedestrian Signal Research, including time-of-day adjustments of pedestrian signal timing, application of correction factors to the Highway Capacity Manual based on vehicle and pedestrian activity levels, and general observations on pedestrian flow characteristics.

Other volumes contained in this series include:
Vol. I - Executive Summary
Vol. II - Identification of Safety and Operational Problems at Intersections
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# PREFACE

This research project was conducted in three phases. Phase I dealt with the investigation and identification of both operational and safety problems encountered by pedestrians and motorists at urban-type intersections. Phase II dealt with the development, evaluation, and design criteria formulation of countermeasures that address the problems identified in Phase I. Phase III evaluated some alternatives to full signalization at intersections requiring pedestrian protection.

Volume I of the Final Report is the Executive Summary of the project. Phase I is reported in Volume II and Phase II is reported in Volumes III and IV. Specifically, Volume III addresses the subject of signal timing for the pedestrian; and Volume IV deals with pedestrian signal displays and signal operation. Phase III is reported in Volume V.

The authors wish to express their thanks to the agencies and individuals who contributed to the conduct of this research. Particular gratitude is expressed to the traffic engineering departments of the cities of Washington, D. C.; Phoenix, Arizona; Tempe, Arizona; Buffalo, New York; and Sioux City, Iowa for permitting signal timing to be changed in the performance of the pedestrian compliance studies.

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# SUMMARY

The primary purpose of this research was to examine the timing of pedestrian WALK/DONT WALK signals from the perspective of safety and delay for both pedestrians and vehicles and to develop procedures which would make signal timing more responsive to the needs of both groups. Three major areas of research were examined, each of which contain three additional subcategories. These are listed below in the order in which they occur in this report.

- Chapter II - Timing for a Combined Pedestrian - Vehicular Interval
    1. Minimum WALK time.
    2. Minimum clearance interval.
    3. Allocation of excess pedestrian time.
- Chapter III - Alternative Phasing Schemes
    1. Early and late release of pedestrians with respect to vehicles.
    2. Scramble pedestrian timing.
    3. Signal phasing for the partial crossing of wide, channelized streets.
- Chapter IV - Other Areas of Pedestrian Signal Research
    1. Time of day adjustments of pedestrian signal timing.
    2. Application of correction factors to the Highway Capacity Manual based on vehicle and pedestrian activity levels.
    3. General observations on pedestrian flow characteristics.

A summary of the approach taken and conclusions derived for each of the above areas of research is presented below. Each conclusion is referenced to a location in the body of the report so that backup information can easily be found.

## TIMING FOR A COMBINED PEDESTRIAN-VEHICULAR INTERVAL

A combined pedestrian-vehicular interval is the type of timing which is used at the vast majority of intersections in the United States. The Manual on Uniform Traffic Control Devices serves to set the standard upon which the WALK and clearance intervals are based. It states that the WALK interval should be timed for at least 7 seconds and that the clearance interval should be timed using an assumed walking speed of 4.0 ft./sec. over the distance from the curb to the middle of the farthest traveled lane. An important reason for this research was to determine whether these guidelines are valid for setting the minimum pedestrian intervals. In general, the guidelines were found to be valid except under very heavy pedestrian volumes, as described in detail below and in Chapter II.

The approach to the timing of the minimum WALK and clearance intervals consisted primarily of developing relationships between pedestrian volume and interval requirements. Field studies were performed in Washington, D.C., Phoenix and Tempe, Arizona, and Buffalo, New York in the performance of this task. Studies on the pedestrian use of parking lanes to reduce the effective crosswalk length were also performed. Where excess pedestrian time is available, that is, where vehicular time requirements exceed those for pedestrians, the best allocation of that time must be determined. The approach to this element involved changing pedestrian signal timing at six crosswalks in the cities mentioned above and observing pedestrian compliance and behavior under several alternatives of excess time allocation. In addition, the impact of the allocation of excess pedestrian time on vehicle and pedestrian delay was determined. Time lapse photography was used in providing the required data base for the analysis of vehicle delay. Conclusions derived for each of the three areas are presented below.

## Minimum WALK Time

- A queue of more than approximately 24 persons must be present before the minimum WALK of 7 seconds will be insufficient to accommodate the queue's discharge from the curb (see page 22).
- An average queue size of approximately 13 persons, measured over an hour, must be present before the 24-person limit will be exceeded more than 5 percent of the time (see page 24).
- Average queue size can be estimated from a knowledge of hourly pedestrian volumes* and signal timing (see page 24).
- A peak hour field study can be performed to determine exact WALK interval requirements where the need for a longer minimum WALK interval is suspected (see page 26).

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*All pedestrian volumes are 2-way by crosswalk unless noted otherwise.

- Queues requiring more than 7 seconds to discharge occur very rarely and will usually be found only in certain sections of large metropolitan areas. It is estimated that 99.7 percent of signalized intersections in the United States do not require minimum WALK intervals longer than 7 seconds (see page 28).

- The minimum WALK interval under low volume conditions (less than 10 pedestrians per cycle) could possibly be lowered to 4 - 5 seconds but the importance of the inattentiveness factor should also be weighted in this decision. In addition, lowering the interval will only be practical where it is desired to reduce the length of the phase and thereby help to alleviate traffic congestion on the opposing phase (see page 30).

## Minimum Clearance Interval

- The percentage of pedestrian platoons walking slower than 4.0 ft./sec. is quite high on high volume crosswalks, and ranged well over 50 percent for some of the crosswalks observed in this study (see page 38).

- A clearance interval based on a pedestrian walking speed of 3.5 ft./sec. should be considered at locations with peak hour pedestrian volumes of over 15 per cycle (see page 50).

- Neither platoon volume nor opposing pedestrian flow appear to have a significant effect on individual pedestrian walking speeds at intersections where crosswalk volumes are less than approximately 30 pedestrians per cycle (see page 45).

- Pedestrians tend not to use the nearside parking lane as a protected area for beginning their crossing (see page 47).

- Far-side parking lanes should be considered as "traveled lanes" unless geometrics or operational constraints preclude pedestrian/ vehicle conflicts in that lane (see page 47).

## Allocation of Excess Pedestrian Time

- In general, allocating excess pedestrian time to the clearance interval increases total intersection delay (see page 78).
- Pedestrian compliance significantly decreases with the allocation of excess time to the clearance interval. Compliance to the WALK interval consistently decreased at all locations observed: from 91 to 69 percent at 20th & M Streets, Washington; from 88 to 49 percent at University and Forest Avenues in Tempe; and from 78 to 50 percent at Grant and Ferry Streets in Buffalo (see page 81).
- In this study, very few pedestrians starting their crossing during the WALK interval failed to complete their crossing in time, even with a minimum clearance interval. This was found to occur because, at the volume levels observed, most pedestrians discharge from the curb within 2-3 seconds after the beginning of the WALK interval, giving them substantial extra clearance time beyond what is actually provided in the clearance interval (see page 83).
- Changes in the allocation of excess pedestrian time between the WALK and clearance intervals do not appear to affect compliance to the solid DONT WALK interval. The percentage of pedestrians starting during the solid DONT WALK was generally less than 10 but at Pearl and Church Streets in Buffalo consistently exceeded 30 percent (see page 85).
- The arrival rate of pedestrians at a signalized intersection crosswalk is not uniform, but is higher just prior to and during the WALK interval (see page 72).
- This analysis has resulted in a method of estimating pedestrian-caused right turn vehicle delay from hourly two-way pedestrian volume (see page 62).
- A recommended practice for traffic engineers is that all excess pedestrian time be allocated to the WALK interval unless peak hour pedestrian volumes exceed 15 per cycle. Where they do, a clearance interval timed at a walking speed of 3.5 ft./sec. would be used as described in the "minimum clearance interval" conclusions above.

### Flow Chart for Timing a Combined Pedestrian-Vehicular Interval

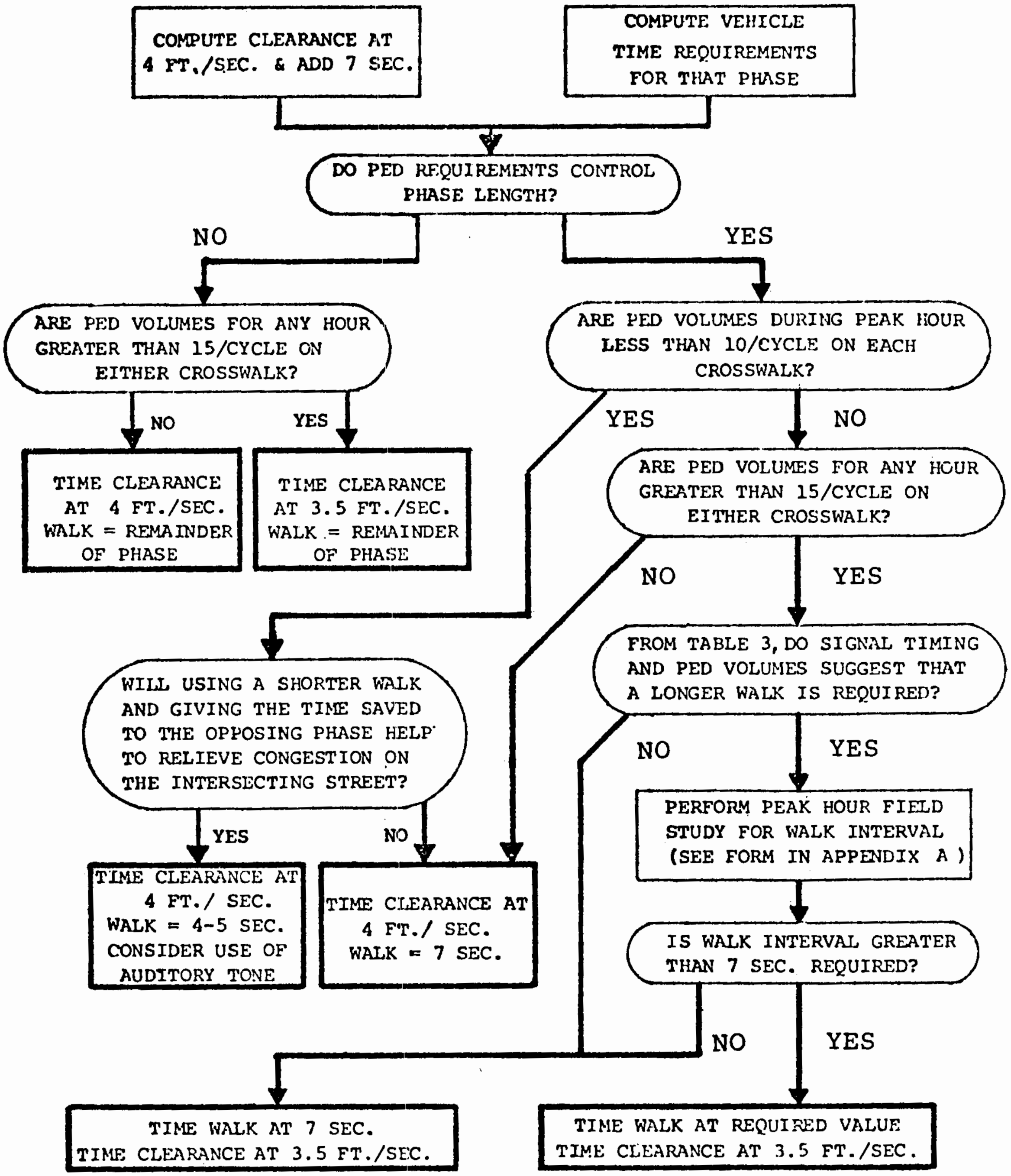
A flow chart depicting the methodology for timing a combined pedestrian-vehicular interval is depicted on Figure S-1. This is repeated later in the report as Figure 20 along with a detailed discussion of the signal timing logic. The figure describes in logical order the steps which would be taken in timing the WALK and clearance intervals for differing levels of vehicle and pedestrian demand. One additional guideline which is not directly related to the timing of pedestrian signals but which has significant impact on pedestrian delay pertains to the determination of cycle length. It is advantageous, from the perspective of pedestrian delay, to keep cycle lengths as short as possible. This will reduce the average time that pedestrians must wait for the display of the next WALK phase, and significantly reduce overall delay.

## ALTERNATIVE TYPES OF PEDESTRIAN SIGNAL PHASING

In this portion of the study, the desirability of using pedestrian signal phasing schemes other than the combined pedestrian-vehicular interval was examined. As before, alternative schemes were examined in light of pedestrian safety and delay. Pedestrian and vehicle delays for the early and late pedestrian release (see definitions on page 97) and scramble phasing schemes were compared to delay for the combined pedestrian vehicular interval. A safety comparison was made for all phasing schemes. A field study was performed for determining compliance at two locations in Sioux City, Iowa which were equipped with late pedestrian release timing. The conclusions derived for each of the phasing alternatives are described below:

### Early and Late Pedestrian Release

- Early pedestrian release phasing significantly increases vehicle delay without reducing pedestrian delay. It may provide some measure of additional safety, but the benefits were not precisely determined in this study (see page 102).
- Late pedestrian release phasing tends to increase overall intersection delay (sum of vehicle and pedestrian delay) at most volume levels. However, where a vehicle queue consistently exists in a right turn lane, late release is a good means for increasing capacity in that lane and with certain combinations of pedestrian and vehicle volumes will reduce overall intersection delay (see page 104).



NOTE: If this process gives different answers for various hours of day and if hardware configuration will allow, change timing by time-of-day.

FIGURE S-1. PEDESTRIAN SIGNAL TIMING FOR WALK AND CLEARANCE INTERVALS.

- Compliance to the late pedestrian release interval in Sioux City was remarkably high, typically with less than 3 percent of pedestrians in violation. However, it is expected that if a late release installation is provided in a city where this has not been tried before, pedestrian acceptance and the resultant compliance may be low. In this case, it is recommended that signs be provided to inform pedestrians that they are not permitted to begin their crossing with vehicles (see page 107).

## Scramble Timing

- Scramble timing always increases pedestrian delay. In the example used in this report, pedestrian delay was increased by over 200 percent with scramble (see page 111).
- Scramble timing may be able to increase the capacity of vehicle right turn lanes, but in so doing will increase delay on the through lanes. The delay effects are minimized where streets are narrow and right turn volumes are high (see page 114).
- Scramble timing creates an exclusive pedestrian phase which, if obeyed, can completely eliminate pedestrian-vehicle conflicts, thus improving the level of safety. However, it was observed in this study that scramble violation rates were generally higher on narrow streets, the condition for which scramble is most suitable from the delay perspective. Although violations are not always true indicators of a safety hazard, they tend to defeat the purpose for which scramble was designed. Scramble may have some application to intersections where the characteristics of the pedestrian population require special consideration. For example, it may be used at locations where there are many elderly pedestrians or young school children. If possible, scramble should be provided on an actuated basis so that the phase will not be introduced when pedestrians are not present (see page 114).

### Timing for the Partial Crossing of Wide, Channelized Streets

- Timing for the partial crossing of wide, channelized streets should be avoided if at all possible. This type of timing tends to leave many pedestrians remaining in the street at the end of the clearance interval (almost 70 percent for the example used in this research). It is desirable for the clearance interval to be timed for the entire crossing unless the median is greater than 20 feet wide. If this type of timing must be used to minimize the side street green time, signs indicating the intent of the timing and/or specially designed barriers should be provided on the median (see page 116).

### Flow Chart for Selecting Pedestrian Signal Phasing

A flow chart depicting the methodology for selecting pedestrian signal phasing is presented in Figure S-2. The methodology is based primarily on delay considerations simply because they are more easily quantified. Safety considerations should also be weighed, particularly in the case of scramble timing. A combined pedestrian-vehicle interval should be used in the vast majority of cases, and alternative types of phasing should be examined very closely before they are selected. This figure is repeated as Figure 25 later in the report, which is accompanied by a detailed discussion of signal phasing selection.

## OTHER AREAS OF PEDESTRIAN SIGNAL RESEARCH

All areas of research which were not directly related to either of the two previous categories were examined in this section. All conclusions were based on the data which had been collected for other elements of the research. The data were manipulated into the form required for the specific area of research addressed. For the research involving pedestrian factors for the Highway Capacity Manual, substantial additional analysis of the collected data was performed. The conclusions for each area are listed below.

### Time-of-Day Adjustments of Pedestrian Signal Timing

- Variations in the required length of pedestrian intervals based on volume conditions over the period of a day will usually be only several seconds at the maximum. This indicates that time-of-day adjustments to timing are of little potential value (see page 125).

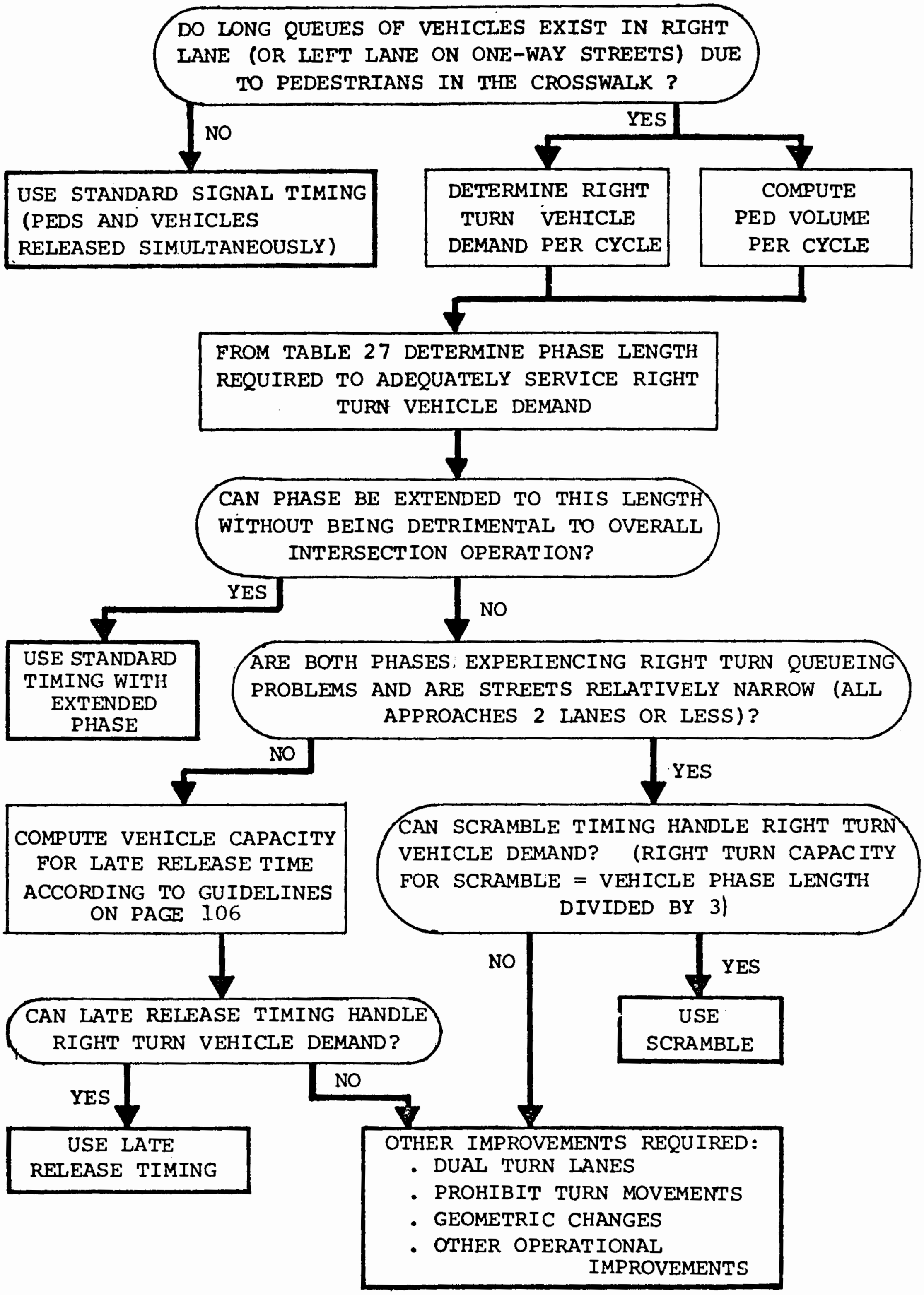


FIGURE S-2. SELECTION OF PEDESTRIAN SIGNAL PHASING.

- Hardware considerations may also preclude the effective use of time-of-day adjustments. The only case in which time-of-day adjustments would be practical is when the signal controller uses a separate off-peak timing plan which does not include one of the three pedestrian peak periods (morning, noon, and evening) (see page 126).

## Application of Correction Factors to the Highway Capacity Manual Based on Vehicle and Pedestrian Activity Levels

- Curves relating vehicle capacity in the right lane to pedestrian volumes and vehicle right turn percentages have been developed and are presented in Figure 26. It was found to be difficult to accurately expand these results to include the right lane in a capacity analysis for the total intersection approach (see page 135).

- The development of final pedestrian correction factors would best take place along with the restructuring of the "Intersection Capacity" chapter of the Manual, should that occur (see page 139).

- Table 27 can be used to estimate the capacity of an exclusive right turn lane based on signal timing and pedestrian volume.

## General Observations on Pedestrian Flow Characteristics

- Pedestrian peak hours for central business districts usually occur during the morning and evening peak traffic periods and during the lunch hour period, depending upon adjacent land use. Peak hours outside the CBD are somewhat less predictable, also being contingent upon the predominant land uses. (see page 139).

- Daily pedestrian volume on one crosswalk has been found to be as much as four times the volume on other crosswalks at the same intersection. Timing requirements should be based on the highest volume crosswalks (see page 139).

- Short pedestrian counts are an effective way to obtain accurate volume data with a minimum of cost (see page 140).

# CHAPTER I

## INTRODUCTION

This report presents the final results of the signal timing element of the FHWA research project "Urban Intersection Improvements for Pedestrian Safety". The report discusses the research which was conducted, and recommends improvements in the timing of pedestrian traffic signals both to maximize safety and to minimize delay.

The subject matter of this volume is limited to the timing of standard pedestrian WALK and DONT WALK signals although some of the results of this study may also be useful for signal timing at intersections without such signals. The Manual on Uniform Traffic Control Devices (MUTCD) (1) has set forth policies on the timing of pedestrian signals and these have been used as the starting point for most of the analyses.

There are two basic decisions which the traffic engineer must face in the timing of pedestrian signals. The first is the selection of the type of timing or phasing at a given location. This is generally a straightforward consideration unless some problem with standard timing has been observed. The second decision is the length of the intervals to be used. This issue is the emphasis of a major portion of the research to be presented in this volume. Specifically, the elements covered in this volume include the following:

- Chapter II - Timing for a Combined Pedestrian - Vehicular Interval
    1. Minimum WALK time.
    2. Minimum clearance interval.
    3. Allocation of excess pedestrian time.
- Chapter III - Alternative Phasing Schemes
    1. Early and late release of pedestrians with respect to vehicles.
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    3. Signal phasing for the partial crossing of wide, channelized streets.
- Chapter IV - Other Areas of Pedestrian Signal Research
    1. Time-of-day adjustments of pedestrian signal timing.
    2. Application of correction factors to the Highway Capacity Manual based on vehicle and pedestrian activity levels.

3. General observations on pedestrian flow characteristics.

For each of the above topics, the study approach is outlined, and the data collection and analysis procedures are described. A substantial amount of data was collected for most of the areas of research. In each case, samples were taken in several parts of the country, the three primary locations being Washington, D.C., Buffalo, New York, and Phoenix, Arizona. Several unique analysis procedures have been used to mold these data into usable concepts, and procedures have been developed for wide application. The conclusions derived from this study should significantly help the traffic engineer in the timing of signals for safe and efficient operation for both pedestrians and vehicles.

# CHAPTER II

## TIMING FOR A COMBINED PEDESTRIAN-VEHICULAR INTERVAL

Timing for a combined pedestrian-vehicular interval, that is, the simultaneous release of vehicles with pedestrians on the parallel crosswalks, is the first of four basic phasing schemes described in the MUTCD (Section 4D-7). It is the most predominant phasing scheme presently in use in the United States. This chapter is devoted to an analysis of the timing procedures used for this type of phasing. First, an analysis of the desirable minimums for the WALK and clearance intervals is presented. Then, an appraisal is made of how to best allocate any excess time which may exist at intersections where vehicle timing requirements exceed those for pedestrians. Although the analysis is primarily geared toward this more conventional type of phasing, many of the conclusions derived, particularly those related to the minimum intervals, are applicable to other types of phasing as well.

### MINIMUM WALK TIME

### Study Approach

The pedestrian WALK interval, whether solid or flashing, is designed to convey the message that a pedestrian may <u>begin</u> his crossing of a street. Theoretically, the length of this interval should be such that all pedestrians waiting to cross the street at the beginning of the interval (this group of waiting pedestrians will be referred to as a "pedestrian queue") are allowed ample time to leave the curb before the flashing DONT WALK clearance interval is displayed. It is <u>not</u> designed to permit pedestrians to complete their crossing, although this is the meaning pedestrians often perceive (2). Under ideal pedestrian behavior, the WALK interval would include a perception/reaction time and the actual time taken for the last pedestrian in the queue to step into the street. The MUTCD recommends that 7 seconds be used for the minimum WALK interval as quoted below:

> "Under normal conditions, the WALK interval should be at least 7 seconds so that pedestrians will have adequate opportunity to leave the curb, before the clearance interval is shown. However, the WALK interval itself need not equal or exceed the total crossing time calculated for the street width, as many pedestrians will complete their crossing during the flashing DONT WALK clearance interval."

It was the objective of this part of the research to establish desirable minimum pedestrian WALK times for a wide range of pedestrian volumes. This minimum is defined here as the pedestrian "discharge time" or the time required for the last pedestrian in a queue to step into the street after the WALK indication is first displayed. It appears logical that discharge times would increase with increasing pedestrian volume. A field study was initiated to observe actual discharge times of pedestrians under various pedestrian volume levels. The two primary questions addressed in the discharge time study were:

- when should the minimum WALK interval be greater than 7 seconds?
- when, if at all, should the interval be less than 7 seconds?

The data collection methodology and analysis procedures used to answer these questions are described below.

## Data Collection

Observations of pedestrian queue discharge times were conducted during the summer and fall of 1975 in Washington, D.C., Phoenix, Arizona, and Buffalo, New York. These cities were selected to gain a reasonably representative sample of urban areas in the United States. Five different intersection crosswalks were observed for a total of 17 hours. Crosswalks with some of the highest available peak hour pedestrian volumes were selected. Pedestrian activity between the hours of 12:00 noon to 2:00 p.m. was observed at each crosswalk, and various evening peak hours were examined at three of the crosswalks. In addition, one location was observed for two consecutive days to examine the variation in pedestrian volumes from day to day.

The specific crosswalks observed are listed below:

| City | Crosswalk | Date | Time |
|---|---|---|---|
| Washington | Crossing M St. and Rhode Island Avenue, east of Connecticut Avenue | 8/12/75 | 12-2 |
| Washington | Crossing 16th St., North of K Street | 8/13/75 | 12-2<br>5-6 |
| | | 8/14/75 | 12-2 |

| City | Crosswalk | Date | Time |
|---|---|---|---|
| Washington | Crossing 14th St., South of New York Avenue | 8/15/75 | 12-2 |
| Phoenix | Crossing Monroe St., East of Central Avenue | 11/20/75 | 12-2<br>3-5 |
| Buffalo | Crossing Main St., North of Eagle Street | 12/02/75 | 12-2<br>3-5 |

In addition to the discharge times, the data collected at the above locations also included studies of pedestrian walking speeds, which were used in the analysis of pedestrian clearance times. That portion of the data collection procedure used to gather pedestrian discharge times is described below.

Two observers were used to collect the data, one on each end of the crosswalk. Each was equipped with a stopwatch, recording forms, a clipboard and pencils. Training of the observers and a practice session took place before the first count began. The observers were instructed to count the number of pedestrians standing in a queue waiting for the onset of the WALK interval and to start the stopwatch the instant the WALK signal was displayed. The time at which the last pedestrian in the queue stepped into the street was noted and recorded in the appropriate place on the form. The watch was kept running for further use in data collection for pedestrian clearance times. Additional pedestrians beginning their crossing after the initial queue had departed were also observed and the number was recorded on the form, thus providing a total directional volume count. A sample form is shown in Appendix A.

## Data Analysis

### Walk Interval Greater than 7 Seconds

The first step in the analysis was to develop relationships between queue size for each cycle and the discharge time for each queue. Graphical plots were developed for some of the data in each city, and a linear least squares curve fit was performed on each hour of collected data. A representative plot for one hour of data in each city is shown in Figures 1-3.

The descriptions of the regression lines are presented in Table 1. The positive slopes of the lines support the relationship between queue size and discharge time which was first hypothesized in the study, that is, increasing

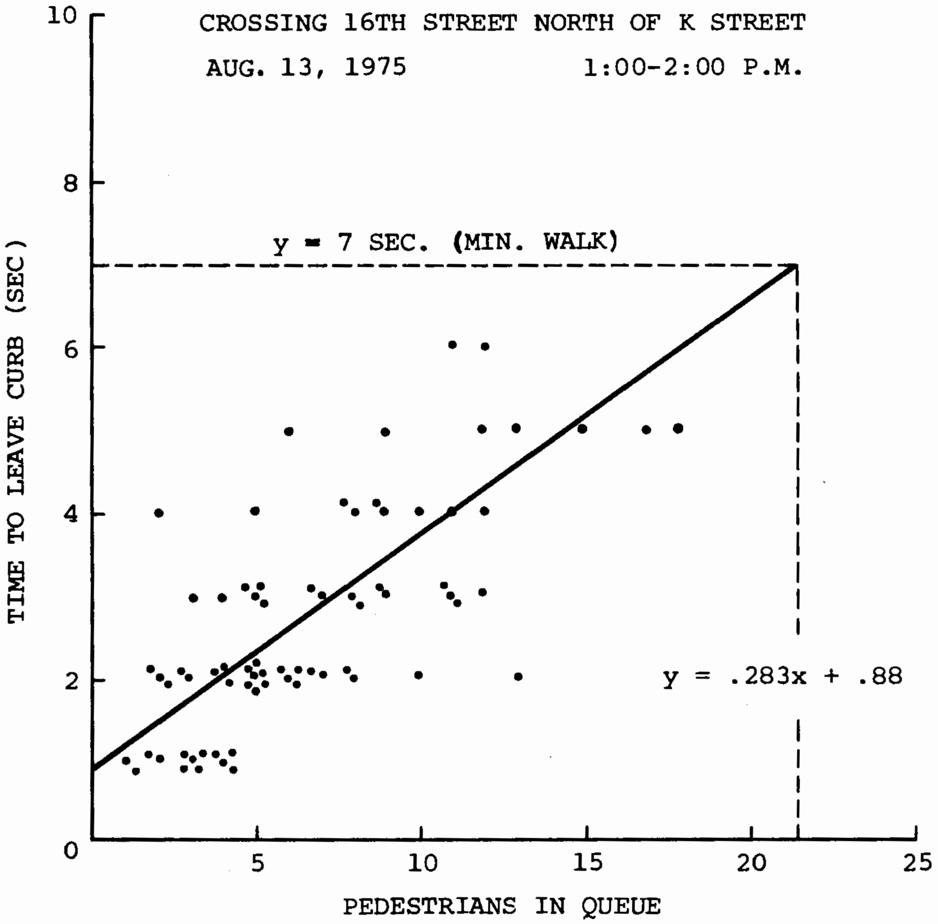

FIGURE 1. QUEUE DISCHARGE TIME VERSUS NUMBER OF PEDESTRIANS IN A QUEUE, WASHINGTON, D.C.

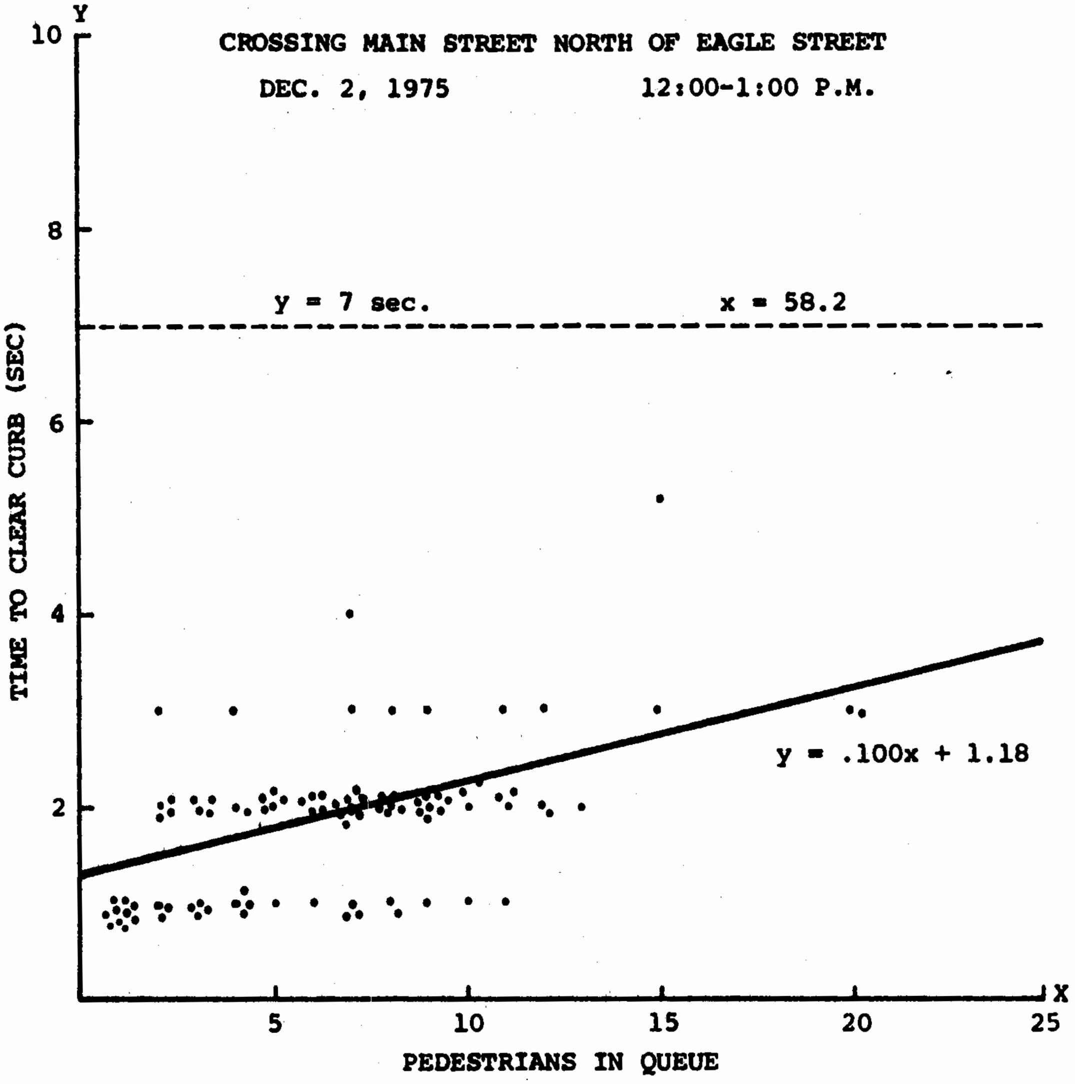

FIGURE 2. QUEUE DISCHARGE TIME VERSUS NUMBER OF PEDESTRIANS IN A QUEUE, BUFFALO.

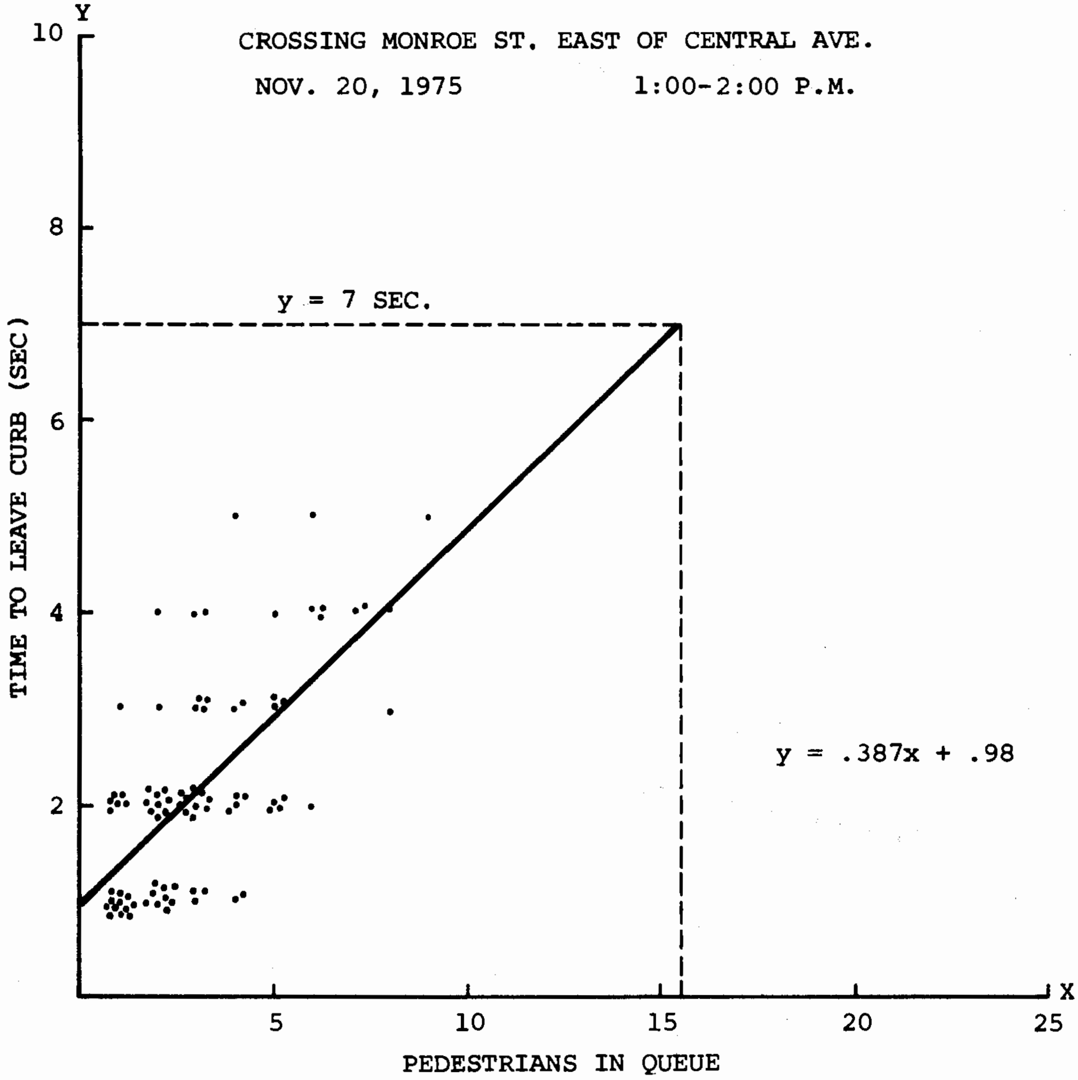

FIGURE 3. QUEUE DISCHARGE TIME VERSUS NUMBER OF PEDESTRIANS IN A QUEUE, PHOENIX.

### TABLE 1. RESULTS OF REGRESSION ANALYSIS OF DISCHARGE TIME AND QUEUE SIZE.

| Intersection | Time | Slope | y - intercept | r | Queue size at y = 7 sec. (min. WALK) |
|---|---|---|---|---|---|
| Washington | | | | | |
| Conn, & M | 12-1 | .151 | 2.51 | .42 | 30.9 |
| | 1-2 | .238 | 1.48 | .63 | 23.2 |
| 16th & K (8/13) | 12-1 | .289 | 0.79 | .42 | 21.5 |
| | 1-2 | .283 | 0.88 | .56 | 21.6 |
| | 5-6 | .241 | 0.88 | .72 | 25.4 |
| 16th & K (8/14) | 12-1 | .257 | 0.88 | .69 | 23.8 |
| | 1-2 | .297 | 0.61 | .86 | 21.5 |
| N.Y. & 14th | 12-1 | .196 | 2.53 | .38 | 22.9 |
| | 1-2 | .218 | 2.17 | .37 | 22.2 |
| Phoenix | | | | | |
| Central & Monroe | 12-1 | .343 | 1.24 | .60 | 16.8 |
| | 1-2 | .387 | 0.98 | .67 | 15.5 |
| | 3-4 | .328 | 1.15 | .47 | 17.8 |
| | 4-5 | .424 | 0.87 | .41 | 14.4 |
| Buffalo | | | | | |
| Main & Eagle | 12-1 | .100 | 1.18 | .54 | 58.2 |
| | 1-2 | .052 | 1.36 | .34 | 108.0 |
| | 3-4 | .100 | 1.32 | .32 | 56.8 |
| | 4-5 | .130 | 1.36 | .36 | 43.3 |

discharge time with increasing queue size. All intercepts along the Y axis are positive, indicating the presence of the perception/reaction time factor. The intercepts range from 0.61 to 2.53 seconds.

It became fairly obvious, even in the observation stage, that pedestrian behavior in the three cities was substantially different. Pedestrians in Washington, D.C. and Buffalo seemed ready to begin their crossing immediately while those in Phoenix appeared to be much less anxious. Intuitively, this would lead one to believe that discharge times in Phoenix would be greater than in the other two cities. The analysis of the data confirmed this observation.

The primary difference in discharge times among the three cities appears to rest in the slope of the lines. Slopes are highest in Phoenix and lowest in Buffalo, indicating a more rapid discharge in Buffalo. The characteristics of the intersections observed in Washington, D.C. were consistently between the Phoenix and Buffalo results. These differences cannot be totally attributed to variations in pedestrian behavior in the three cities, but must also be examined in light of the individual intersections observed.

First, it should be noted that pedestrian volumes observed in Phoenix for all but one hour were by far the lowest of the three cities. As a result, few data points were obtained for the larger queue sizes and most of the queues consisted of five persons or less. This may have caused some of the discharge times to be based on occasional "late starters" in the queue rather than on the actual effect of volume on discharge time. For example, if one inattentive pedestrian in a small queue takes 5 seconds to leave the curb, the time recorded will not be indicative of the volume effect but on the behavior of that one pedestrian. Where volumes are higher, pedestrians begin to queue behind one another and discharge times increase, usually to a value greater than the discharge time of the inattentive pedestrian. This suggests that the discharge time vs. queue size relationship may actually be non-linear, most likely a curve with decreasing positive slope. Because of the general lack of higher volume values in Phoenix, only the 12:00-1:00 hour is useful for drawing conclusions as to the effect of volume on queue discharge time.

The explanation for the low slopes in the Buffalo data probably lies with both the nature of the city and the intersection observed. It was found during the study in Buffalo that the compliance of pedestrians to signal indications was relatively low. People appeared to be in much more of a hurry in Buffalo than in Phoenix (the much lower temperatures during data collection in Buffalo may have been partially responsible for this). Contributing even more to the low slope, however, may be the length of curb space available to pedestrians for queueing at this particular intersection. Main and Eagle Streets in Buffalo form a "T" intersection with few restrictive influences, such as parking, and almost unlimited curb space was available for queueing. Consequently, it was observed that pedestrians often spread out all along the curb rather than form a queue in a limited space. It is not surprising that discharge times would be low under these conditions.

It is a significant observation that even though intersections with high volumes and good compliance were selected for study in the three cities, only a very few queues were found to have discharge times greater than 7 seconds, the minimum WALK time. In fact, only in Washington, D.C. were any of that magnitude observed. Certainly, higher volumes can be found in other cities, but the above results indicate that the necessity of increasing the minimum WALK interval to discharge a waiting queue is not widespread.

Although it was found that a minimum WALK interval of greater than 7 seconds would seldom be required for the cities studied, it was felt by the research team that an analysis of when it is required would be of assistance to the traffic engineer faced with such a problem. The Washington, D.C. data, which was the most consistent of the three cities and contained the highest pedestrian volumes, forms a good base for performing this analysis.

From Table 1, an average of the queue sizes at y = 7 seconds (last column in the table) for the 9 hours of data collected in Washington, D.C. indicates that a queue size of approximately 24 persons would be the threshold beyond which a WALK interval of greater than 7 seconds would be required. This value will vary depending on the length of curb utilized by pedestrians for queueing. The curb space utilized by pedestrians at the intersections studied in Washington, D. C., generally ranging between 15 and 30 feet for heavy pedestrian volumes, is probably typical of that utilized in other major urban areas.

The next step in the analysis was to develop a relationship between the critical average queue size value of 24 persons and the 95 percentile queue size based on cyclic volume distributions (Figure 4). The mean queue size ($\bar{Q}$) for each hourly data sample was plotted on the Y axis and the corresponding value of the upper 95 percent confidence limit ($\bar{Q} + 1.96\sigma_Q$) on the X axis. A linear regression fit

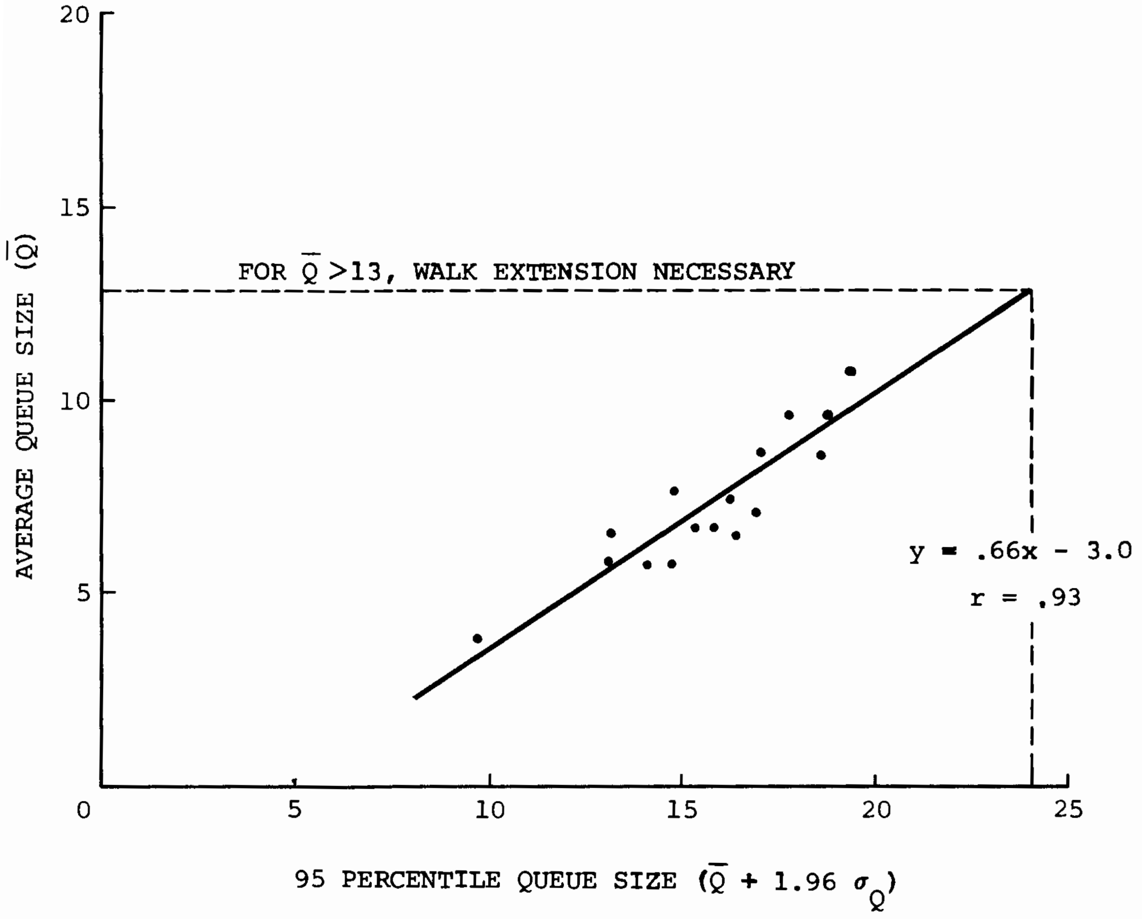

FIGURE 4. AVERAGE QUEUE SIZE VS. 95TH PERCENTILE QUEUE SIZE.

was made to the data as shown in the figure. It can be deduced from this figure that when the average queue is less than 13 pedestrians, 95 percent of the queues will be less than 24 persons, the critical queue size value. This 95 percent figure is probably a reasonable goal for the accommodation of the discharge of pedestrian queues at this high volume level.

Having established a threshold value of average queue size, it would be advantageous to proceed one step further and develop a threshold for hourly volume, the data typically available to the traffic engineer. The variables linking hourly volume to average queue size include the pedestrian arrival rate, signal timing, and directional split.

Typical directional distributions were examined for each hour of data collection and are summarized in Table 2. The greatest split during the midday period across all cities was approximately 60/40. The split during the p.m. peak hour at the Buffalo intersection was about 79/21, significantly higher. This intersection serves as a primary link between a major shopping and office complex and nearby parking area and bus stop. During the p.m. peak hour, flow tends to be highly imbalanced toward the parking area and bus stop, explaining the magnitude of this high directional split. A reasonable estimate of the split for use in relating hourly volumes to average queue size was taken to be 60/40. This assumption is used in all further analysis in this study. These splits are likely to vary from location to location depending on land use and proximity of parking and transit facilities. A higher split could be used for analyzing a particular intersection if desired.

In theory, a queue of pedestrians forms during the time allocated to the solid and flashing DONT WALK intervals. Due to violation rates, which may be different at each location, the number of people actually waiting at the beginning of WALK may be somewhat less than this. Given a certain arrival rate and the assumed queueing time, an estimate of average queue size may be made mathematically. A study of arrival rates was performed in this research. The arrival rate was found to be highest during the WALK interval and toward the end of the solid DONT WALK interval. However, a conservative assumption of uniform arrivals was used in this analysis. Arrival rates are discussed further on Page 69. The average queue size in the heaviest direction of flow can be approximated by the formula below:

$$\bar{Q} = \frac{V_T}{3600} \times 0.6 \times (FDW + SDW)$$

## TABLE 2. DIRECTIONAL DISTRIBUTION OF PEDESTRIANS ON A CROSSWALK.

| Intersection | Time | NB or EB Vol. | SB or WB Vol | Total Vol. | Highest one-way vol. over total Vol. |
|---|---|---|---|---|---|
| Washington | | | | | |
| Conn & M | 12-1 | 725 | 710 | 1435 | 50.5 |
| | 1-2 | 645 | 806 | 1451 | 55.5 |
| 16th & K (8/13) | 12-1 | 648 | 589 | 1237 | 52.4 |
| | 1-2 | 527 | 603 | 1130 | 53.4 |
| | 5-6 | 586 | 346 | 932 | 62.9 |
| 16th & K (8/14) | 12-1 | 648 | 581 | 1229 | 52.7 |
| | 1-2 | 522 | 608 | 1130 | 53.8 |
| N.Y. & 14th | 12-1 | 629 | 548 | 1177 | 53.4 |
| | 1-2 | 550 | 468 | 1018 | 54.0 |
| Phoenix | | | | | |
| Central & Monroe | 12-1 | 379 | 362 | 741 | 51.1 |
| | 1-2 | 197 | 212 | 409 | 51.8 |
| | 3-4 | 118 | 136 | 254 | 53.5 |
| | 4-5 | 109 | 117 | 226 | 51.8 |
| Buffalo | | | | | |
| Main & Eagle | 12-1 | 799 | 542 | 1341 | 59.6 |
| | 1-2 | 455 | 543 | 998 | 54.4 |
| | 3-4 | 310 | 160 | 470 | 66.0 |
| | 4-5 | 477 | 128 | 605 | 78.8 |

where $\bar{Q}$ = Average queue size in heaviest direction of flow

$V_T$ = Total two-way hourly pedestrian volume

FDW = Time allocated to flashing DONT WALK

SDW = Time allocated to solid DONT WALK

0.6 represents 60/40 directional split

Table 3 was constructed from this formula for a range of pedestrian volumes and DONT WALK times where consideration should be given to extending the WALK interval beyond the minimum of 7 seconds. This table should not be used to set the minimum but should serve to identify those locations which should be further examined for minimum WALK interval extensions. Field studies should be performed to determine the exact timing requirements for these locations. As an example of the use of the table, consider a crosswalk with 1200 pedestrians per hour and 40 seconds of flashing and solid DONT WALK time (see Table 3). The average queue size would be 8 which is less than the critical value of 13. Higher pedestrian volumes or longer DONT WALK times may yield a computed queue size of at least 13 (see other examples in Table 3). This would necessitate the performance of a field study to determine the length of the WALK interval required.

Field observation in this study revealed that violation rates were often quite high (see page 80). This would result in significant overestimates of average queue size if Table 3 were used. In cities where a greater compliance is achieved, the table will be more accurate. The formula presented above can be adjusted to take compliance in to account by reducing the length of the WALK time in proportion to the estimated rate of non-compliance. The 0.6 value can also be adjusted, if desired, to correspond to a higher or lower directional split.

Where no pedestrian volume counts exist from which to make an estimation of WALK extension needs, counts should be taken to provide the necessary information. These should be taken at the anticipated peak pedestrian hours and need not involve observation over an entire hour. A discussion of short pedestrian counts is presented on page 140.

The field study for actually setting the minimum WALK time would consist of observing the discharge time for the largest queue each cycle on the given crosswalk. Queues would be observed over the peak pedestrian hour (or hours)

# TABLE 3. AVERAGE QUEUE SIZE FOR VARIOUS HOURLY VOLUMES AND QUEUEING PERIODS.

| SDW + FDW Time (Sec.) | Two-way Hourly Pedestrian Volumes | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 600 | 800 | 1000 | 1200 | 1400 | 1600 | 1800 | 2000 | 2200 | 2400 | 2600 | 2800 | 3000 |
| 20 | 2 | 3 | 3 | 4 | 5 | 5 | 6 | 8 | 7 | 8 | 9 | 9 | 10 |
| 25 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 13 |
| 30 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
| 35 | 4 | 5 | 6 | 7 | 8 | 9 | 11 | 12 | 13 | 14 | 15 | 16 | 18 |
| 40 | 4 | 5 | 7 | 8 | 9 | 11 | 12 | 13 | 15 | 16 | 17 | 19 | 20 |
| 45 | 5 | 6 | 8 | 9 | 11 | 12 | 14 | 15 | 17 | 18 | 19 | 21 | 23 |
| 50 | 5 | 7 | 8 | 10 | 12 | 13 | 15 | 17 | 18 | 20 | 22 | 23 | 25 |
| 55 | 6 | 7 | 9 | 11 | 13 | 15 | 17 | 18 | 20 | 22 | 24 | 26 | 28 |
| 60 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 |
| 65 | 7 | 9 | 11 | 13 | 15 | 17 | 20 | 22 | 24 | 26 | 28 | 30 | 33 |
| 70 | 7 | 9 | 12 | 14 | 16 | 19 | 21 | 23 | 26 | 28 | 30 | 33 | 35 |
| 75 | 8 | 10 | 13 | 15 | 18 | 20 | 23 | 25 | 28 | 30 | 32 | 35 | 38 |
| 80 | 8 | 11 | 13 | 16 | 19 | 21 | 24 | 27 | 29 | 32 | 35 | 37 | 40 |
| 85 | 9 | 11 | 14 | 17 | 20 | 23 | 25 | 28 | 31 | 34 | 37 | 40 | 42 |
| 90 | 9 | 12 | 15 | 18 | 21 | 24 | 27 | 30 | 33 | 36 | 39 | 42 | 45 |
| 95 | 10 | 13 | 16 | 19 | 22 | 25 | 28 | 32 | 35 | 38 | 41 | 44 | 47 |
| 100 | 10 | 13 | 17 | 20 | 23 | 27 | 30 | 33 | 37 | 40 | 43 | 47 | 50 |

Assumptions: Uniform Arrivals
60/40 Direction Split

Note: This is a very conservative estimate of average queue size to be used only for eliminating those locations not requiring further study for extended WALK time. Values below dashed line indicate locations where a field study should be performed to determine WALK time requirements.

and timing would be set to accommodate the discharge time requirements for at least 95% of the cycles observed. A sample form for the performance of this study is presented in Appendix A.

As a part of this research effort, a market analysis was carried out to determine the applicability of these results to other cities in the United States. This has been done by extrapolating the research from Washington, Buffalo, and Phoenix to other cities based on general population and CBD characteristics. The major issue was to determine at how many signalized intersections there are likely to be crosswalks where the 7-second WALK interval is insufficient to clear the volumes of pedestrians present.

Using the data in Table 3 as a basis, it can be readily seen that for crosswalks with peak pedestrian volumes less than 800 per hour, the 7-second WALK should virtually always be adequate. Between 800 pedestrians per hour and 2000 pedestrians per hour, some crosswalks may require an additional field study to determine WALK interval requirements. For crosswalks with volumes greater than 2000 pedestrians per hour, nearly all such crosswalks will require a field study to determine the length of the WALK interval, and it is quite likely that an extension of the minimum WALK interval will be required. The number of signalized intersections that are likely to fall into each of these categories is shown in Table 4, for Washington, D.C., Buffalo, and Phoenix. Also shown in the table are other major United States cities where the number of affected locations has been estimated.

For Washington, there are only four signalized intersections where crosswalk volumes are greater than 2000 pedestrians per hour. None were encountered in Buffalo or Phoenix. Translating this data to other United States cities, it is estimated that there may be approximately 80 of these locations in New York City, perhaps 30 in Chicago, and far fewer in other large cities. It is expected that few if any locations will be found in metropolitan areas smaller than Atlanta. Urban characteristics that produce these types of pedestrian volumes include high density development and heavily utilized public transportation systems. For example, locations in New York and Chicago where these volumes occur are invariably in the vicinity of subway and railroad stations which serve major office or commercial areas.

TABLE 4. ESTIMATED PEDESTRIAN VOLUME CHARACTERISTICS OF UNITED STATES CITIES.

| Rank | City | SMSA 1970 Population x 1000 | SMSA Approx. No. of Signalized Intersections | Estimated Number of Signalized Intersections With Hourly Crosswalk Volumes | |
|---|---|---|---|---|---|
| | | | | 800-2000 | >2000 |
| 7 | Washington | 2,861 | 2,000 | 30 | 4 |
| 34 | Phoenix | 957 | 800 | 0 | 0 |
| 24 | Buffalo | 1,349 | 900 | 4 | 0 |
| 1 | New York | 11,529 | 11,000 | 500 | 80 |
| 2 | Los Angeles | 7,032 | 5,000 | 40 | 5 |
| 3 | Chicago | 6,979 | 5,000 | 300 | 30 |
| 4 | Philadelphia | 4,818 | 3,000 | 100 | 15 |
| 5 | Detroit | 4,200 | 3,000 | 80 | 10 |
| 6 | San Francisco-Oakland | 3,110 | 2,100 | 40 | 5 |
| 8 | Boston | 2,754 | 2,300 | 80 | 10 |
| 9 | Pittsburgh | 2,401 | 2,100 | 40 | 5 |
| 10 | St. Louis | 2,363 | 2,000 | 40 | 5 |
| 11 | Baltimore | 2,071 | 1,600 | 40 | 5 |
| 12 | Cleveland | 2,064 | 1,500 | 60 | 8 |
| 13 | Houston | 1,985 | 1,200 | 20 | 0 |
| 14 | Newark | 1,857 | 1,200 | 30 | 5 |
| 15 | Minneapolis | 1,814 | 1,000 | 20 | 3 |
| 16 | Dallas | 1,556 | 900 | 20 | 0 |
| 17 | Seattle | 1,422 | 800 | 20 | 0 |
| 18 | Anaheim-Santa Ana | 1,420 | 600 | 10 | 0 |
| 19 | Milwaukee | 1,404 | 700 | 20 | 2 |
| 20 | Atlanta | 1,390 | 700 | 20 | 2 |
| | | | 44,500 | 1,480 | 200 |

Source of population data: The World Almanac and Book of Facts, 1974 Edition.

Using a national average of one signal per 1000 population* in urban areas, there are approximately 150,000 signalized intersections in the United States. From the preceding analysis, it would appear that between 200 and 500 or about 0.3 percent of all signalized intersections in the United States exceed the pedestrian volumes suggested. Finally, it is estimated that the vast majority of these signal locations presently have WALK intervals above the recommended 7-second minimum. Thus, it appears that this issue does not have major national significance. Approximately 99.7% of the signalized intersections do not require WALK intervals greater than 7 seconds. Those that do are scattered in the nation's larger cities, and most of these already have WALK intervals long enough to accommodate the discharge times of peak hour pedestrian queues.

### Walk Interval Less than 7 Seconds

Another important issue is the establishment of the minimum WALK time for low pedestrian volumes, that is, whether the WALK interval should ever be less than 7 seconds. The determinants for this lower limit are both the perception/reaction time of a pedestrian at the signal and the probability of inattentiveness on the part of the pedestrian. The y intercepts in Table 1 are generally indicative of the perception/reaction factor. Taking the highest values of the group might suggest that a value of 2.5 seconds be used. Two or three seconds should probably be added to the figure to allow pedestrians to proceed a reasonable distance into the street before the clearance interval begins. This would mean that a WALK interval of at least 4-5 seconds should be provided.

In order to examine the question in greater depth, queues of less than 4 persons were isolated from the discharge time samples. This limit was used because at this queue size there is always sufficient space for each pedestrian to stand directly at the curb so that all pedestrians in the queue can discharge independently of others waiting on the curb. Thus, the discharge time for this subsample consists basically of the perception/reaction factor only. Of the approximately 450 queues of this size observed, 96 percent had discharge times of 3 seconds or less, 99 percent had discharge times of 4 seconds or less and only two queues had a discharge time of as high as 7 seconds. These results appear to confirm the conclusion that a 4 to 5 second WALK

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* Selected data from 1965 National Safety Council Annual Traffic Inventory, Chicago, Illinois.

is adequate to serve these low pedestrian volumes. At volumes of less than 10 pedestrians per cycle over 95 percent of the queue discharge times should be under 4 seconds. Approximately 99 percent would be less than 5 seconds.

Compounding the problem of establishing the minimum WALK, however, is the factor of pedestrian inattentiveness. Some pedestrians, while waiting to cross the intersection, may be engaged in conversation, observing the scenery or other activities that may prevent them from seeing the WALK indication immediately. If the WALK interval is sufficiently short, the pedestrian may not look up to see the signal until the clearance interval begins. This may cause some confusion in the pedestrian's mind and may lead to an undesirable behavior. The question, of course, is one of the importance and probability of this inattentiveness factor. It is one that may vary with age group (e.g., it may occur most frequently with school children), with location and with other environmental factors as well. Inattentiveness may occur more often at low volume locations (one or two pedestrians per cycle) because of the scarcity of other pedestrians, whose movement may alert the inattentive pedestrian to the display of the WALK interval. Although the data presented here suggest that inattentiveness occurs infrequently, it is certainly a factor which deserves consideration especially where it is expected more frequently. A conclusive answer cannot be attained by field studies, but should probably rest on the combined judgement of engineers and transportation policy makers. At any rate, the perception/reaction factor dictates an interval of at least 4 to 5 seconds, only 2 to 3 seconds less than the present minimum.

One means of dealing with the inattentiveness problem might be to provide a short auditory tone at the beginning of each WALK interval. Some research has been performed on auditory signals (3) but more research should be performed for this specific application to determine its effectiveness.

The discussion to this point has suggested that lowering the minimum WALK interval to 4 or 5 seconds will not significantly impact pedestrian safety. However, this does not mean that this minimum should always be used at these volume levels. It should only be used when doing so will "significantly benefit" traffic on an opposing signal phase. This significant benefit would consist of the full or partial relief of traffic congestion during some part of the day. For example, if pedestrian volumes indicate that the minimum WALK interval on the minor street phase can be reduced (less than 10 pedestrians per cycle as mentioned previously is a reasonable threshold) and long peak hour queues exist on the major street, 2-3 seconds can be taken from the minor street green phase and allocated to the major street green phase. This would increase capacity on the major street by approximately one vehicle per lane per cycle. When expanded to an hourly basis, this could increase traffic capacity in some cases by over 10 percent and thereby help to alleviate the queuing problems. Where such congestion problems do not exist, lowering the minimum WALK intervals will only slightly reduce vehicle delay, a benefit which probably would not justify the use of the lower minimum. It would be justified in the previous case because vehicle delay under congested conditions (i.e. many loaded cycles) is much greater since many vehicles may be required to wait through more than one cycle. Observation in the field should be used to determine if all the necessary conditions for lowering the WALK interval are met.

## Review of Related Data

During the performance of this research a book was published containing data addressing some of the same issues as addressed in this study. The title of the publication is Urban Space for Pedestrians and is a 1975 report of the Regional Plan Association of New York City authored by Pushkarev and Zupan (4). The relationship between that study and this research is briefly discussed below.

Urban Space for Pedestrians compasses many aspects of pedestrian characteristics and space requirements. Within the document, approximately 6 pages are devoted to pedestrian behavior and requirements at signalized intersections, most of which is based on data developed by Oeding in West Germany. A formula for computing the starting time required for pedestrians was developed by Oeding as follows (page 115 in Pushkarev and Zupan):

Starting time (seconds) = Relative Accumulation (persons per foot of curb space) x 1.64 + 4

Pushkarev and Zupan note that three persons per foot of crosswalk width would require about 9 seconds of starting time. Starting times for accumulations of up to 10 persons per foot are presented in Table 5, taken directly from their book.

## TABLE 5. REQUIRED STARTING AND CROSSING TIMES AT VARYING RATES OF PEDESTRIAN FLOW.

| Average Sidewalk Flow peds./min/ft | | Minimum Pedestrian Green Time (starting time + crossing time), secs |
|---|---|---|
| Crossing 65-ft roadway with 60 secs nongreen time per cycle | | |
| 0.5 | OPEN | 4.4 + 14.5 = 18.9 |
| 1 | UNIMPEDED | 4.8 + 14.5 = 19.3 |
| 2 | | 5.6 + 17.0 = 22.6 |
| 3 | IMPEDED | 6.5 + 20.0 = 26.5 |
| 4 | | 7.3 + 20.0 = 27.3 |
| 5 | | 8.1 + 20.0 = 28.0 |
| 6 | | 8.9 + 20.0 = 28,9 |
| 7 | CONSTRAINED | 9.7 + 20.0 = 29.7 +? |
| 8 | | 10.6 + 20.0 = 30.6 +? |
| 9 | | 11.4 + 20.0 = 31.4 +? |
| 10 | | 12.2 + 20.0 = 32.2 +? |
| Crossing 34 ft roadway with 40 secs nongreen time per cycle | | |
| 0.5 | OPEN | 4.3 + 7.5 = 11.8 |
| 1 | UNIMPEDED | 4.5 + 7.5 = 12.0 |
| 2 | | 5.1 + 7.5 = 12.6 |
| 3 | IMPEDED | 5.6 + 7.5 = 13.1 |
| 4 | | 6.2 + 7.6 = 13.8 |
| 5 | | 6.7 + 7.9 = 14.6 |
| 6 | | 7.3 + 8.3 = 15.6 |
| 7 | CONSTRAINED | 7.8 + 8.7 = 16.5 |
| 8 | | 8.4 + 9.2 = 17.6 |
| 9 | | 8.9 + 10.2 = 19.3 |
| 10 | | 9.5 + 10.4 = 19.9 +? |

Source: Reference 4, page 115

This data can be compared to the results of the discharge study by multiplying the accumulation in persons per foot by the effective width of the crosswalk. Typical crosswalk widths for the high volume intersections studied in Washington, D.C. were generally between 15 and 20 feet. It should be noted, however, that the width actually used by pedestrians was not normally constrained to the painted crosswalk lines but often expanded beyond those boundaries sometimes to over 30 feet. The critical queue size value of 24 pedestrians found in this study would relate to an accumulation of 1.2 pedestrians per foot on a 20 foot crosswalk. According to Oeding's formula, 6 seconds of starting time would be required, reasonably close to the value of 7 seconds found in this study for a 24 person queue. The 9-second starting time value cited in the paragraph above would correspond to upwards of 60 pedestrians waiting at the crosswalk. This volume of pedestrians is much higher than was found at any of the intersections studied. There were few pedestrian queues greater than about 20 pedestrians at the locations observed even during peak pedestrian hours.

The highest hourly pedestrian volume count found for the District of Columbia was approximately 2700 pedestrians per hour. This translates into approximately 30 pedestrians per cycle in each direction on the crosswalk assuming a 50/50 split. Even if these 30 pedestrians were all in the initial queue at the beginning of the WALK interval (an unlikely situation), it would still relate to less than 2 persons per foot of crosswalk width. Using Oeding's formula, this would put the average starting time in the range of 7 seconds, still within the minimum recommended by the MUTCD.

In summary, fairly good agreement is achieved between starting times found in this study and the formula derived by Oeding. If anything, WALK time requirements computed by Oeding's method are less than those found here for the higher volume ranges. The major point of emphasis is that the volume levels required to create starting times in the higher ranges shown in Table 5 will occur at few locations and be concentrated in the nation's large urban areas.

## Conclusions and Recommendations

The following conclusions have been attained as a direct result of this research:

- A queue of more than approximately 24 persons must be present before the minimum WALK of 7 seconds will be exceeded.

- An average queue of 13 persons must be present before the 24 person limit will be exceeded a significant number of times.

- Average queue size can be estimated from a knowledge of hourly pedestrian volumes and signal timing.

- Queues requiring more than 7 seconds to discharge occur very rarely and will usually be found only in certain sections of large metropolitan areas. It is estimated that 99.7 percent of signalized intersections in the United States do not require minimum WALK intervals longer than 7 seconds.

- The minimum WALK interval under low volume conditions (less than 10 pedestrians per cycle) could possibly be lowered to 4-5 seconds but the importance of the inattentiveness factor should also be weighed in this decision. In addition, lowering the interval will only be practical where it is desired to reduce the length of the phase and thereby help to alleviate traffic congestion on the opposing phase.

It appears from this study that the 7-second interval now used is long enough to accommodate pedestrian queues at the vast majority of locations and under most conditions. Only in several major cities such as New York and Chicago will a large number of intersections be affected. In these situations the methodology presented here will assist the traffic engineer in applying these principles. It can be used to identify locations at which an extension of the WALK interval should be considered. The exact length of the extension must be determined on a site specific basis using a field study of peak hour conditions.

Shortening the minimum WALK interval may be applicable where pedestrian time is the determining factor in the length of a signal phase. It would permit the additional 2 or 3 seconds to be added to the major approach phase, which could hold significant benefit when that approach has heavy queue buildups. It would likely allow capacity to be increased by one vehicle per lane per cycle, yielding a substantial increase when multipled over one hour. As stated, however, this would be advisable only for volumes of under 10 pedestrians per cycle and where increasing vehicle capacity is a major concern. Field observation should be performed to verify that these conditions are met. Where the WALK interval is shortened, it is suggested to supplement the signal with an auditory tone sounded at the beginning of the interval to decrease the probability that an inattentive pedestrian will miss the display. Additional research should be performed to determine the effectiveness of this particular application.

## MINIMUM CLEARANCE INTERVAL (FLASHING DONT WALK TIME)

### Study Approach

The pedestrian clearance interval is intended to allow persons who have started to cross the street to have adequate time to complete their crossing before the onset of conflicting vehicular traffic. The MUTCD states existing policy for timing the clearance interval.

> "A pedestrian clearance interval shall always be provided where pedestrian signal indications are used. It shall consist of a flashing DONT WALK indication. The duration should be sufficient to allow a pedestrian crossing in the crosswalk to leave the curb and travel to the center of the farthest traveled lane before opposing vehicles receive a green indication (normal walking speed is assumed to be 4 feet per second)."

Many studies of pedestrian walking speeds have been made in the past. Most have dealt with percentiles and confidence limits of individual pedestrian walking speeds. The approach taken in this study was to relate speeds to platoons of pedestrians as they cross a street. A platoon is defined as consisting of the pedestrians which were in the initial queue at the beginning of WALK, but have now begun their crossing. There are two primary advantages to using platoon, rather than individual walking speeds. First, it enables a relationship to be established between walking speed and platoon size (volume). Second, it is more directly applicable to signal timing procedures since, in many cases, more than one pedestrian will be crossing at a time.

The effective length of the crosswalk also has an impact on the length of the clearance interval. The major variables involved, other than street width, are the subtraction of half of the width of the farthest traveled lane from the distance crossed and the utilization of the near and far-side parking lanes by pedestrians and vehicles. As part of this study, the behavior of pedestrians on crosswalks with parking lanes was examined. For near-side parking lanes, it was noted to what degree pedestrians took advantage of the refuge afforded to reduce the crossing length. For far-side parking lanes, the rationale behind what constitutes a parking lane was examined (i.e. distance of parked cars from the intersection).

## Data Collection

Data collection for pedestrian platoon speeds was performed concurrently with the queue discharge observations at the same locations and during the same time intervals. The specific item of data desired was the time interval between the discharge of the last pedestrian in a queue (as described in the queue discharge study) and the arrival of the last pedestrian in the platoon at the far curb. Note that the last pedestrian to arrive is not necessarily the one that was last to discharge.

The data collection procedure is a continuation of that performed in the measurement of queue discharge times (see page 12). Each observer, having started the stopwatch at the beginning of the WALK interval, continued to watch the platoon consisting of the pedestrians in the initial queue as it crossed the street. Once the last pedestrian in the platoon had stepped onto the far curb, the watch was stopped and the time recorded. The discharge time of the queue, which had been obtained previously, was then subtracted from this value to obtain the clearance time of the platoon. Note that the speeds observed were not averages of all individuals in the platoon but were speeds of the slowest part of the pedestrian platoon.

The second phase of the data collection procedure pertained to the inclusion or deletion of parking lanes in the clearance interval computation. Several sites were selected in Washington, D.C. for testing pedestrian behavior at near-side parking lanes. The locations selected were:

- Crossing 16th Street North of L Street
- Crossing 17th Street North of L Street
- Crossing 16th Street South of K Street

Each location had parallel vehicular parking along the curb during the data collection period. The cars were 10 feet from the crosswalk at location 1; 30 - 60 feet at location 2; and 5 feet at location 3. All crosswalks were equipped with pedestrian signals. Data were taken during time periods when pedestrian volumes were relatively low. This was necessary to insure that all pedestrians had a free choice of whether or not to make use of the parking lane in crossing.

Two observers were used, one on each end of the crosswalk. They were instructed to count the number of pedestrians leaving from the curb and the number leaving from the edge of the parking lane during each cycle. Samples of 10 cycles were taken at each location. General observations of pedestrian and vehicle use of far-side parking lanes were also made at these and other locations but no actual data was collected.

No field data collection was undertaken to evaluate the "middle of the farthest traveled lane" assumption. This is more of a judgmental matter than one which lends itself to empirical evaluation. At any rate, this factor can be accounted for in the speed assumed in the clearance time computation.

## Data Analysis

### Platoon Walking Speeds

The item of most interest in the determination of the minimum clearance interval was the assumed walking speed. Over 1400 speed samples of pedestrian platoons of various sizes were available from the field observations. Cumulative distributions of these speeds are shown for two-hour sets of data for each city (Figures 5-7). Table 6 shows the percentage of speeds slower than 4 ft./sec. for each hour of data and presents the associated volume characteristics.

In general, the percentage of platoon speeds slower than 4.0 ft./sec. increased as pedestrian volumes increased. This data was particularly consistent in Washington and Phoenix. In Buffalo, however, the data was extremely inconsistent and did not support this trend. It would appear that the extremely cold weather and the poor compliance of pedestrians with the signals was the major contributing factor.

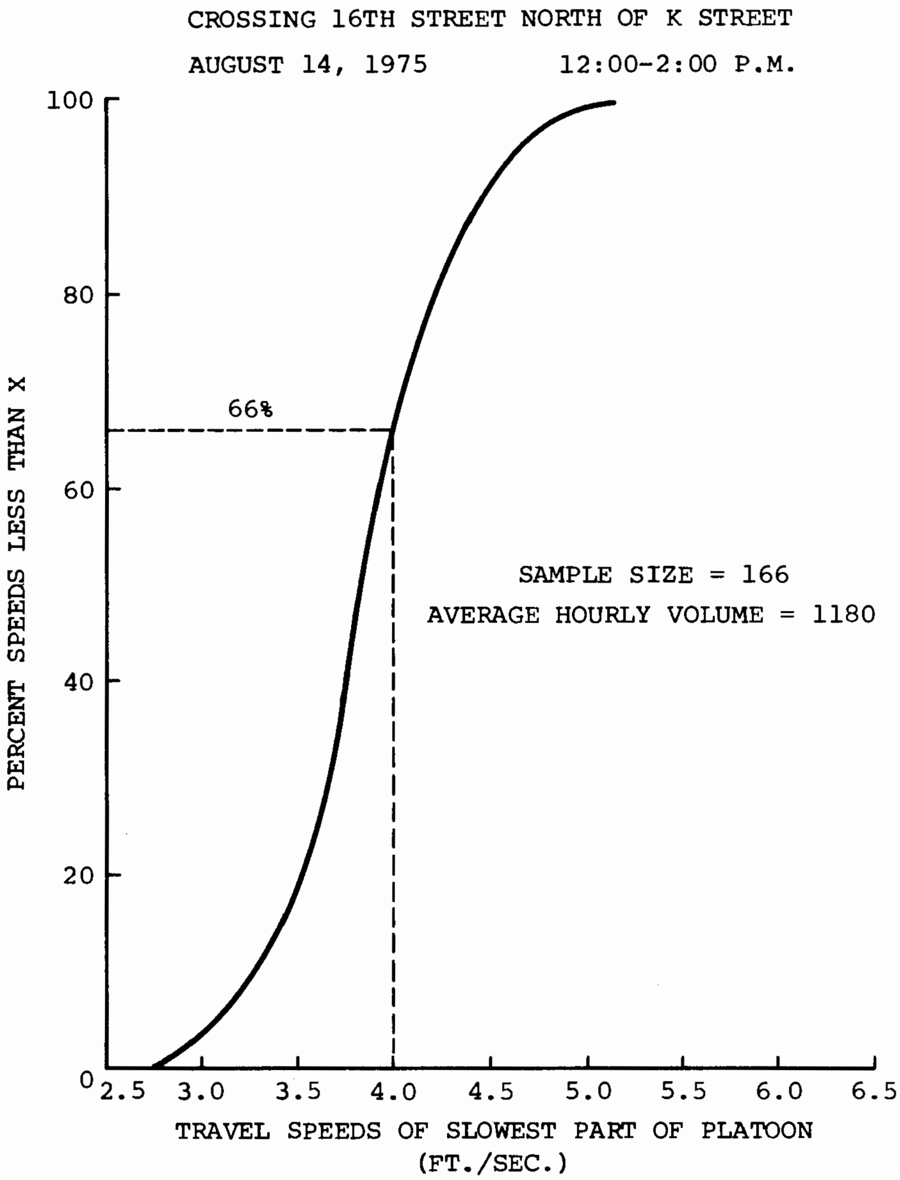


FIGURE 5. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS IN WASHINGTON, D.C.

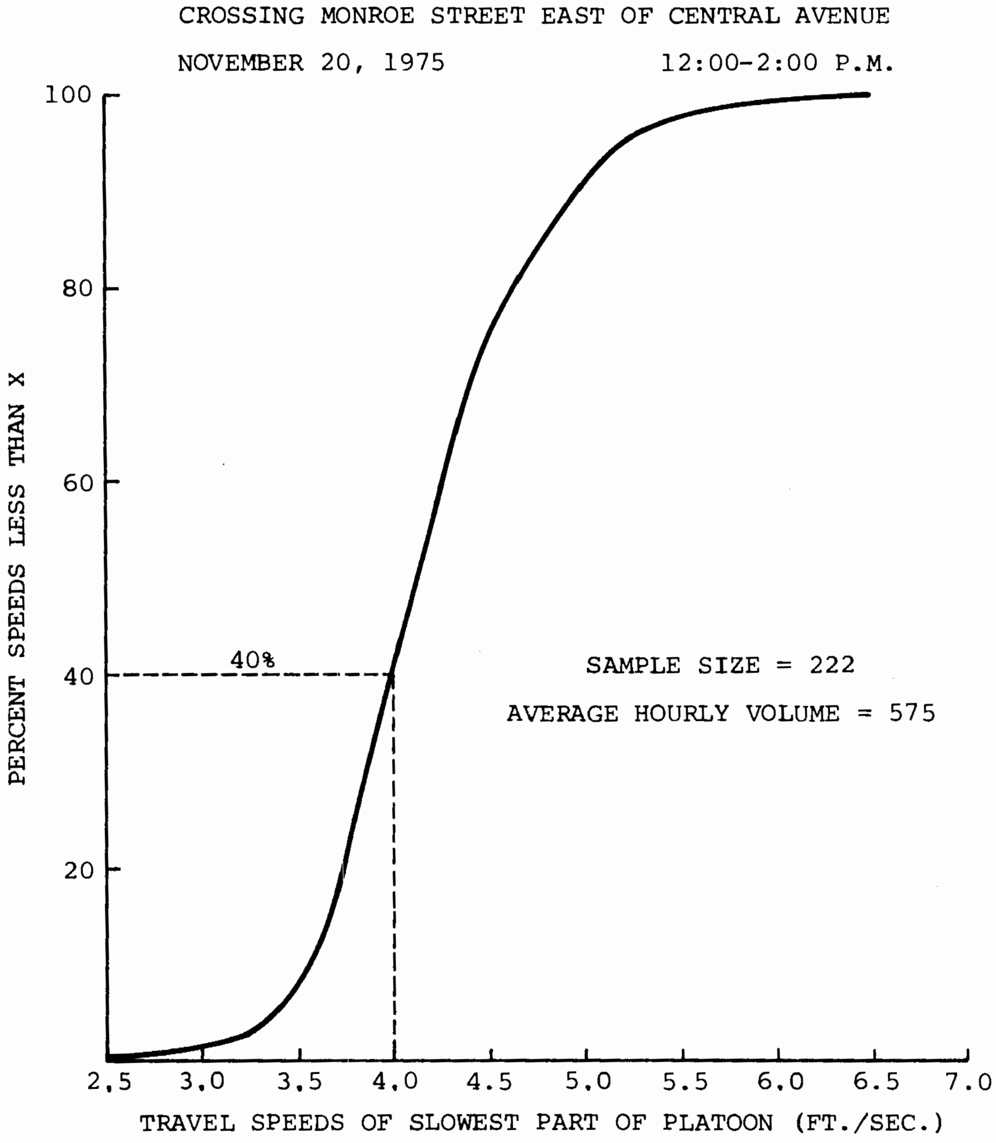


FIGURE 6. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS IN PHOENIX.

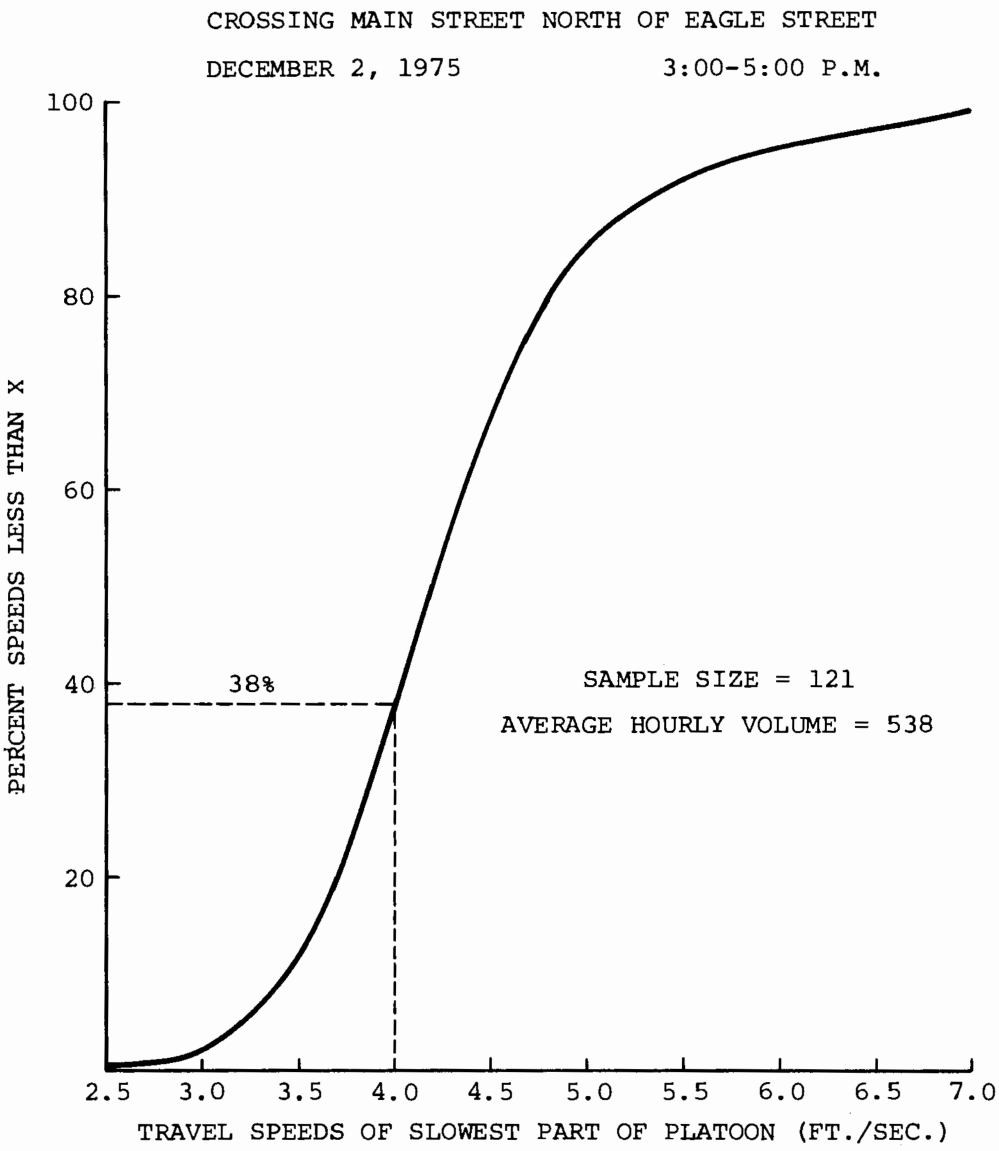


FIGURE 7. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS IN BUFFALO.

# TABLE 6. PERCENT SPEEDS LESS THAN 4.0 FT./SEC.

| Intersection | Time | Total Pedestrian Volume | Total Peds in Platoons | Average Number in Platoon* | Percent Platoon Speeds less than 4.0 ft./sec. |
|---|---|---|---|---|---|
| **Washington** | | | | | |
| Connecticut & M | 12-1 | 1435 | 841 | 9.34 | 70 |
| | 1-2 | 1451 | 781 | 8.67 | 67 |
| 16th & K (8/13) | 12-1 | 1237 | 607 | 7.78 | 65 |
| | 1-2 | 1130 | 528 | 6.51 | 62 |
| | 5-6 | 932 | 486 | 6.07 | 56 |
| 16th & K (8/14) | 12-1 | 1229 | 639 | 7.43 | 68 |
| | 1-2 | 1130 | 584 | 7.12 | 65 |
| New York & 14th | 12-1 | 1177 | 796 | 8.84 | 68 |
| | 1-2 | 1018 | 692 | 7.95 | 67 |
| **Phoenix** | | | | | |
| Central & Monroe | 12-1 | 741 | 455 | 3.32 | 48 |
| | 1-2 | 409 | 337 | 2.83 | 28 |
| | 3-4 | 254 | 176 | 2.00 | 25 |
| | 4-5 | 226 | 149 | 1.75 | 20 |
| **Buffalo** | | | | | |
| Main & Eagle | 12-1 | 1341 | 585 | 6.35 | 19 |
| | 1-2 | 998 | 491 | 5.45 | 22 |
| | 3-4 | 470 | 173 | 3.14 | 45 |
| | 4-5 | 605 | 250 | 3.78 | 32 |

*Note: Average does not include instances in which there were no pedestrians in the platoon. For the lower volumes, no pedestrians may be present a significant number of times.

The percentage of platoon walking speeds slower than 4.0 ft./sec. are extremely high, particularly in Washington at the higher volume locations. Thus, it first appears that a significant number of pedestrians may not complete their crossings where only a minimum clearance interval exists. However, it should be noted that most platoons, unless they are extremely large, have left the curb before the WALK interval terminates. This gives the platoons at least several seconds of additional time in which to complete their crossing. Consequently, the percentage of pedestrians not completing their crossing in time is substantially below the percentage of platoon walking speeds slower than 4.0 ft./sec. An analysis of how many pedestrians actually complete their crossing begins on page 80.

A plot of the percentage of cycles in which there was at least one platoon walking slower than 4.0 ft./sec. versus average volume per cycle was made to determine whether a definite relationship could be established between the two variables (Figure 8). As indicated, the Buffalo data appeared to contradict the data obtained in Washington, D.C. and Phoenix. It is expected that the reason for this contradiction rests with either the very cold temperatures (below freezing), the low compliance rate (see page 80), or a significant difference in the makeup of pedestrian platoons during different parts of the data collection period (e.g., a higher percentage of elderly pedestrians during certain hours). Eliminating the Buffalo data, a straight regression line can be fit to the data points with high correlation (r = .97).

The fact that speeds of the slowest part of the platoon do decrease with an increase in pedestrian volume seems to be evident from the data regardless of the existence of the Buffalo data. A second test was devised to further identify the reason for the relationship. The three possible explanations are:

- conflicts between pedestrians moving in the same direction.
- conflicts between pedestrians moving in opposite directions
- the presence of a greater number of slow walkers in the platoon at higher volumes.

The importance of the third factor was deemed to be particularly relevant to this study in view of the way in which platoon speeds were defined. The test was therefore set

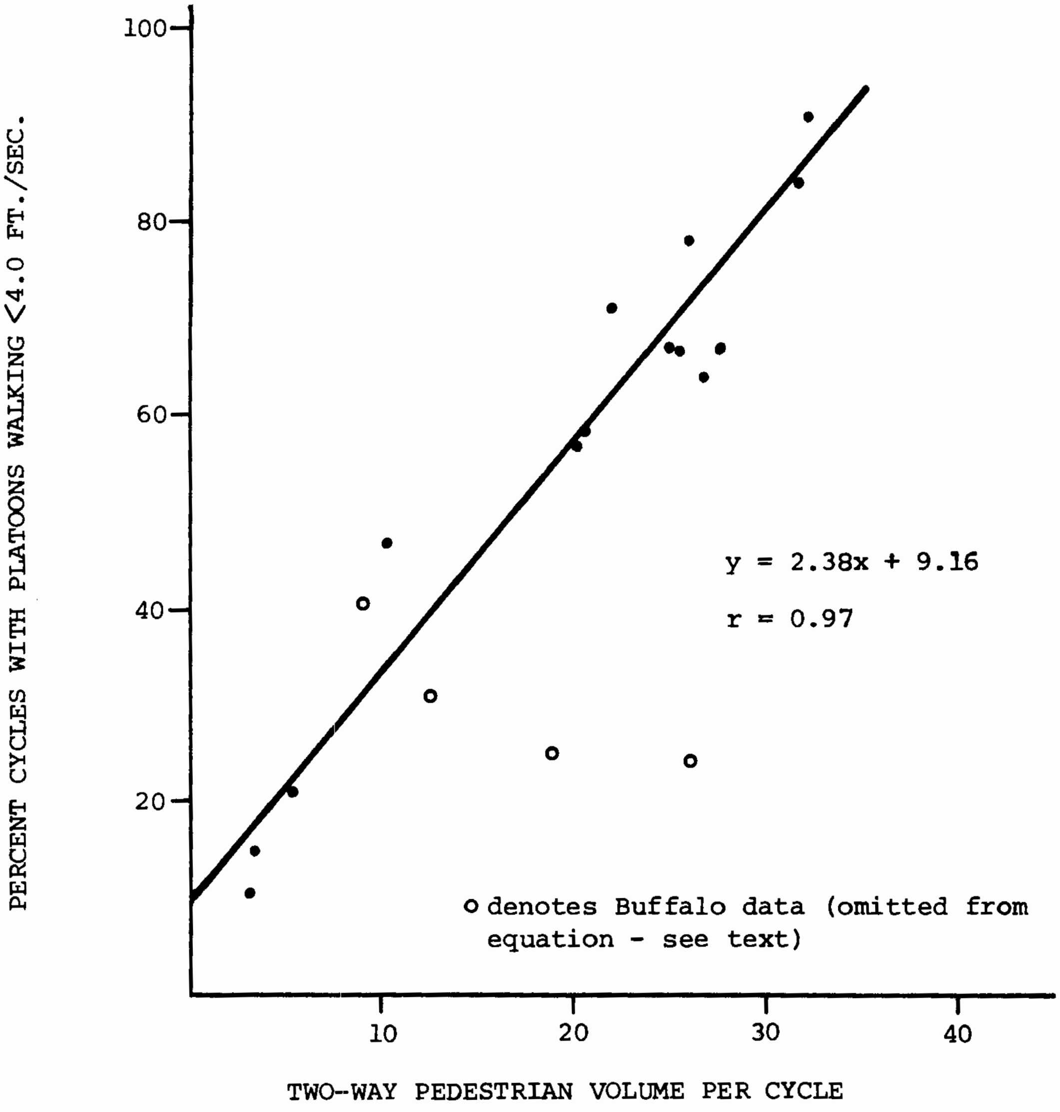


FIGURE 8. PERCENT CYCLES WITH PLATOON SPEEDS SLOWER THAN 4.0 FT./SEC. AS A FUNCTION OF PEDESTRIAN VOLUME.

up to determine whether that factor could totally explain the speed/volume relationship. If not, then it would have to be assumed that one or both of the other factors also have some influence at the volume levels observed.

First, hypothetical platoons of pedestrians were constructed from the speed distribution previously documented in Traffic Engineering by Hoel (5). Hoel's distribution of walking speeds was derived from observations of pedestrians in a central business district. The probability of there being a pedestrian walking slower than 4.0 ft./sec. can be computed for each hypothetical platoon. For instance, in Hoel's distribution, approximately 10 percent of individual pedestrians crossing intersections walk at less than 4.0 ft./sec. Therefore, given a platoon of three people, the probability that at least one of them is walking slower than 4.0 ft./sec. is one minus the probability that all three are walking at least that fast. Mathematically this would be expressed as $1 - (0.9)^3 = 0.27$. There will, of course, be some deviation from the probabilities in reality since people of similar walking characteristics often walk together (e.g., the elderly).

The next step was to compare actual platoon speeds with those theoretically derived above. Since only the Washington, D.C. data offered sufficient numbers of the higher volume platoons, the comparison was restricted to this data set. The number of platoon speeds less than 4.0 ft./sec. were summarized from the field data by platoon size as shown in column 3 of Table 7. Next, the number of platoons slower than 4.0 ft./sec. which would be expected given that Hoel's distribution holds true, was established. A chi-quare goodness of fit test was used to determine whether the observed and theoretical distributions were significantly different. A significant difference would imply that there was some factor affecting platoon speeds other than merely the probability of slower walking pedestrians. The outcome of the test showed that the distributions were not significantly different, which suggests that conflicts between pedestrians moving in either the same or opposing directions has no direct effect on platoon speeds on crosswalks at the volume levels observed. Higher volume levels may introduce these additional impacts on platoon speed.

Observational experience also suggested that these factors had little influence. First, pedestrians tend to spread out longitudinally as they leave the curb in order to allow a comfortable distance for walking strides. It appears that densities on the crosswalk itself are seldom so high as to lower the walking speed of a pedestrian. The

## TABLE 7. CHI-SQUARE TEST OF OBSERVED AND EXPECTED PLATOON SPEEDS.

| Platoon Size | Number Platoons Observed | Number of Platoons with Speeds <4 ft./sec. | Expected Number <4 ft./sec. From Hoel's distn. | $X^2$ |
|---|---|---|---|---|
| 1 | 32 | 2 } 9 | 3 } 11 | .364 |
| 2 | 42 | 7 | 8 | |
| 3 | 37 | 11 | 10 | .100 |
| 4 | 64 | 26 | 22 | .727 |
| 5 | 57 | 24 | 23 | .043 |
| 6 | 69 | 40 | 32 | 2.000 |
| 7 | 48 | 26 | 25 | .040 |
| 8 | 68 | 35 | 39 | .410 |
| 9 | 51 | 29 | 31 | .129 |
| 10 | 46 | 23 | 30 | 1.633 |
| 11 | 33 | 23 | 23 | 0.000 |
| 12 | 51 | 32 | 37 | .676 |
| 13 | 31 | 25 | 23 | .174 |
| 14 | 31 | 15 | 16 | .063 |
| 15 | 18 | 16 | 14 | .286 |
| 16 | 12 | 9 | 10 | .100 |
| 17 | 6 | 5 | 5 | 0.000 |
| 18 | 9 | 9 | 8 | .125 |
| 19 | 6 | 5 | 5 | 0.000 |

Degrees of Freedom = (2-1) (18-1) = 17 $\sum = 6.870$

$X^2_{.05}$ (1-tailed) = 27.59 > 6.870

∴ difference is not significant

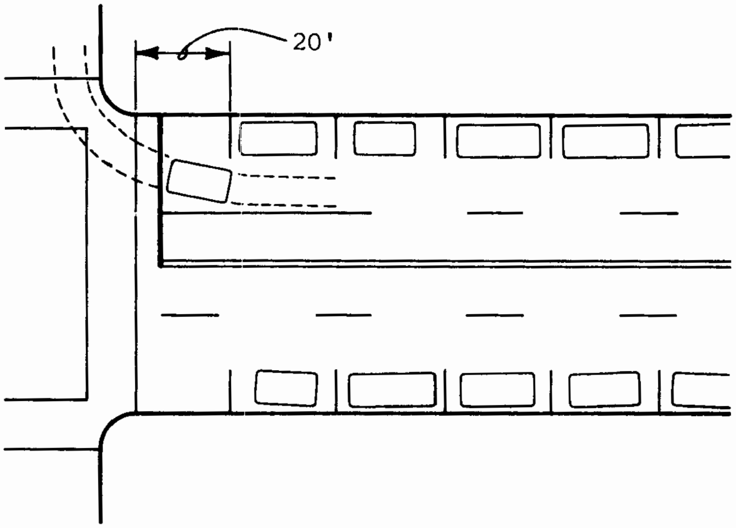
second observation was that pedestrians can readily avoid conflict with those proceeding in the opposite direction. This is done either by walking outside the crosswalk boundaries or by the pedestrians walking behind one another in a platoon. Past research has shown that volume and opposing flow do have an effect on speed at locations where walkway width is restricted, but a laterally unrestricted walkway such as an intersection crosswalk apparently allows free movement regardless of these factors. The factors may have more effect on crosswalks with higher volumes than observed in this study, but again, crosswalks with higher volumes are rare.

### Use of Parking Lanes

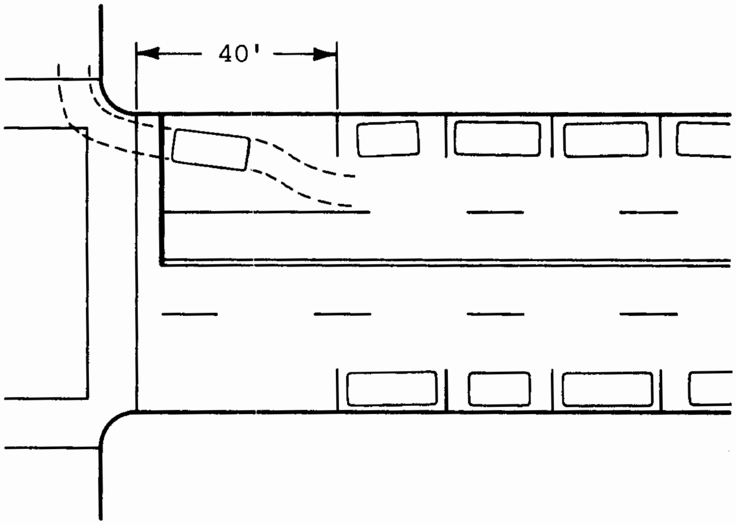
The observation of the use of parking lanes indicates that pedestrians tend not to use the area protected by parked vehicles at the beginning of their crossing. This was true whether parked vehicles were immediately adjacent to the crosswalk or whether they were some distance away. This study found that over 70 percent of the pedestrians leave from the curb even when this additional protected area is available.

The question which arises with respect to the far-side parking lane is whether it should be considered as a "traveled lane" under certain conditions. In observing pedestrian and vehicle behavior at far-side parking lanes, it was noticed that having cars parked along the curb does not guarantee that a corresponding length of crosswalk will not be infringed upon by vehicles. For instance , when a vehicle approaching an intersection with parking is turning right, his path normally takes him closer to the curb as soon as he has passed the last parked vehicle. Figure 9 shows typical paths for right turning vehicles at intersections with parking at various distances from the crosswalk. The farther the parking is from the crosswalk, the more that lane is likely to become a utilized lane at the intersection.

It is evident from Figure 9 that the parking lane will almost always be a "traveled lane" and that it must be considered in the crosswalk length. The only exception will be where right turns cannot be made because of either geometric or operational constraints (e.g., one-way streets). If a clearance interval is timed to exclude the far-side parking lane from the crosswalk length, a pedestrian walking at 4 ft./sec. could conceivably be caught having to cross 1½ lanes after the onset of the opposing phase.



PARKING 20 FEET FROM CROSSWALK



PARKING 40 FEET FROM CROSSWALK

FIGURE 9. TYPICAL PATHS FOR RIGHT TURNING VEHICLES AT INTERSECTIONS WITH PARKING.

## Review of Related Data

The previously cited reference by Pushkarev and Zupan (4) also contains substantial information on pedestrian crossing times. The authors have divided the crossing time into three components:

- the time needed for the two platoons to walk up to each other
- the time needed to penetrate each other
- the time to walk the rest of the distance to the curb on the opposite side of the street.

Citing an analysis by Oeding, the authors summarize the crossing time by stating:

> "... crosswalk speeds seem to be at or above free flow levels when the sum of the two relative sidewalk accumulations is less than 1 person per foot of crosswalk width. As total accumulation rises from 1 to 3 persons per foot of width, average crossing speed drops from the neighborhood of 270 to about 200 feet per minute and then stays at that level up to a total accumulation of about 6 persons per foot."

Crossing times for various pedestrian volumes and street widths are shown in Table 6, presented earlier and taken from Pushkarev and Zupan.

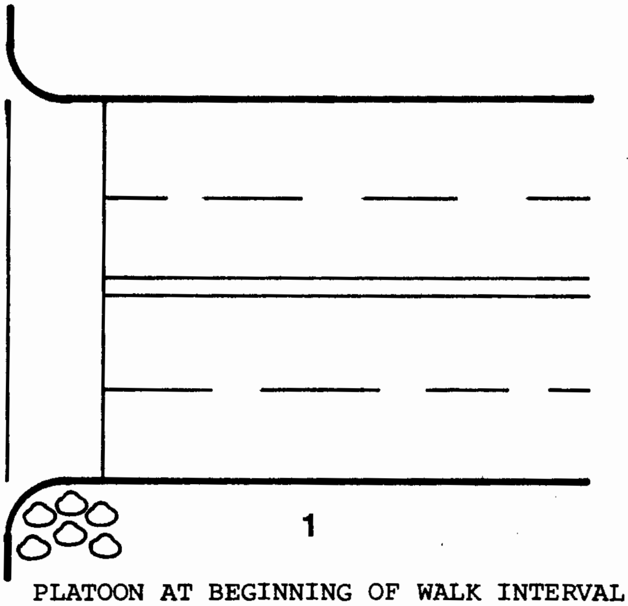
The first point that should be made in comparing these results with those of this study pertains to the definition of crossing speeds. Oeding has apparently used an "average" crossing speed while the speed of the slowest part of the pedestrian platoons has been used in this research. It is the latter definition which is of most value to the traffic engineer since the slowest part of the platoon is the basis on which the clearance interval should be timed. Pushkarev and Zupan do not include a discussion of how the average crossing time was determined.

The average platoon sizes observed in this study were somewhat less than the volume required to produce noticeable decreases in speed. Average platoon sizes were generally less than 10 pedestrians, which on a 20 foot crosswalk, equates to 0.5 accumulation at each end of the crosswalk or a total accumulation of 1 person per foot. As can be seen from Table 7, even the larger platoons (ranging to almost 2 persons per foot total accumulation) did not encounter a significant decrease in speed other than what would normally be expected due merely to the presence of more slow walking pedestrians in the platoon. Again, this pertains to the speed of the slowest part of the platoon rather than to an average. It is possible that the average speed may have decreased while the speed of the slowest part may have remained nearly the same.

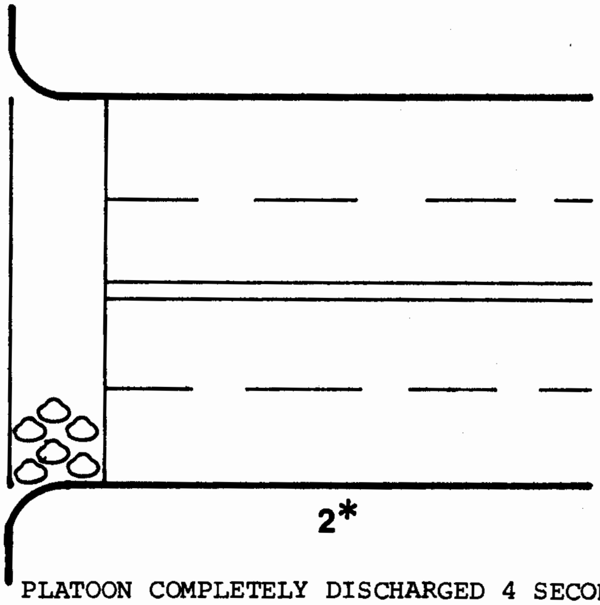
In summary, it is viewed that the two sets of data discussed above are in general agreement, although they cannot be precisely compared because of differences in the way that the components have been defined. The major point made here is that there are very few instances in which pedestrian volumes at intersections will be so high as to fall into the "impeded" or "constrained" ranges defined by Pushkarev and Zupan. These cases will be confined to a few major cities, and even there only in limited locations and during limited hours.

### Effect of the Results on the Timing of the Clearance Interval

The results of the platoon speed studies bring to light a very significant question, that is, "When should a walking speed slower than 4.0 ft./sec. be assumed?" At first glance, it appears that at intersections where anything greater than small pedestrian volumes exist, a significant number of those pedestrians are not given adequate time to cross the street. However, as shown in the discharge time study, the 7 seconds of the minimum WALK interval will be fully utilized by the initial queue only for very heavy pedestrian volumes (see Table 3). For the lower volumes (less than 10 pedestrians per cycle) discharge time will usually be less than 5 seconds. In effect, this means that a platoon discharging in less than 7 seconds has extra time beyond the normal clearance interval to complete the crossing (See Figure 10). In most cases, this extra time is enough to permit those pedestrians walking slower than 4.0 ft./sec. to finish their crossing before the solid DONT WALK is displayed. The only pedestrians who are in danger of not finishing in time are usually those who begin their crossing at the end of the WALK interval and walk slower than 4.0 ft./sec. The number of pedestrians beginning their crossing near the end of the WALK interval is very small except where discharge times are frequently equal to or greater than the time allowed for the WALK interval.



PLATOON AT BEGINNING OF WALK INTERVAL.



PLATOON COMPLETELY DISCHARGED 4 SECONDS LATER.

*(Note that, in no. 2, the platoon has 3 additional seconds (7 minus 4 seconds) which can be used as clearance time over and above the regular clearance interval. This is enough additional time to permit most pedestrians walking slower than 4.0 ft./ sec. to complete their crossing in time. Refer to text for further discussion.)

FIGURE 10. ILLUSTRATION OF WHY SLOW WALKERS USUALLY COMPLETE THEIR CROSSING BEFORE THE SOLID DONT WALK IS DISPLAYED.

It is a difficult matter to establish a volume threshold which, when exceeded, would mandate timing the clearance interval at 3.5 rather than 4.0 ft./sec. A reasonable value would appear to be an average of 10 pedestrians per cycle during a given hour. At this volume level, discharge times will seldom exceed 4-5 seconds, giving the pedestrians in the platoon an extra 2-3 seconds of crossing time (i.e. with a minimum 7-second WALK interval) in addition to the normal minimum clearance interval. At an hourly volume of more than 15 pedestrians per cycle, a significant percentage (over 5 percent) of discharge times may exceed 5 seconds. This will allow less than 2 seconds of the WALK interval to be applied toward the clearance time, which will not permit those pedestrians walking at 3.5 ft./sec. to complete their crossing by the time that the opposing vehicular green phase begins. Timing the clearance interval for the case where the WALK interval is greater than 7 seconds is discussed on page 92.

## Conclusions and Recommendations

The data collected for this phase of the research has yielded a number of significant results. The major conclusions implied by these results are listed below:

- The percentage of pedestrian platoons walking slower than 4.0 ft./sec. is quite high on high volume crosswalks, and ranged well over 50 percent for some of the crosswalks observed in this study.
- A clearance interval based on a pedestrian walking speed of 3.5 ft./sec. should be considered at locations with peak hour pedestrian volumes of over 15 per cycle.
- Neither platoon volume nor opposing pedestrian flow appear to have a significant effect on pedestrian platoon walking speeds at intersections with pedestrian volume levels less than approximately 30 pedestrians/cycle.
- Pedestrians tend not to use the near-side parking lane as a protected area for beginning their crossing.

- Far-side parking lanes should be considered as "traveled lanes" unless geometrics or operational constraints preclude pedestrian/vehicle conflicts in that lane.

Some of the conclusions listed above present an added dilemma to the traffic engineer. Where the use of a slower walking speed assumption lengthens minimum phase requirements, it may be necessary to take green time away from the opposing phase to maintain the same cycle length. If taking this time away will either create or increase traffic congestion, it may be wise to use the 4.0 ft./sec. walking speed assumption. However, there should be few cases in which the 3.5 ft./sec. speed will be warranted, and using that speed will create traffic congestion. The slower speed will usually be warranted only in the central business districts of large metropolitan areas.

## ALLOCATION OF EXCESS PEDESTRIAN TIME

### Study Approach

At most signalized intersections with cycle lengths greater than 60 seconds, the time for each phase normally exceeds minimum pedestrian time requirements. This normally means that the WALK or clearance intervals can be extended beyond the minimum required. Wide major streets with low vehicular volumes on the minor street will be the most prevalent case where no extension time is available. The purpose of this phase of the study was to develop a methodology to determine how to allocate any excess time which is available in the phase between the WALK and clearance intervals.

The two most important factors in the allocation of excess pedestrian time are delay and safety. Pedestrian delay is incurred by pedestrians having to wait for a permissive signal indication. Vehicle delay, beyond that which is normally introduced by the signal, is incurred when turning vehicles encounter pedestrians in the crosswalk. The emphasis in this study is only on right turn delay, as left turn pedestrian-caused delay is less severe and not as easily quantified. The safety factor is viewed from the standpoint of the pedestrian.

It was hypothesized prior to the commencement of the analysis that the advantages of allocating the excess time to the WALK interval would be reduced pedestrian delay and improved compliance to the indications. The primary advantage envisioned for allocating the time to the clearance interval was the possible reduction in vehicular delay due to fewer pedestrian conflicts. The approach to this problem included the consideration of both pedestrian and vehicle delays.

## Data Collection

### Vehicle and Pedestrian Delay Data

The approach taken in the evaluation of vehicle and pedestrian delays was primarily mathematical. However, it was found that insufficient information was available on which to base vehicular right turn delay. Consequently, a preliminary study of vehicle delay was undertaken using time lapse photography. The data were collected from films taken in Washington, D.C., Phoenix, Arizona, and Cambridge, Massachusetts. Table 8 lists the locations, dates, and times of filming. Films from previous studies in Akron, Ohio and Washington, D.C. were also used (references in Table 8). In total, approximately 68 approach-hours of film were reviewed.

The selection of sites for filming was based on the following prerequisites:

- little or no skew at the intersection
- street widths ranging from approximately 35 to 80 feet
- pretimed signal control
- existence of pedestrian signals
- pedestrian volumes ranging between approximately 1 and 30 pedestrians per cycle
- heavy right turn vehicular volumes
- good pedestrian compliance
- adequate location from which to film

Obviously all of the criteria could not be perfectly satisfied at every location. However, it is felt by the research team that the locations selected form a good cross-section

## TABLE 8. DELAY STUDY FILMING LOCATIONS.

| City | Intersection | Date and Approx. Time Filmed | Number Approaches Observed |
|---|---|---|---|
| Washington, D.C. | 17th St. & Pennsylvania Ave. (from a previous study)* | 3/12/74<br>1:00-2:00 PM | 2 |
| Washington, D.C. | 14th St. & H St. | 10/23/75<br>10:00-2:00 PM | 4 |
| Washington, D.C. | 17th St. & H St. | 10/24/75<br>10:00-2:00 PM | 4 |
| Washington, D.C. | 14th St. & Constitution Ave. | 10/25/75 (Sat.)<br>1:00-2:00 PM | 4 |
| Cambridge, Mass. | Massachusetts Ave. & Prospect Rd. | 10/20/75<br>4:15-5:15 PM<br>10/21/75<br>12:00-1:00 PM | 3<br>3 |
| Phoenix, Ariz. | Central Ave. & Van Buren St. | 11/18/75<br>10:30-1:30 PM | 4 |
| Phoenix, Ariz. | Central Ave. & Monroe St. | 11/19/75<br>8:00 AM - 12:00 Noon | 2 |
| Akron, Ohio | S. Main St. & E. Bowery Street (from previous study)** | 12/18-12/19/72<br>Various times | 2 |

*Allen, J.C., J.L. Kay and J.M. Bruggeman, Evaluation of UTCS/BPS Control Strategies, prepared for FHWA, February, 1975.

**Berger, W.G., Urban Pedestrian Accident Countermeasures Experimental Evaluation, prepared for NHTSA and FHWA, February 1974 of the types of intersections where vehicle right turn delay would be of concern. All but one of the locations selected were intersections of two-way streets.

The following data were extracted from the films on a cycle-by-cycle basis:

- two-way pedestrian volume
- number of vehicles per cycle in right lane
- number of vehicles turning right
- time to the nearest second that the front bumper of a vehicle crossed the near side of the approach crosswalk ($t_1$)
- time to the nearest second that the rear bumper of the behicle crossed the far side of the exit crosswalk($t_2$)

Figure 11 graphically illustrates the $t_1$ and $t_2$ definitions.

Table 9 shows a sample of the form used for collecting the data and several cycles of actual data observed at 17th Street and Pennsylvania Avenue in Washington. Each $t_1$ and $t_2$ value describes the activity of one vehicle. When the vehicle was not a right turning vehicle, no $t_2$ value was recorded. Thus, the percentage of right turns could be computed by dividing the number of entries with a $t_2$ value by the total number of entries. Note that this is the right turn percentage for the right lane only. The above data yielded a very complete history of the times taken by each turning vehicle to complete the right turn maneuver. These delays could then be related to pedestrian volumes on the conflicting crosswalk.

One other brief study was required for use in the determination of pedestrian delay. It dealt with the arrival rates of pedestrians at an intersection crosswalk and consisted merely of counting the number of pedestrians arriving at a particular crosswalk during 5-second increments within each signal cycle. Eight crosswalks were observed for at least 20 minutes each to form the data base.

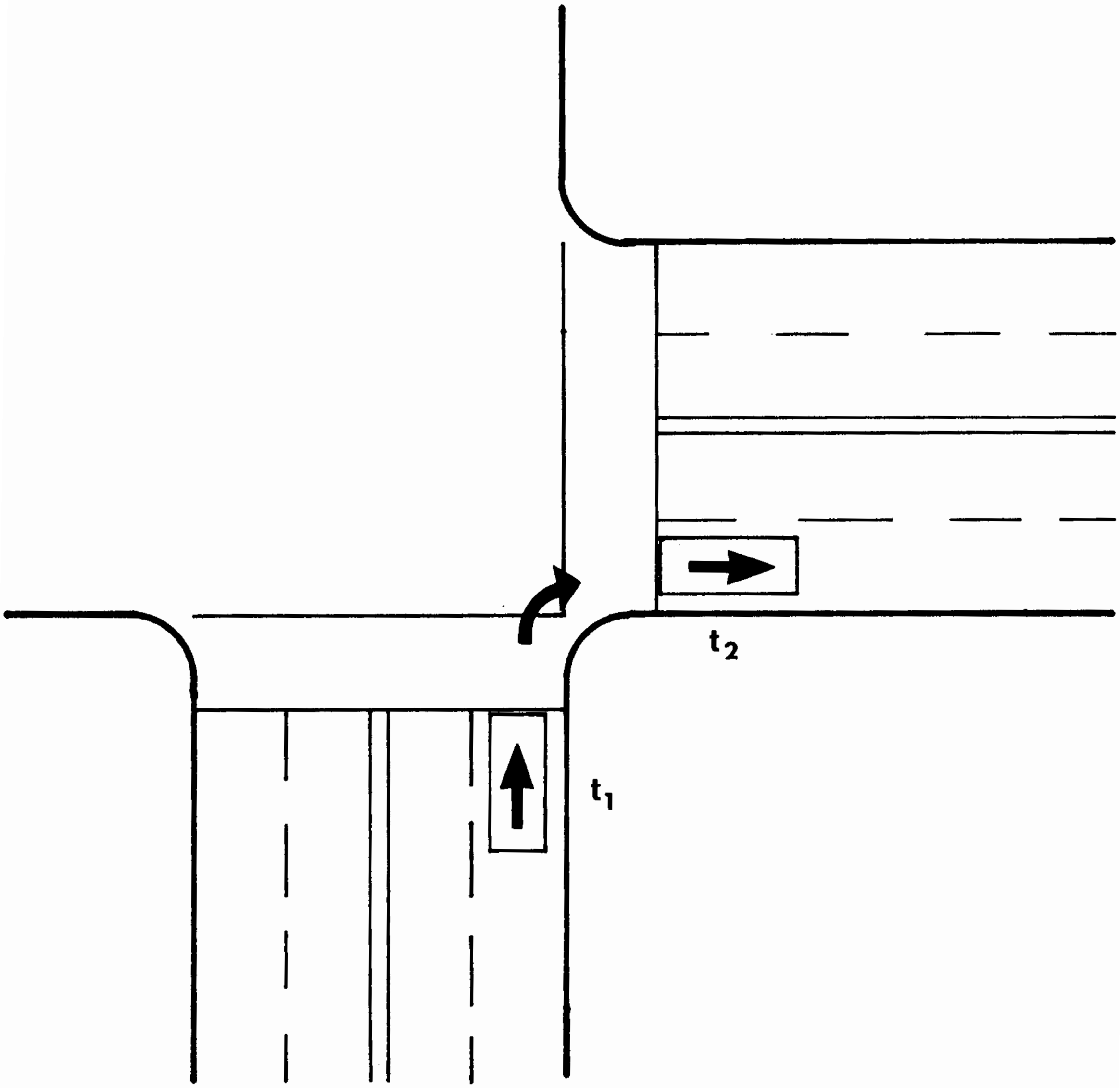


FIGURE 11. ILLUSTRATION OF $T_1$ AND $T_2$ DEFINITIONS.

# TABLE 9. SAMPLE RAW DATA FORM FOR DELAY ANALYSIS.

| Cycle No. | 2-way Pedestrian Volume | Vehicle Number | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| | | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ | $t_1/t_2$ |
| 1 | 10 | 0/ | 4/8 | 6/11 | 11/18 | 16/21 | 19/ | 22/ | 24/ | 27/ | 28/ |
| 2 | 9 | 0/ | 7/ | 11/15 | 14/28 | 20/31 | 28/35 | | | | |
| 3 | 16 | 0/ | 7/ | 9/14 | 12/24 | 21/ | 25/29 | 28/33 | | | |
| 4 | 18 | 0/9 | 4/ | 8/12 | 11/15 | 13/23 | 16/25 | 28/38 | | | |
| 5 | 10 | 0/8 | 6/ | 9/13 | 12/ | 14/ | 21/ | 25/29 | 27/31 | 30/33 | |
| 6 | 9 | 0/9 | 3/ | 7/24 | 11/25 | 23/28 | 27/31 | 30/36 | 32/38 | | |
| 7 | 17 | 0/8 | 6/ | 9/ | 13/ | 15/31 | 19/33 | 31/37 | | | |
| 8 | 29 | 0/6 | 6/12 | 9/ | 11/34 | 15/36 | | | | | |
| 9 | 11 | 0/17 | 16/25 | 23/30 | 30/ | | | | | | |
| 10 | 25 | 0/15 | 5/33 | 16/37 | | | | | | | |
| 11 | 14 | 0/8 | 5/11 | 7/13 | 10/16 | 13/ | 16/ | 19/ | 21/ | 24/ | |
| 12 | 12 | 0/7 | 5/10 | 8/12 | 10/14 | 13/17 | 17/27 | 26/ | 29/33 | | |

Note: Data taken from vehicles eastbound on Pennsylvania Avenue turning southbound onto 17th Street, Washington, D.C.

$t_1$ = time to nearest second that the front bumper of a vehicle crossed the near side of the approach crosswalk.

$t_2$ = time to nearest second that the rear bumper of the vehicle crossed the far side of the exit crosswalk (no $t_2$ value indicates a thru vehicle).

## Pedestrian Compliance and Behavior Data

The third field study performed for determining the best allocation of excess pedestrian time related to the safety factor. To observe the safety impacts of allocating excess pedestrian time in different ways, 6 crosswalks were selected in Washington, D.C.; Phoenix/Tempe, Arizona and Buffalo, New York at which to vary the allocation of the WALK and clearance intervals from one extreme to the other. The locations selected and timing alternatives used are shown in Table 10. The criteria used for selecting the crosswalks are given below:

- range of street widths
- 10 or more seconds of time difference between minimum pedestrian time and total phase time
- pretimed signal control
- moderate to heavy pedestrian volumes
- clearly visible pedestrian signals

Two of the crosswalks selected traversed one-way streets. At Monroe Street in Phoenix, the exit leg crosswalk was observed and at 20th Street in Washington, D.C. the approach crosswalk was observed. The remaining locations were intersections of two 2-way streets.

At each site the solid DONT WALK time was held constant while the other two intervals were varied. The standard 7 second minimum WALK with the remaining time allocated to the clearance interval was used as one alternative and a minimum clearance interval timed using the 4.0 ft./sec. walking speed assumption was used as the opposite extreme for a second alternative. In one case, 15th and H Streets, the clearance interval could not be timed as precisely as desired due to limitations of the controller. In Phoenix and Buffalo, one additional alternative was selected in the median range between the two extremes. In Washington, D.C. two mid-range alternatives were used. The first alternative used was always the one closest to existing timing so that drastic changes in timing would be avoided. The result was that in three of the six cases, alternatives were implemented chronologically from minimum WALK to minimum clearance and for the other three the process was reversed. It was felt that this might reveal any difference in pedestrian behavior caused by changing timing in the two different ways. For each location, at least one week was allowed between the change of timing for the next alternative and the performance of

# TABLE 10. LOCATIONS SELECTED FOR STUDYING THE ALLOCATION OF EXCESS PEDESTRIAN TIME.

| City | Crosswalk | Curb-to-Curb Length (ft.) | Dates Observed | Interval Times (sec.) W | Clear | SDW |
|---|---|---|---|---|---|---|
| Washington, D.C. | Crossing H St. West of 15th St. | 57 | 11/12/75 | 7.2 | 37.6 | 35.2 |
| | | | 11/20/75 | 15.2 | 29.6 | 35.2 |
| | | | 12/2/75 | 21.6 | 23.2 | 35.2 |
| | | | 12/16/75 | 29.6 | 15.2 | 35.2 |
| Washington, D.C. | Crossing 20th St. South of M St. | 48 | 11/14/75 | 7.2 | 36.8 | 36.0 |
| | | | 11/21/75 | 16.0 | 28.0 | 36.0 |
| | | | 12/3/75 | 24.0 | 20.0 | 36.0 |
| | | | 12/17/75 | 32.8 | 11.2 | 36.0 |
| Phoenix, Ariz. | Crossing Monroe St. West of Central Ave. | 52 | 11/18/75 | 18.5 | 11.5 | 20.0 |
| | | | 12/3/75 | 13.0 | 17.0 | 20.0 |
| | | | 12/10/75 | 7.0 | 23.0 | 20.0 |
| Tempe, Ariz. | Crossing Forest Ave. North of University Ave. | 43 | 11/19/75 | 37.8 | 9.1 | 23.1 |
| | | | 12/4/75 | 21.7 | 25.2 | 23.1 |
| | | | 12/11/75 | 7.0 | 39.9 | 23.1 |
| Buffalo, N.Y. | Crossing Pearl St. South of Church St. | 47 | 11/25/75 | 7.0 | 26.6 | 36.4 |
| | | | 12/3/75 | 14.7 | 18.9 | 36.4 |
| | | | 12/10/75 | 23.1 | 10.5 | 36.4 |
| Buffalo, N.Y. | Crossing W. Ferry St. West of Grant St. | 40 | 11/26/75 | 21.6 | 8.4 | 30.0 |
| | | | 12/3/75 | 14.4 | 15.6 | 30.0 |
| | | | 12/11/75 | 7.2 | 22.8 | 30.0 |

the field study for that alternative. This permitted pedestrian activity to stabilize before data was collected.

For each site and alternative, two types of studies were carried out, one dealing with pedestrian compliance and the other with pedestrian behaviors as defined in Phase I of the project (2). These were conducted simultaneously for four hours each between approximately 10:00 a.m. and 2:30 p.m. For the compliance study, the signal cycle was divided into three parts defined by the three basic pedestrian intervals. The number of pedestrians leaving the curb during the three intervals in each direction was recorded for each cycle and classified by the interval in which they arrived on the far curb. Ideally, the arrival time would be to the middle of the farthest traveled lane. However, arrival at the far curb was used to simplify the data collection procedure and if anything, makes the data slightly conservative in favor of the pedestrian. A sample form is shown in Appendix A. This method of recording permitted an analysis to be made not only of obedience to the indication displayed but also of whether the pedestrian completed his crossing during a safe or unsafe interval. This latter factor relates primarily to walking speed. For example, a pedestrian may be non-compliant to the clearance interval, but because he realizes he must hurry to complete his crossing in time, he may quicken his pace. The end result is the transformation of a potentially unsafe crossing into a safe crossing.

The second study related to safety consisted of the collection of data on four types of behaviors: B, RTV, TV and MVM as defined below:

- (B) - Backup Movement - Momentary reversal in pedestrian direction of travel in the traffic lane, or hesitation in response to a vehicle in a traffic lane.
- (RTV) - Running Turning Vehicle Conflict - Running in a traffic lane in response to a TV (defined below).
- (MVM) - Moving Vehicle (modified) - Any vehicle moving through the crosswalk while the pedestrian is in a traffic lane during the solid DONT WALK interval.
- (TV) - Turning Vehicle - Number of turning vehicles involved coming within 20 feet of a pedestrian (in path of vehicle).

A sample data collection form is shown in Appendix A. Concurrent with the behavioral data, turning and through traffic volumes crossing each crosswalk were collected. These were used in developing meaningful results from the TV and MVM data.

Three persons were generally used to collect the data, one for both the behaviors and vehicle data and one for compliance in each direction on the crosswalk. The observers were trained prior to the commencement of data collection and had little problem in obtaining accurate data. In addition, the weather was surprisingly similar for all tests within each particular city, further contributing to the confidence which can be placed in the data. However, temperature differences among the three cities were fairly significant, which may partially explain differences in overall compliance data. Washington, D.C. temperatures averaged approximately 45 degrees, Phoenix, 70 degrees and Buffalo, 30 degrees.

## Data Analysis

### Vehicle Right Turn Delay-Preliminary Analysis

Once the raw delay data had been extracted from the films, it was translated into usable form for the mathematical analysis. This was done by correlating the expected vehicle right turn delay to pedestrian volumes for vehicles arriving during each 3-second time increment during the phase. The delay to a vehicle was defined to be $t_2 - t_1$ minus the average time for a right turn maneuver with no pedestrian interference. An undelayed right turn was found to average approximately 4 seconds from $t_1$ to $t_2$ except for the first vehicle which took about 5 seconds because of starting delay. Complicating the analysis was the fact that a vehicle, after it has passed the $t_1$ position, is sometimes delayed by the preceding right turn vehicle. Accordingly, the delay encountered by the following vehicle while the preceding vehicle was still being delayed was subtracted from the actual $t_2 - t_1$ value of the following vehicle. A pseudo-arrival time ($t_1$) was then established by adding the simultaneous delay time to the $t_1$ value of the following vehicle. This essentially says, for the purpose of the delay analysis, that vehicles never arrived at $t_1$ before the preceding vehicle had ceased being delayed by pedestrians.

To further illustrate, consider the 4th and 5th vehicle in cycle number 2 in Table 9. The best way to determine the delay to be subtracted from the value for the 5th vehicle is to consider the 5th vehicle to have

The analysis of the sensitivity of the clearance interval to pedestrian volumes and travel speeds is also based on the results of research described previously in this report (see page 38 ). First, it has been shown that the walking speeds of pedestrian platoons on a crosswalk decreases with volume due to the increased presence of slower walking pedestrians. Thus, large fluctuations in volume over the day may warrant adjustments in the length of the clearance interval at a given location. However, the safety implications of doing this are not entirely clear. If the length of the clearance interval is changed at one location, some pedestrians may think that they have more time remaining than they actually do during that particular time of day.

The sensitivity of the clearance interval to variations in individual walking speeds by time-of-day was also examined. The basis for this analysis was the data reported by Hoel (5).concerning hourly travel rates in central business districts. Hoel found slightly higher speeds during the a.m. vehicle peak hour than during the mid-day hours. Higher speeds might also be expected during the p.m. peak hour, but it is evident that there is not enough difference between peak and off-peak speeds (probably no more than 0.2 ft./sec. from Hoel's study) to warrant changes in the clearance interval.

The analysis of time-of-day adjustments to timing based on environmental factors was more subjective in nature. No data collection was conducted for nighttime pedestrian behavior. It was felt that such observations would not yield conclusive evidence regarding the practicality of using a longer pedestrian interval for nighttime purposes.

The practicality of time-of-day adjustments to timing is also somewhat limited because of hardware constraints. For instance, a 3-dial controller with one timing pattern for each peak hour and one for the off-peak would not be capable of responding to low pedestrian volume conditions since all three timing patterns would include periods of heavy pedestrian activity. If one of the dials is used exclusively for nighttime traffic, an adjustment to pedestrian timing might be possible. Therefore, the ability to make such a change is a direct function of the number of timing plans and split changes which can be made, which implies a level of signal system sophistication not available in many cities.

# CHAPTER IV

# OTHER AREAS OF PEDESTRIAN SIGNAL TIMING RESEARCH

## TIME-OF-DAY ADJUSTMENTS OF PEDESTRIAN SIGNAL TIMING

### Study Approach

In the previous chapters, it was found that pedestrian signal timing requirements can vary with pedestrian and vehicle volumes. Fluctuations in pedestrian and vehicular volumes throughout the day may thus dictate time-of-day adjustments of pedestrian signal timing. The approach to this phase of the research involved the determination of the practicality of adjusting signal timing by time-of-day based on volume or other factors on which these adjustments might be based. Other criteria investigated included differences in walking speed by time-of-day and changes in environmental conditions (particularly daytime versus nighttime).

### Data Collection

Most of the data required for this analysis were taken from the observations of the pedestrian volumes described in Chapter II. Of particular interest were the discharge times of pedestrian queues and the walking speeds of platoons of pedestrians crossing the street. The relevant data collection and analysis procedures can be found beginning,on pages 15 and 37. Hourly volume counts from two sources were also used in this analysis (10, 11).

### Data Analysis

The objective of the analysis was to examine very high pedestrian volumes and very low pedestrian volumes at the same location and to determine whether significant differences in pedestrian WALK and clearance intervals were required to serve the pedestrian volumes present. The analysis of the minimum WALK interval has shown that an extension of the minimum WALK interval beyond 7 seconds is seldom necessary and that it should be reduced by no more than 2 to 3 seconds for low volumes (see page 30 ). Based on these results, it can be seen that the minimum WALK interval is very insensitive to pedestrian volumes. Even if volumes on one crosswalk were in the range of 2,000 or more pedestrians per hour, the extension of the WALK interval would probably be no more than 2 seconds. Nevertheless, time-of-day adjustments to the WALK interval could be made if very wide fluctuations in volume exist at one location.

be used. If it does, pedestrian volume per cycle should then be computed and vehicle right turn demand estimated. Next, using Table 27 the phase length required to service the right turn demand under standard timing can be determined. If the phase can be fairly easily extended to that length, standard timing should again be used.

If standard timing is still unable to service the right turn demand, other phasing schemes should be examined. If both phases are experiencing queuing problems, scramble would probably be best suited to the conditions. If only one phase was experiencing the problems, late pedestrian release would be the best candidate. The right turn capacity for both scramble and late release can be determined using the methodologies shown in the flow-chart. If neither of them can solve the problem, either it will have to be tolerated or other solutions such as dual turn lanes, turn prohibitions or geometric changes will be required.

In selecting the appropriate type of signal phasing, safety must be considered as well as delay. Scramble would appear to be the "safest" type of phasing in that vehicle and pedestrian movements are completely separated. However, it must be recognized that delay, particularly delay to pedestrians, can impact the safety of scramble and other phasing schemes as well. It was found in this study that scramble significantly increases pedestrian delay compared to standard timing. Because delays with scramble are longer, pedestrians may have more of a tendency to violate the signal during a solid DONT WALK interval, particularly on the interval during which pedestrians would normally cross under standard timing. It was observed in this study that scramble violation rates were generally higher on narrow streets, the condition for which scramble is most suitable from the delay perspective. Although violations are not always true indicators of a safety hazard, they tend to defeat the purpose for which scramble was designed.

In spite of these difficulties scramble may have some application to intersections where the characteristics of the pedestrian population require special consideration. For example, it may be used at locations where there are many elderly pedestrians or young school children. These locations should be selected very carefully so as not to cause significant increases in violation rates or create traffic congestion. If possible, scramble should be provided on an actuated basis so that the phase will not be introduced when pedestrians are not present.

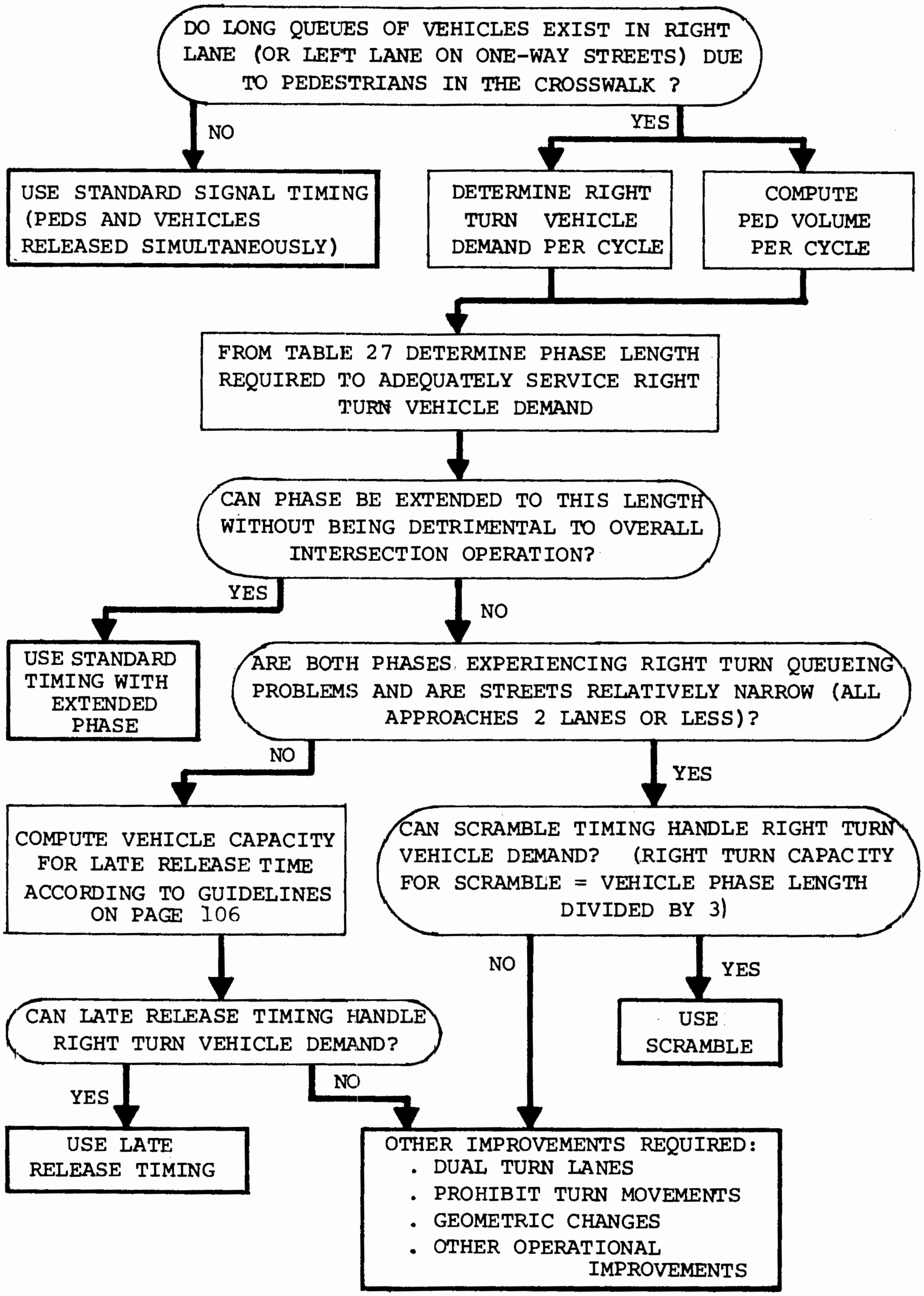


FIGURE 25. SELECTION OF PEDESTRIAN SIGNAL PHASING.

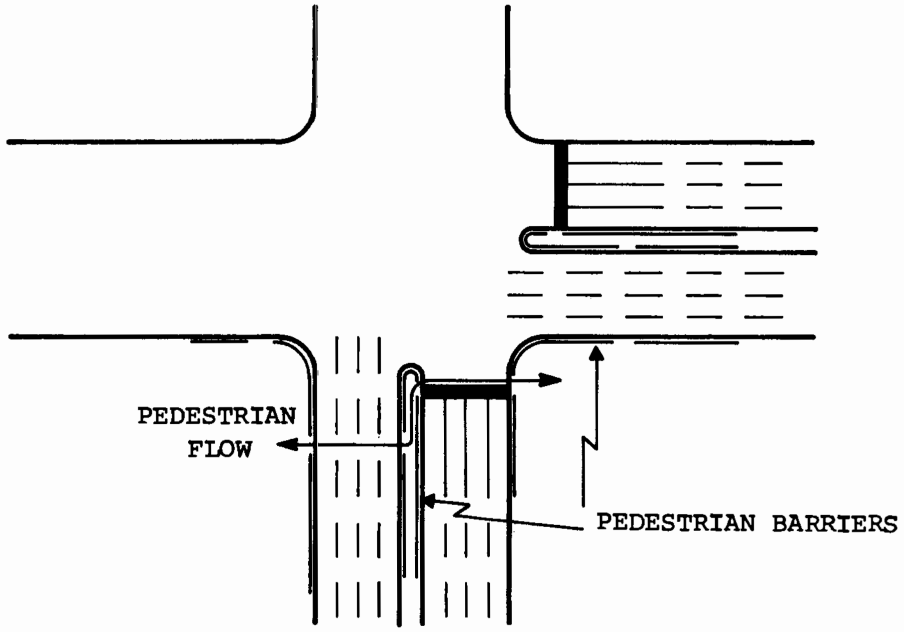
reducing delay. Both scramble and early pedestrian release timing are geared toward pedestrian safety. Scramble creates an exclusive pedestrian phase which, if obeyed, completely eliminates vehicle-pedestrian conflicts. Early pedestrian release creates a partially exclusive phase.

Like the summary on the timing of the WALK and clearance intervals, the methodology for selection of pedestrian signal phasing is presented in flow chart form (Figure 25). The flow chart is formulated primarily on the basis of pedestrian and vehicle delay. Safety justification for the various types is discussed following the flow chart presentations.

The principles upon which the methodology is based are stated below:

- Standard pedestrian phasing (concurrent vehicle-pedestrian phase) almost always minimizes total intersection delay.
- Late pedestrian release phasing can help to alleviate a vehicle right turn capacity problem. It should be used <u>only</u> when such a problem exists.
- Scramble timing is best used, from the perspective of delay, when <u>both</u> phases are experiencing queuing problems in the right turn lane due to pedestrian conflicts and when street widths are relatively narrow.
- Timing for the partial crossing of wide, channelized streets should be avoided if at all possible. It is desirable for the clearance interval to be timed for the entire crossing unless the median is over approximately 20 feet wide. If this type of timing must be used to minimize the side street phase, signs indicating the intent of the timing and/or pedestrian barriers should be provided on the median.

The decision process for the selection of phasing begins by determining whether a problem of vehicle queuing in the right turn lane exists during any hour of the day due to the vehicle-pedestrian conflict. This condition will usually require very heavy pedestrian and vehicle right turn volumes. If this condition does not exist, delay considerations dictate that standard timing



Note: Pedestrians cross to the median during the first cycle, are stored between the barriers, and finish their crossing on the second cycle.

FIGURE 24. USE OF BARRIERS TO CONTROL PEDESTRIAN FLOW ACROSS A MEDIAN.

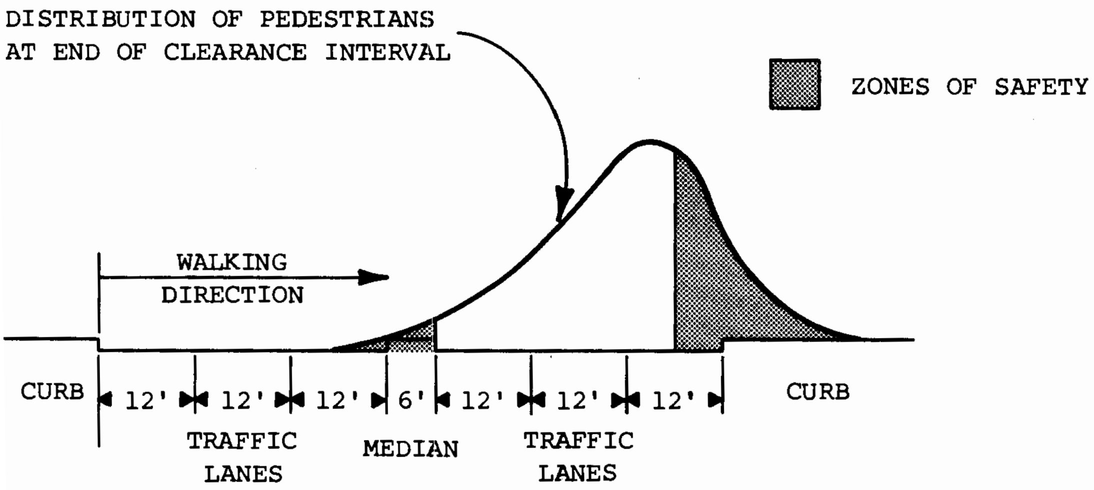
The question now remains as to how one would minimize side street green time and yet provide for the needs of the pedestrian. To solve this problem with this type of timing one would have to greatly reduce the variance in pedestrian speeds and/or reduce the time interval in which they are allowed to leave the curb. The first is very difficult to accomplish and the second would violate a recommended standard. The provision of signs in the median indicating that pedestrians should wait for the next cycle would, if observed, essentially produce more uniformity in average curb-to-median speeds, but good compliance may not actually occur.

One device which has been used by the United Kingdom in dealing with this problem is a set of barriers to control pedestrian flow across the median (see Figure 24). The variance in pedestrian speeds would be accommodated by providing offset gates on the median barrier as shown. The distance between the two gates would have to be coordinated with signal timing to insure proper operation.

In light of the fact that few, if any, other alternatives exist for minimizing side street green time, it is recommended that timing for the partial crossing of wide streets with median refuges be avoided if at all possible. Timing for the full crossing will afford the greatest degree of safety to the pedestrian. Streets with very wide medians (approximately 20 feet or more) will be the most conducive to timing for partial pedestrian crossings. When this timing must be used, it should be supplemented with barriers as stated above or with signs at the median stating that pedestrians should wait until the next cycle to complete their crossing. Messages such as "WAIT HERE FOR NEXT WALK" would be appropriate. In addition, it would be desirable to reduce the length of the WALK interval to 5 or even 4 seconds and to time the clearance interval for a walking speed closer to the average, perhaps 4.5 ft./sec., rather than the standard 4.0 ft./sec. This would tend to time a greater percentage of pedestrian arrivals at the median for the end of the clearance interval. Caution would have to be exercised, however, to insure that side street green is not reduced to below what is required.

## SUMMARY OF PEDESTRIAN SIGNAL PHASING

As mentioned previously in this report, the two primary concerns of pedestrian signal timing are increasing safety and reducing delay for both pedestrians and vehicles. Some of the alternative pedestrian phasing schemes examined in this research are more oriented toward increasing safety while others are more suitable for



| NUMBER OF SECONDS INTO WALK INTERVAL | PERCENT PEDESTRIANS DEPARTING |
|---|---|
| 1 | 25 |
| 2 | 35 |
| 3 | 20 |
| 4 | 5 |
| 5 | 5 |
| 6 | 5 |
| 7 | 5 |

DEPARTURE DISTRIBUTION

| SPEED (FT./SEC.) | PERCENT FREQUENCY |
|---|---|
| 3.0 - 3.5 | 2 |
| 3.5 - 4.0 | 7 |
| 4.0 - 4.5 | 18 |
| 4.5 - 5.0 | 31 |
| 5.0 - 5.5 | 26 |
| 5.5 - 6.0 | 11 |
| 6.0 - 6.5 | 4 |
| 6.5 - 7.0 | 1 |

SPEED DISTRIBUTION

(from Hoel - ref. 5)

FIGURE 23. ANALYSIS OF TIMING FOR PARTIAL CROSSING OF WIDE, CHANNELIZED STREETS.

vehicle travel, but in doing so, encourages the pedestrian leaving at the beginning of WALK to continue past the median. Consequently, those initial pedestrians may find themselves in the middle of the far side of the street at the beginning of the opposing green phase.

Further complicating this matter is the variation in pedestrian walking speeds which may often range between 3.0 and 6.0 feet per second. The faster pedestrians will proceed along the crosswalk at up to 2.0 feet per second faster than the assumed walking speed. Even those pedestrians which begin at the end of WALK may thus also find themselves in the middle of traffic lanes at an undesirable time.

To quantitatively illustrate these problems a hypothetical street was established with three 12 foot traffic lanes in each direction and a median 6 feet wide for a total width of 78 feet. The WALK interval would be 7-seconds and the clearance interval 8-seconds. Now suppose that the distribution of departures from the curb during the WALK interval and the distribution of pedestrian speeds are those presented in Figure 23. The distribution of pedestrians that will actually be located within the two zones of safety at the end of the clearance interval, given that pedestrians do not pause at the median, is also shown in Figure 23. This figure reveals the remarkable fact that the majority of pedestrians (approximately 70%) will end up not in the zones of safety afforded by either the median or far curb, but in the middle of the far side of the street at the end of the clearance interval. A very small percentage actually end up safely at the median, the purpose for which the timing is intended. Of course, the distribution will not be precisely obtained in reality, because pedestrians will likely adjust their behavior to afford a greater degree of safety. However, it does serve to point out that this is the <u>tendency</u> for pedestrians under this type of timing. Very significant changes in pedestrian behavior must therefore be made if this type of timing is to be effective. If this cannot be done, pedestrian signals should be timed for a full, rather than for a partial crossing.

A further analysis of the figure suggests that any variations in the intervals might never be truly effective from the standpoint of safety. For instance, if one of the intervals were reduced in length (preferably the WALK interval), centering the distribution more over the median and compacting it somewhat more tightly, still only 40-50 percent of pedestrians would tend to finish in the median zone of safety.

was concluded from the observations of pedestrian usage of the median that pedestrians, once committed to the crosswalk, will continue their passage until prevented by moving vehicles from doing so.

Field observation during the course of this research tended to support this statement. The primary reason appears to be that pedestrians need only to watch for vehicles in one direction at a time and do not need to wait until the entire roadway is clear in both directions at once before they begin their crossing. The lower the vehicle volumes, the more opportunities there will be to cross and thus the greater will be the probability of violation. The median serves as a mid-crossing refuge for pedestrians, giving them greater confidence in violating the signal. The number of violations is not necessarily indicative of safety problems but suggests that crossings with pedestrian refuges are difficult to control with signalization.

The problem of controlling violations at such locations was very noticeable from about two hours of observing pedestrians crossing Connecticut Avenue at L Street in Washington, D.C. Connecticut Avenue has a narrow (about 5 feet) street level median at that location. The clearance interval provides just enough time for pedestrians to cross to the median before the next phase begins. Those pedestrians who did use the median as a refuge did not usually wait for the next WALK interval to complete their crossing but took advantage of the next available gap in the traffic stream. Another interesting observation was that from 2:30 to 3:30 p.m., progression was fairly good in both directions on Connecticut Avenue at L Street. Consequently, the vehicles moving through the crosswalk discouraged pedestrian movements to the median from either curb on that phase. At approximately 3:30 p.m. timing changed to favor northbound Connecticut Avenue traffic and, as a result, few southbound vehicles arrived at the intersection during the green phase. After that time, numerous pedestrians crossed the southbound lanes on the solid DONT WALK interval with very little risk.

There are two basic problems inherent in this type of timing. First, the pedestrian WALK interval must be timed at the minimum of 7-seconds. A pedestrian leaving at the beginning of that interval and walking at 4.0 ft./sec. would proceed 28 feet into the street before the WALK is terminated. This is well over half of the distance to be crossed at one time on most divided roadways. The clearance interval is timed to allow pedestrians starting to cross at the end of the WALK interval to cross one direction of

> 6. Modern street lighting meeting I.E.S. standards for illumination at each intersection under consideration."

Other opportunities for its use may be at school crossing locations, where safety is of special concern and at "T" intersections with heavy vehicle and pedestrian volumes. Each scramble location must be assessed individually, taking into account the effect of scramble on pedestrian and vehicular delay and safety. A method has been presented on page 114 whereby capacity for a right turn lane under scramble timing can be compared to that for standard timing. Scramble timing would be most applicable where long queues develop in the right lane on both phases under standard timing.

## TIMING FOR THE PARTIAL CROSSING OF WIDE, CHANNELIZED STREETS

The Manual on Uniform Traffic Control Devices has set forth a policy which is particularly applicable to pedestrian signal timing for wide streets with medians separating the directions of vehicle travel. The policy is stated as follows:

> "On a street with a median at least 6 feet in width, it may be desirable to allow only enough pedestrian clearance time on a given phase to clear the crossing from the curb to the median. In the latter case, if the signals are pedestrian-actuated, an additional detector shall be provided on the island."

No formal categories of discussion have been used for this area of research. A general analysis of timing for wide, channelized streets is presented below.

The purpose of this type of timing is primarily to minimize the length of the side street green phase which, if governed by a full-length pedestrian crossing of the major street, may result in heavily under utilized green time on the side street. This may take away much needed green time on the major street, possibly resulting in a capacity deficiency.

Although the intent of this policy is well-founded, its practical application is much more difficult. A 1964 study in Dade County, Florida (9) addressed this question at mid-block crosswalks on streets with medians between 5 and 18 feet wide. The study is not directly applicable in that timing was significantly longer than the minimum required for pedestrians to cross to the median; but it criteria for which scramble is most applicable. Most of these violations were found to occur on the vehicular phase normally used by pedestrians under standard timing. A lack of right turn maneuvers generally encourages the most violations for those locations. The reaction to scramble undoubtedly varies in different parts of the country and in various size cities. Scramble timing may tend to be more respected in smaller cities. At least one study has indicated that scramble can reduce pedestrian accidents (7).

Scramble and similar exclusive pedestrian phases have been widely applied to school crossings and justifiably so, regardless of its impact on delay. School crossing guards are sometimes present at these locations to supplement the signal. Scramble may also be helpful at "T" intersections where vehicles from the side street must turn, and in so doing either reduce the gaps available to pedestrians or incur substantial delay themselves. Where scramble is selected for use at an intersection, it is usually desirable to provide it on a pedestrian actuated basis. This will allow the pedestrian phase to be introduced only when pedestrians are present.

## Conclusions and Recommendations

Although scramble timing has been found to decrease vehicle delay for narrow streets with heavy pedestrian and vehicle right turn volumes, it tends to increase pedestrian delay by an amount equal to and often significantly greater than this value. Conditions other than those examined in this study will be much less conducive to scramble. This research has supported the statement of scramble warrants by Dier (8) as stated below:

"1. Extremely heavy vehicular volumes complicated by a steady flow of pedestrian traffic in such numbers that turning movements at intersections are limited to amber intervals.

2. Absence of excessive through traffic volumes.

3. Low average speeds.

4. Intersecting streets of relatively narrow widths (approximately 50 feet or less curb to curb).

5. Sufficient sidewalk width to permit storage of large numbers of pedestrians.

The delay incurred for the same vehicles under standard timing would be approximately 185 person-seconds per cycle for 6 vehicles and over 400 person-seconds for 8 vehicles. Again, the large value for 8 vehicles arises out of the inability of standard timing to service over 6 vehicles per cycle for the given assumptions. Thus, for both arrival patterns, the delay saved for 8 vehicles under scramble timing nearly equalizes the delay incurred by pedestrians. It should be emphasized that this is probably one of the most favorable combinations of intersection and traffic characterisitcs for scramble. It is very rare that such ideal conditions will exist in reality, particularly such a high turning percentage. A lower turning percentage would reduce the vehicle delay advantages of scramble substantially. If very few vehicles turned, or if the street was wider, scramble would more likely increase vehicle delay than reduce it.

The capacity of a right turn lane under scramble timing can be estimated by dividing the length of the exclusive interval by 3, the approximate average right turn headway for unobstructed turns. This can be compared to the estimated capacity of the lane under a combined vehicle-pedestrian phase presented in Chapter IV (Table 27, page 136). For the timing alternatives considered here, scramble would yield a turn capacity of 9-10 vehicles per cycle while standard timing, with 20 pedestrians per cycle, could have as little as a 5 vehicle capacity. Thus, the prime advantage of scramble would accrue to vehicles by increasing right turn capacity under conditions of heavy vehicle right turn and pedestrian volumes. In cases where a queuing problem exists in the right turn lane, scramble may be a means by which the problem can be alleviated. It would be particularly useful where such problems exist on both vehicle phases. Late release phasing may be more applicable where queuing problems exist on only one phase.

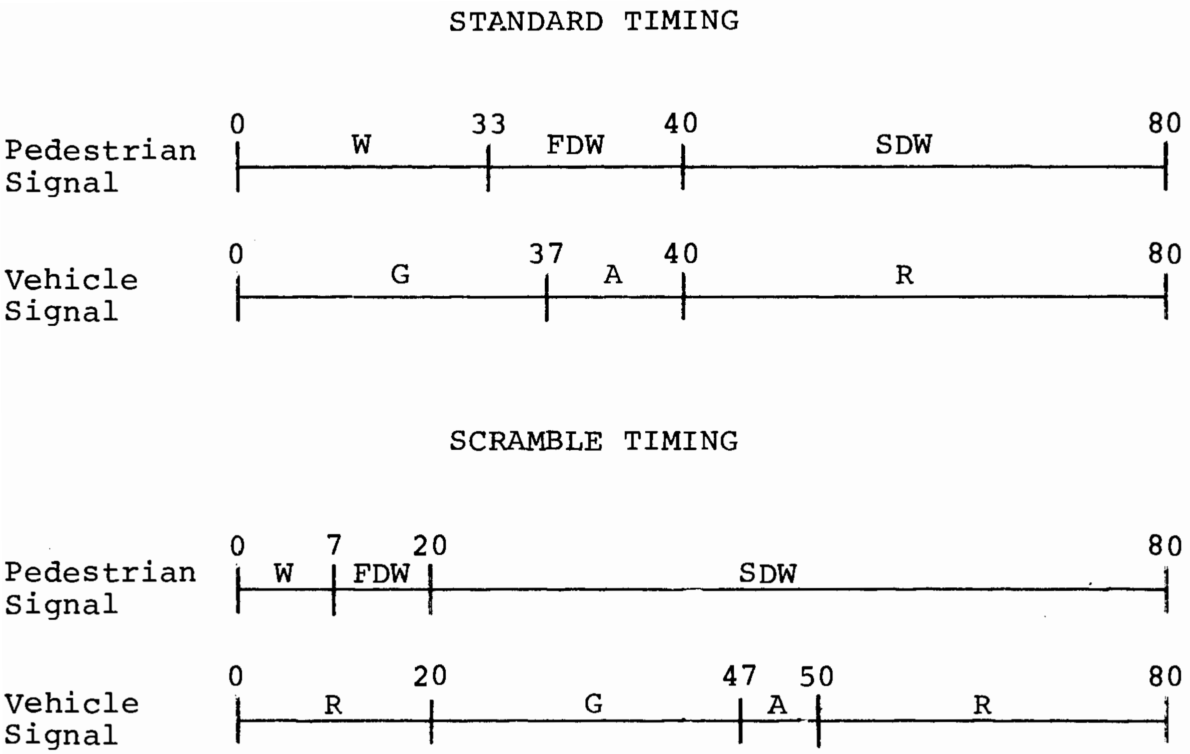
## Safety Aspects of Scramble Timing

Despite its drawbacks from the delay standpoint, scramble does have a number of possible applications because of its safety features. Assuming that its indications are obeyed, scramble can completely separate pedestrian and vehicle movements, thereby reducing the potential occurrence of a pedestrian accident. Where violations are frequent, scramble may be more of a safety hazard than an accident prevention measure. Some observations of pedestrians at several scramble locations revealed that violations are more frequent for narrower streets, which is the geometric for a multitude of street widths, pedestrian volumes and cycle lengths. The increase is primarily due to the additional delay encountered by pedestrians on the parallel crosswalks. However, the results also indicate that overall delay is increased not only for those crossing parallel but also for those crossing diagonally.

The comparison of vehicular delays is much more difficult because it is highly dependent on vehicle arrival patterns. The arrival patterns, in turn, depend on the characteristics of the street system upstream from the intersection, particularly signal offset. In an interconnected signal system, a scramble installation would influence delay not only on the intersection itself, but on adjacent intersections as well. It is usually unlikely that good signal progression can be established in areas where scramble would normally be considered (i.e., shopping and business districts). Large turning percentages and vehicular friction often cause progression to break down in these areas before it would normally break down in a less densely developed area.

In order to assess the differences in vehicle delay between scramble and standard timing, two arrival patterns were used. One assumed uniform vehicle arrivals while the other assumed that all vehicles were in a platoon which arrived at the beginning of green. Such conditions are uncommon in everyday experience, but probably form the two extremes of possible arrival patterns. Normal arrival patterns will fall between these two extremes, suggesting that the delays computed will also form the boundary between the maximum and minimum effect of scramble on vehicle delay for the given street width, signal timing and volume levels. Given a uniform vehicular arrival rate and the stated assumptions, it was found that six vehicles turning right would incur approximately 80 vehicle-seconds or 120 person-seconds less vehicular delay with scramble than with conventional signal timing. With 8 vehicles turning right, scramble reduces vehicle delay by over 300 person-seconds. This large difference in delay between the 6 and 8 vehicle level occurs because only 6 of the vehicles can be served per cycle by standard timing whereas all can be served under scramble. The vehicle delay saved with 8 vehicles turning right begins to compare with the pedestrian delay caused by scramble.

Examining the other extreme, that of vehicles arriving in a platoon at the beginning of green, it can be seen that no delay would be incurred by vehicles under scramble timing since there would be no conflicts with pedestrians.



where G = Green indication
A = Amber indication
R = Red indication
W = WALK interval
FDW = flashing DONT WALK interval
SDW = solid DONT WALK interval
Numbers indicate time from the beginning of the cycle.

FIGURE 22. TIMING ALTERNATIVES EVALUATED FOR STANDARD AND SCRAMBLE TIMING.

- perfect pedestrian compliance
- uniform pedestrian arrivals for scramble timing and the distribution previously discussed (see page 72) for standard pedestrian timing
- 40 foot street with parking on both sides
- 80-second cycle with 50-50 split
- 3-second average vehicle right turn headways at saturation flow
- 100 percent right turns (the extreme case most favorable to scramble)
- 6 vehicles per cycle
- 20 pedestrians per cycle

The two alternative timing schemes investigated included one with normal phasing and one with scramble phasing as shown in Figure 22. The pedestrian clearance interval was timed for the diagonal crossing and assumes a curb radius of 15 feet. One parking lane was subtracted from the total street width in the computation of the distance to be crossed (this practice may not be advisable based on the results of the parking lane studies (see page 47) but is done here to make the assumptions conservative in favor of scramble timing).

The analysis of pedestrian delay was based on a 2:1 ratio of pedestrians using the parallel crosswalk to those crossing diagonally. This ratio was representative of the travel characteristics at several scramble locations in Washington, D.C. For the example used here, the 20 pedestrians per cycle would translate into 13 crossing parallel and 7 crossing diagonally. For the non-scramble alternative, the pedestrians desiring to cross diagonally were presumed to use the WALK intervals on which they would incur the least delay.

The results indicate that pedestrian delay due to scramble is far greater than the delay which exists at the same location with standard timing. The delay to the 20 pedestrians with standard timing was approximately 200 person-seconds while that with scramble was in the range of 650 person-seconds, an increase of over 200 percent. This percentage of increase will be approximately the same or the pedestrians no longer perceive its purpose, often resulting in a high violation rate. It is therefore recommended that standard timing be used unless the right turn capacity problem cannot be tolerated.

## EFFECT OF SCRAMBLE OPERATIONS

### Study Approach

"Scramble" pedestrian timing (i.e., providing an exclusive pedestrian phase for all directions including diagonal) has been used in a variety of situations in the past. Probably the most frequent use now occurs in shopping and business districts where pedestrian volumes are very high and at school crossings where safety is of utmost concern. Although some studies concerning its impact have been made, the advantages and disadvantages of its use as related to both pedestrian and vehicular delay and safety are not entirely clear.

The approach taken in this research phase was to assess the delay and safety aspects of scramble timing. Again, a mathematical approach to delay was taken using the previously generated delay data as a base. A condition was selected in which scramble would probably be most warranted in order to assess its greatest potential benefit. The approach to safety was based on the past experience of other researchers and on the observation of actual scramble timing installations.

### Data Collection

No additional data were required for the delay analysis with only general observation necessary for the assessment of safety.

### Data Analysis

#### Delay Analysis

The first step in the analysis was the determination of conditions for which scramble timing is usually most well suited at a normal four-legged intersection. These conditions include high pedestrian volumes, low vehicular through volumes with medium to heavy right turn volumes and narrow street widths.

The delay analysis was undertaken using the following assumptions which were based on the scramble warrants given above.

- Late pedestrian release tends to increase overall intersection delay except where vehicle right turn volumes are high. It can be used to increase the capacity of a right turn lane.

- Pedestrians have been found to generally comply with the late release interval at the locations tested in this research. Late release has been used at these locations for over 10 years so that it is uncertain whether compliance would be high at new installations in other cities.

The analysis of signal timing for early release of pedestrians has shown that it is inferior to standard timing in terms of total intersection delay. It is likely, however, that some improvement in safety will occur, although the degree of improvement was not quantified in this study. Late release may be advantageous in terms of delay where there are medium to heavy right turn vehicular volumes (or left turn volumes on one-way streets). Vehicle delay will usually be reduced and pedestrian delay will always be increased with such timing so that the perceived advantage of this device largely rests on the relative weight placed on the two factors. Indications from the safety aspect are that no unusual hazards are created by late release timing.

The period immediately following the installation of either of these timing schemes is probably the most critical. Unless sufficient precautions are taken, unwanted safety hazards may be created. Signs for both pedestrians and vehicles should be used to minimize the initial adverse consequences. Late release is probably the more critical in this regard as pedestrians are more likely to disregard traffic signals. It would be wise to install permanent signs for pedestrians for late release, particularly if the location at which it is used is the only one or one of the few installed in a given city.

Finally, a short late release, such as the 7-second interval used here, does not adequately relieve vehicle right turn capacity problems caused by conflicts with heavy pedestrian volumes. Either a much longer late release interval must be provided or other methods, such as prohibiting pedestrian crossings (undesirable) or creating dual turn lanes, must be used. A major problem with providing late release timing is that it will normally be warranted only for certain hours of the day. During the remaining hours, either it creates additional delay

## TABLE 21. RESULTS OF LATE RELEASE COMPLIANCE STUDY

| Location | Frequency/% Frequency of Occurrence | | | | | | |
|---|---|---|---|---|---|---|---|
| | Begin on Walk | Begin on Clearance | | Begin on SDW | | Begin on Arrow | Total Peds. |
| | | End on Clearance | End on SDW | End on SDW | End on Walk | | |
| 5th & Nebraska Sts. (w/late release) | 503/76.6 | 76/11.6 | 25/3.8 | 33/5.0 | 0/0 | 20/3.0 | 657/100 |
| 6th & Douglas Sts. (w/late release) | 434/71.1 | 48/7.9 | 24/3.9 | 85/13.9 | 4/0.7 | 15/2.5 | 610/100 |
| 6th & Douglas Sts. (w/std. timing) | 615/81.6 | 52/6.9 | 14/1.9 | 55/7.3 | 18/2.4 | -- | 754/100 |

## Results of Late Release Compliance Study

The results of the compliance study for late release in Sioux City are presented in Table 21. The most interesting data with respect to late release is in the "begin on arrow" column. It indicates the degree to which pedestrians tend to remain on the curb while vehicles are allowed to proceed. The violation rate of this late release interval is remarkably low at both intersections, ranging between 2.5 and 3.0 percent. This suggests that pedestrians are not overly concerned about the 9-10 second additional delay. Compliance rates for the remainder of the categories appear to be comparable to the results of the previous studies described in this report. The data obtained at 6th and Douglas Streets under standard timing seem to confirm this point.

One might hypothesize that the rate of compliance to the late release interval in other cities might be proportional to the respective overall compliance rates. However, it is difficult to ascertain what the effect would be on pedestrian behavior in other cities without actually installing such a system. The long time period and number of locations in which late release has been used in Sioux City has made both the pedestrian and driver quite familiar with its operation, possibly contributing to the high compliance rate. It is not unlikely that such installations would have a long acclimation period when introduced into other cities. In fact, this problem was experienced in reverse when timing was changed from late release to standard operation in Sioux City. Vehicles approaching the 6th and Douglas Street intersection naturally expected to receive the green arrow giving them preference over pedestrians at the beginning of the phase. Many drivers confronted with this situation assumed that they still had the right-of-way, causing several near misses on the crosswalk. The situation stabilized within a period of several weeks as motorists and pedestrians became accustomed to the new timing.

## Conclusions and Recommendations

The analysis of early and late pedestrian release phasing alternatives has resulted in the following conclusions:

- Early pedestrian release significantly increases overall intersection delay. Its contribution to pedestrian safety is unclear.

substantial excess pedestrian time is available, all of the excess could be put into a late release interval. Where pedestrian volumes are less than 10 per cycle, the WALK interval might be reduced to 4-5 seconds to provide additional time for uninhibited right turn flow (see page 30 for a discussion on shortening the WALK interval). Thus, for the 80-second cycle and street width used here, the late release phase could be extended to as long as 21 seconds, permitting at least 7-8 vehicles to turn right under the heaviest of pedestrian volume conditions. Under standard timing and 20 pedestrians per cycle it would be possible to serve only 6 vehicles per cycle. Where there is not enough excess pedestrian time available to create a late release interval of sufficient length to increase capacity, it would be necessary to lengthen the entire phase.

The vehicle right turn capacity which exists with late release timing can be approximated by the following formula:

$$LRC = LRT \div 3 + x$$

where LRC = late release capacity per cycle (vehicles)
LRT = time allocated to the pedestrian late release interval (seconds)
3 = approximate average vehicle headway for right turns
x = capacity of remainder of the phase after pedestrians are released (vehicles)

The value of x would depend on the time allocated to the WALK and clearance interval and on the pedestrian volume. A table which lists the approximate right turn capacity for a range of interval times and pedestrian volumes is presented in Chapter IV under the section on capacity factors. This table can be found on page 138 (Table 27).

Another means of dealing with a right turn capacity problem would be to completely prohibit pedestrian crossings on the intersection leg affecting vehicle turns. This would be a particularly applicable device on crossings of 1-way streets, as the increased volume on the opposite crosswalk would not impact vehicles. It would probably be difficult to prevent illegal crossings, however, unless some sort of barrier was installed and proper enforcement provided. A third method might be to create dual vehicular turn lanes, possibly in combination with late release timing. All of these alternatives will result in considerable delay to the pedestrian.

## TABLE 20. EFFECT OF LATE RELEASE OF PEDESTRIANS ON PEDESTRIAN AND VEHICULAR DELAY

| Pedestrian Volume Per Cycle | Increase in Ped. Delay (person-sec) | Max./Min. Increase in Vehicle Delay (delay in person-sec/cycle) | | | | Max./Min. Increase in Total Delay (delay in person-sec/cycle) | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 2 vpc[1] | 4 vpc | 6 vpc | 8 vpc | 2 vpc | 4 vpc | 6 vpc | 8 vpc |
| 0 | 0 | 0/0 | 0/0 | 0/0 | 0/0 | 0/0 | 0/0 | 0/0 | 0/0 |
| 2 | 9 | 0/-8[2] | 2/-20 | 6/-32 | 12/-44 | 9/1 | 11/-11 | 15/-23 | 21/-35 |
| 5 | 21 | 0/-15 | 3/-36 | 12/-57 | 21/-78 | 21/6 | 24/-15 | 33/-36 | 42/-57 |
| 10 | 41 | 0/-29 | 5/-68 | 21/-107 | 42/-146 | 41/12 | 46/-27 | 62/-66 | 83/-105 |
| 20 | 83 | 0/-53 | 8/-139 | 26/-185 | 165/-312 | 83/30 | 91/-56 | 109/-102 | 248/-229 |

[1] vpc = vehicles per cycle

[2] Negative sign indicates that vehicle delay decreased with late release.

vehicle capacity is likely to exist. This is not to say, however, that early release should be used at these locations since it has been shown to increase delay.

### Results of Late Release Delay Analysis

For late pedestrian release, there is obviously a significant increase in total pedestrian delay due to the 7-second advance green for vehicle movements. The major question is whether this can be offset by a reduction in vehicular delay. The vehicle arrival pattern which will be most benefited is where the first three vehicles turn right with the remainder of the platoon being through vehicles. In this case, there would be no pedestrian-induced delay to vehicles and a considerable improvement over standard phasing would result. On the other hand, the least benefit would be gained where the first three vehicles proceed through and the remainder of the platoon turns right. With this condition, there would actually be additional delay incurred by vehicles when compared with standard phasing since there will be no right turning vehicles in position to take advantage of the late pedestrian release.

The results of this analysis for various pedestrian and vehicle volume levels are shown in Table 20. It can be seen that total delay is always increased by late release for low vehicular volumes and is either increased or decreased for higher volumes. Since the analysis was limited to the extremes of vehicle arrival patterns, it is not easy to determine whether late release would, on the average, increase or decrease total delay. The table suggests that total delay would be nearly equal if an average of the two delay values could be used to estimate delay for the entire distribution of arrival patterns. Where nearly all vehicles turn right and pedestrian volumes are heavy, late release would significantly reduce vehicle delay, since it allows the first several vehicles to proceed unobstructed. It would also increase vehicle capacity to some extent by concentrating pedestrian movements into a shorter period of time, thus increasing time available for free vehicular movement. The potential increase in the capacity of a right turn lane would be even more significant if a longer late release interval were used.

This introduces the subject of effectively dealing with a vehicle right turn capacity problem using late release timing. Where such a problem exists and where

### TABLE 19. EFFECT OF EARLY RELEASE OF PEDESTRIANS ON PEDESTRIAN AND VEHICULAR DELAY

| Pedestrian Volume Per Cycle | Increase in Ped. Delay (person-sec) | Max./Min. Increase[1] in Vehicle Delay (delay in person-sec/cycle) | | | | Max./Min. Increase in Total Delay (delay in person-sec/cycle) | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 2 vpc[2] | 4 vpc | 6 vpc | 8 vpc | 2 vpc | 4 vpc | 6 vpc | 8 vpc |
| 0 | - | 21/0 | 42/0 | 63/0 | 84/0 | 21/0 | 42/0 | 63/0 | 84/0 |
| 2 | 0 | 18/0 | 36/0 | 54/0 | 72/0 | 18/0 | 36/0 | 54/0 | 72/0 |
| 5 | 0 | 18/0 | 36/0 | 54/0 | 72/0 | 18/0 | 36/0 | 54/0 | 72/0 |
| 10 | 0 | 15/0 | 30/0 | 45/0 | 60/0 | 15/0 | 30/0 | 45/0 | 60/0 |
| 20 | 0 | 12/0 | 24/0 | 36/0 | 48/0 | 12/0 | 24/0 | 36/0 | 48/0 |

[1] Maximum possible increase in vehicle delay for early release versus standard timing occurs when first vehicle turns right and remainder are thru. Minimum increase occurs when at least the first three and possibly all vehicles are thru.

[2] vpc = vehicles per cycle

vehicle delay. This would produce a range of possible results so that the actual expected delay would fall somewhere between the two extremes. The expected vehicle delay would be dependent upon the probability of all possible arrival patterns. Ideally, to develop more specific values, the delay analysis would be more completely treated by the use of a computer.

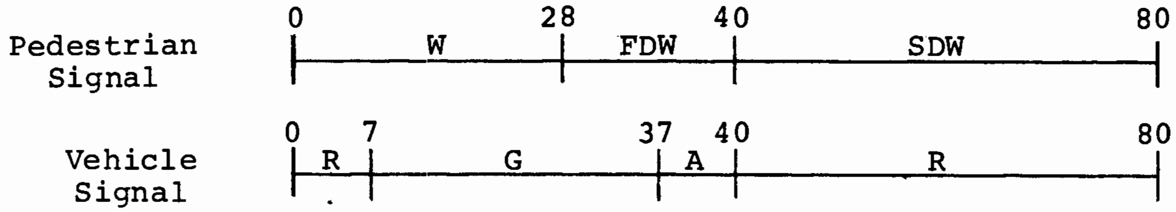
### Results of Early Release Delay Analysis

Considering early pedestrian release versus the base case of standard timing, the vehicle arrival pattern which incurs the greatest additional vehicle delay is one in which the first vehicle is turning right and the remainder are going straight. In this case all vehicles in that lane would be delayed by 7-seconds plus whatever pedestrian-induced delay was encountered by the first vehicle (assuming that vehicle could not be by-passed). The other extreme would be the case where at least the first three and possibly all vehicles in that lane were going straight. In this case delay for both standard timing and early release would be equal since there would be no penalty introduced to any of the vehicles by early release timing. However, because this feature holds little advantage where the second arrival pattern is more predominant, early release would most often be considered where right turn movements are fairly frequent. Thus, significant levels of vehicle delay are inherent in this type of timing.

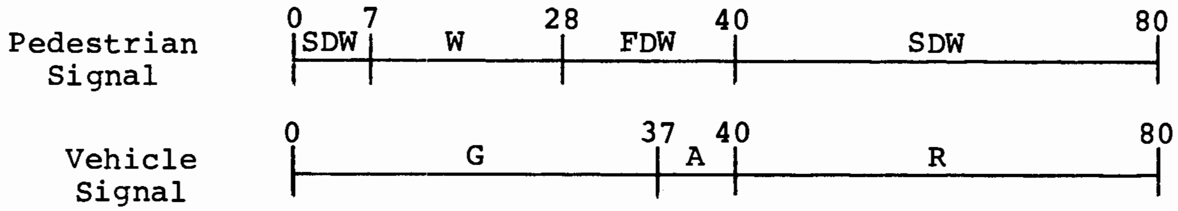
The results of the delay analysis for the two arrival patterns are presented in Table 19. It should be noted that pedestrian delay is not increased with early release since the length of the WALK interval is the same as with standard timing. Overall, early pedestrian release will always result in additional total person-delay at an intersection when compared to standard phasing.

It should be noted that early pedestrian release will require additional traffic signal equipment in order to individually control each vehicle lane approaching the signal. During the advance pedestrian phase, signals would be green for through traffic only, and red for all turning traffic. The best location for using early release would be where left turns are prohibited (or where they did not conflict because of one-way streets). The intersection of two one-way streets would be an acceptable location for this type of signalization, since individual lane control can be more easily exercised, and additional

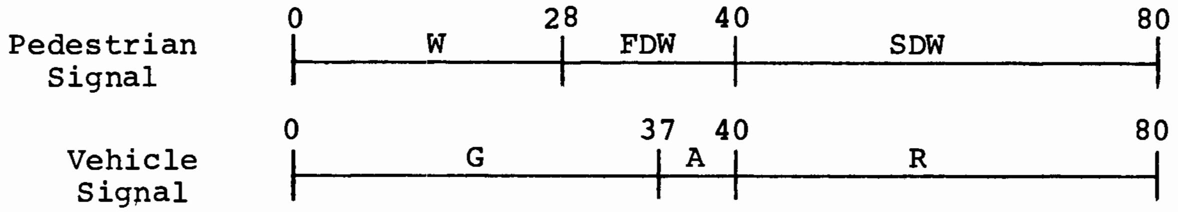
EARLY PEDESTRIAN RELEASE



LATE PEDESTRIAN RELEASE



STANDARD PHASING



where W = WALK interval
FDW = flashing DONT WALK interval
SDW = solid DONT WALK interval
G = green indication
A = amber indication
R = red indication
Numbers indicate time from beginning of the cycle.

FIGURE 21. TIMING USED IN ANALYSIS OF EARLY AND LATE RELEASE.

- 3-second headways for right turn vehicles and 2-second headways for through vehicles at saturation flow

- 52 foot crosswalk

- 80-second cycle with 50-50 split

- all vehicles in the right lane are in the queue at the beginning of green

- various combinations of through and right turn vehicles to simulate the least and greatest delay differences between early or late release and standard timing

- a 7-second late pedestrian release when this strategy was being considered

- a 7-second early pedestrian release (vehicular right turn lane stopped) when this strategy was considered

A high pedestrian violation rate of the late release would lower pedestrian delay, but would probably increase vehicular delay. The 7-second late release was designed to allow approximately three vehicles to turn before pedestrians step into the crosswalk. The 7-second early release was selected to provide time for pedestrians from the near side of the street to pass the zone of conflict prior to the release of vehicles. Longer early or late release times could be used if there is sufficient time in the vehicle phase to insure that the pedestrian minimums are maintained.

The two timing schemes to be tested are graphically displayed in Figure 21 and compared to standard signal phasing for a hypothetical intersection.

The delay computations for this phase of the research were conducted in a manner similar to that used for the allocation of excess pedestrian time (see page 62). To compute pedestrian delay, delays for the late pedestrian release and early pedestrian release were calculated using the assumed arrival rate and compared to standard signal phasing. To compute vehicular delay, two calculations were performed for each case, based on the particular vehicle arrival patterns which would result in the maximum difference in vehicle delay and the minimum difference in as well. Pedestrian delay was computed on the basis of the pedestrian arrival rate discussed on page 72.

A field study was initiated in Sioux City, Iowa, to observe pedestrian compliance to late release installations. Late release has been used at many of the intersections in downtown Sioux City for over ten years. Most of these are at intersections of one-way streets. The installations include a right turn arrow for vehicles which is displayed for all but the last 2 to 3 seconds of the late release interval for pedestrians.

Two crosswalks equipped with the late release features were observed, 6th and Douglas Streets and 5th and Nebraska Streets. The 5th and Nebraska Street intersection also included dynamic pedestrian signals. A compliance study very similar to that used for determining the best allocation of excess pedestrian time was performed at each intersection. The only difference was the addition of a column in the form to record those pedestrians leaving the curb on the DONT WALK interval while the vehicular right turn arrow was being displayed (see Appendix A for a sample form). Pedestrians were observed during midday in October, 1975 for a total of 5 hours at 6th and Douglas Streets and 2 hours at 5th and Nebraska Streets. At 6th and Douglas, a second study was performed after having dropped the late release feature to determine compliance rates under standard timing. A period of one month was allowed between the change of timing and data collection for the second alternative to provide a stabilization period for pedestrian behavior. No observations of pedestrian behavior under early release timing were made.

## Data Analysis

### Delay Analysis

As was done for the allocation of excess pedestrian time, a range of pedestrian and vehicular volumes were used for the early and late release delay studies. These ranged from 0 to 20 pedestrians per cycle and 2 to 8 vehicles per cycle in the right lane. The basic assumptions included:

- perfect pedestrian compliance
- a distribution of pedestrian arrivals similar to that used in the analysis of excess pedestrian time in Chapter II (see Figure 16)

The measurement of delay was primarily mathematical and included both vehicle and pedestrian delay. It was felt that a mathematical analysis with several simplifying assumptions would provide a general basis on which to evaluate effects of early and late release. It was also felt that such an approach would allow the measurement of total person-delay for a wide range of pedestrian and vehicular volumes. It was necessary for the purpose of evaluating these alternatives to limit this phase to an analysis of the extremes, that is to examine each case under the conditions most favorable to that case. The output could then be expressed in terms of ranges of delay values with the most probable value somewhere in between. A compliance study was performed at one location already equipped with the late release feature but this analysis was not conducted for early release. For the former, pedestrian safety would be the primary objective while for the latter, reducing vehicular right-turn delay would be the major emphasis.

The reasoning behind releasing pedestrians before vehicles is to allow pedestrians traveling in the same direction as vehicles to pass the zone of conflict prior to the release of right-turn vehicles. Pedestrians from the opposite end of the crosswalk would enter the zone of conflict shortly after vehicles are released (this would vary depending on street width), but they are better equipped to react to the movement of right-turning vehicles because of more direct eye contact with the vehicles. Holding vehicles while pedestrians are released would necessitate a separate signal indication for right turning vehicles unless through and left turn movements are delayed as well. It has been assumed here that through vehicles, even if in the right lane, are permitted to move during the early release interval unless the path is blocked by a right turning vehicle. The logic behind releasing pedestrians late is to permit several vehicles to turn before there is a chance for pedestrian conflict, possibly increasing capacity and reducing vehicle delay. It would be most desirable, although not necessary, to provide a green right turn arrow for the initial vehicle interval. This arrow would revert to the normal green indication several seconds before the pedestrian WALK commences to inform vehicles that they no longer have the right-of-way.

## Data Collection

The delay data collected for the study of WALK and clearance interval time was incorporated to estimate vehicular delay for the analysis of early and late release

# CHAPTER III

# ALTERNATIVE PEDESTRIAN SIGNAL PHASING SCHEMES

Although the combined pedestrian-vehicular interval covered in Chapter II is by far the predominant type of pedestrian signal phasing used in the United States, other types of phasing are also being used. This chapter examines several alternative phasing schemes including:

- Early release of pedestrians with respect to vehicles.
- Late release of pedestrians with respect to vehicles.
- Scramble timing.
- Timing for the partial crossing of wide, channelized streets.

The analysis of early and late release phasing have been combined into a single section. All of these alternatives are defined within the appropriate section.

## EARLY AND LATE RELEASE OF PEDESTIRANS WITH RESPECT TO VEHICLES

### Study Approach

One method of reducing the conflict between pedestrians and vehicles, and possibly of reducing delay, is to provide a separation between their movements. This separation need not be provided by an exclusive pedestrian phase, but may be accomplished by either of the following:

- Allowing pedestrians to leave the curb before vehicles turning right are released (subsequently referred to as "early release").
- Holding pedestrians at the curb until several of the right turning vehicles have passed the crosswalk (subsequently referred to as "late release").

These types of phasing were evaluated in terms of vehicle and pedestrian delay and also in terms of pedestrian compliance for late release. The approach to the delay analysis was to develop a general level of delay produced by each case, and to compare these to the delay produced under the combined pedestrian-vehicular interval.

safety to the entire population as a whole. Much opportunity is available for imaginative solutions which will contribute toward this goal.

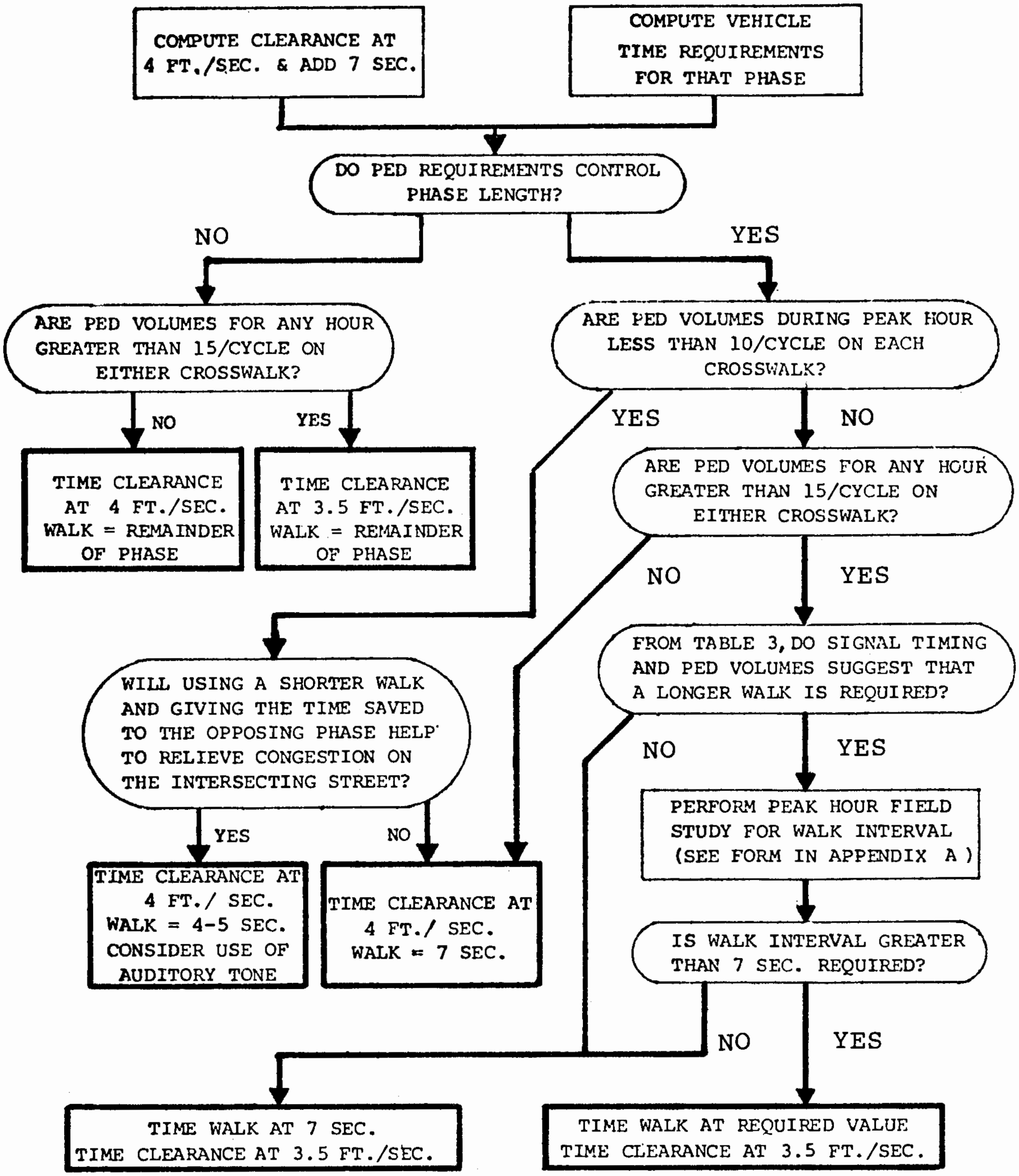
It should also be pointed out that the primary objective of most pedestrians is to minimize their delay, sometimes even at the expense of greater risk. If a pedestrian perceives that no vehicles will block his path in crossing the street during a prohibitive phase, he will often proceed to do so, if it will significantly reduce his delay. Insofar as possible, timing should seek to satisfy these desires, because it is very difficult to change this natural tendency in pedestrian behavior. One very effective way in which timing can respond to these desires is through minimizing cycle lengths. This will guarantee that only a short interval will exist before another opportunity arises for the pedestrian to cross legally. The fact that this also tends to minimize vehicle delay, given adequate capacity, has been reported in other research. Minimizing pedestrian delay will almost always result in a greater degree of safety, since the pedestrian will be more willing to comply with the devices provided to control his behavior.

interval should be reduced to this value only if doing so will help to relieve traffic congestion on the opposing phase. For example, the time by which the WALK interval is shortened on the minor street phase could be added to the green time on the major street phase. This would be done if it reduces the length of long, peak hour vehicle queues. If no major queuing problems exist, the WALK should remain at 7 seconds. If volumes are between 10 and 15 pedestrians per cycle, the 7-second minimum WALK and clearance timed at 4.0 ft./sec. should also be used.

If pedestrian time requirements control the length of the phase and pedestrian volume is more than 15 per cycle, the necessity of extending the WALK interval beyond 7 seconds must be examined. The methodology described on page 26 would be used to make this determination. The WALK interval would be set as required even though it may dictate a longer phase. Whether a WALK interval of over 7 seconds is used or not, the clearance interval would be timed at 3.5 ft./sec. At intersections where there is an unusually high proportion of slow walkers (i.e., elderly, handicapped, etc.), it may be advisable to use the 3.5 ft./sec. walking speed assumption with less than 15 pedestrians per cycle.

It may at first appear that this methodology will result in frequent deviations from the normal minimum pedestrian intervals. This is not likely to be the case, however, as can be seen upon close examination of pedestrian volume levels. For example, crosswalks with peak hour volumes greater than 15 pedestrian per cycle will be relatively rare except in the CBD's of major urban areas. Consequently, most clearance intervals would still be timed at 4.0 ft./sec. As stated in Chapter II, occasions in which a WALK interval longer than 7 seconds will be required are very few. The number of times a shorter WALK would be justified is also quite small. Although a shorter WALK is frequently acceptable in the basis of pedestrian volumes, the vehicle congestion criterion will seldom be met.

Dealing with the pedestrian is probably one of the more challenging tasks in the traffic engineering field. There is a wide variation in the characteristics of pedestrians which must be accommodated. Signalization must accommodate the elderly, normally the group having the slowest walking speeds and reaction times, and yet accommodate the desires of the opposite extreme, the fast walkers. It must be recognized that it is virtually impossible to serve both extremes at one time. Therefore, the goal of signal timing should be to provide the greatest degree of



NOTE: If this process gives different answers for various hours of day and if hardware configuration will allow, change timing by time-of-day.

FIGURE 20. PEDESTRIAN SIGNAL TIMING FOR WALK AND CLEARANCE INTERVALS.

will not complete their crossing where only a minimum WALK and clearance interval exist. At volumes above this level, a 3.5 ft./sec. walking speed assumption should be used in timing the clearance intervals (see page 52).

- Far side parking lanes should <u>not</u> be subtracted from the street width to be crossed in timing the clearance interval unless geometrics or operational constraints prelude the lanes from being traversed by turning vehicles (see page 47).

- Where excess pedestrian time exists, it should <u>always</u> be allocated to the WALK interval rather than to the clearance interval. (It must be remembered that the clearance interval can be timed using either a 4.0 or 3.5 ft./sec. walking speed assumption depending on pedestrian volumes).

Figure 20 is a flow chart depicting the methodology for pedestrian signal timing. The initial step is no different than what is routinely done at present, that is, the computation of pedestrian time requirements (7-second WALK and clearance at 4.0 ft./sec.) and vehicle time requirements. These two phase times are then compared. If the vehicle time requirements are greater than those for pedestrians, the best allocation of the excess pedestrian time must be determined. The peak hour pedestrian volume should be determined and divided by the number of cycles per hour to obtain the average volume per cycle. The short pedestrian count technique presented on page 140. is useful for providing accurate volume counts if none are available. If pedestrian volumes are greater than 15 per cycle on either crosswalk, the clearance interval should be timed at 3.5 ft./sec.; otherwise, the original 4.0 ft./sec. assumption is adequate. The remainder of the excess time should be allocated <u>entirely</u> to the WALK interval.

If pedestrian time requirements control the length of the phase, that is, the minimum WALK and clearance intervals are used, a larger number of alternatives must be considered. If pedestrian volumes on each crosswalk during the peak pedestrian hour are less than 10 per cycle, a 4-5 second WALK interval will be adequate to accommodate the discharge of pedestrian queues. However, the WALK

In summary, it is recommended that any excess pedestrian time always be allocated to the WALK interval. The only viable exception would be to time the clearance interval at 3.5 feet per second where heavy pedestrian volumes exist (greater than 15 pedestrians per cycle as defined in the section on the minimum clearance interval).

In addition to the above conclusions, this chapter has presented the tools by which a detailed analysis of pedestrian delay and vehicle right turn delay can be performed. The delay curves presented in Figure 14 are viewed to be a very important and practical finding. The procedures used in the delay calculations may be applied by other traffic engineers with variations in assumptions to conform to a given intersection with other cycle lengths and splits than those assumed in this study.

## SUMMARY OF TIMING FOR A COMBINED PEDESTRIAN-VEHICULAR INTERVAL

This section contains in capsule form, a methodology for the timing of pedestrian WALK and clearance intervals which is responsive to the needs of both pedestrians and vehicles. The timing principles have already been stated in this chapter but are interfaced in flow chart form so that the progression of the methodology can be more easily discerned. The basic principles upon which the methodology is based are stated below.

- A 7-second WALK interval is adequate to accommodate the discharge of pedestrian queues from the curb at a vast majority of signalized intersections. A procedure has been presented by which extensions of the minimum WALK interval can be justified (see page 16).

- A 4-5 second WALK interval is adequate to to accommodate the discharge of pedestrian queues when crosswalk volumes are less than 10 pedestrians per cycle. The use of an auditory tone sounded at the beginning of the WALK interval should be considered to alert any inattentive pedestrians (see page 30).

- Platoon walking speeds decrease with increasing pedestrian volume. At a volume level of approximately 15 pedestrins per cycle, a significant number of platoons

All indications from the perspective of overall intersection delay are that the WALK interval should be extended to the point in time at which the minimum clearance rules. The only exception may occur when pedestrian volumes are so high that they do not enable vehicular demand to be served. Even if heavy pedestrian and vehicular right turn volumes do exist, it is doubtful whether providing a shorter WALK will help vehicles since it has been shown that pedestrians tend not to comply with the longer clearance interval.

Conclusions from the safety perspective are somewhat more difficult to draw. The results can be stated in terms of changes in compliance and behavior, but whether these are good indicators of the degree of safety is uncertain. For instance, those who violate the signal most often may consist of those individuals which are most aware of conditions under which it may be relatively safe to violate. They may know how to look for oncoming vehicles and be able to quickly react to potentially hazardous situations. This is an unproven hypothesis, of course, and is used merely to suggest that low compliance and erratic behaviors do not necessarily prove that a certain set of intersection characteristics will create a severe accident hazard.

It can be concluded, however, that compliance is much lower where excess pedestrian time is allocated to the clearance interval rather than to the WALK interval. Pedestrians tend to be aware of the time which remains to cross the street before the opposing phase begins. They will generally accelerate their pace according to the requirements of safely reaching the far curb. This result has significant consequences on the primary potential advantage of allocating the additional time to the clearance interval, this being reduced vehicle delay. In theory, this type of timing would allow right turning vehicles to proceed without pedestrian interference during the latter part of the phase. This would be particularly beneficial where right turn and pedestrian volumes are high. In reality, it appears as if this benefit will almost always be cancelled because of the associated low pedestrian compliance, due either to the lack of knowledge as to intent of the timing or unwillingness of the pedestrian to be delayed. Thus, some other means of dealing with a vehicle right turn delay problem must be found (see Chapter III for potential methods).

was virtually no consistency from location to location. Figure 19 probably gives a better idea of the true results than do the statistical tests, because one would expect trends consistent with the length of the WALK interval to develop if relationships are actually valid. The fact that these relationships did not evolve leads to the conclusion that the behavioral data have not detected any significant differences in the timing alternatives with regard to pedestrian safety. Thus, the weight of the safety analysis must fall on the compliance study alone.

## Conclusions and Recommendations

The conclusions derived with regard to the allocation of excess pedestrian time are as follows:

- In general, allocating excess time to the clearance interval increases total intersection delay.
- Pedestrian compliance significantly decreases with the allocation of excess time to the clearance interval.
- Very few pedestrians starting their crossing during the WALK interval fail to complete their crossing in time, even with a minimum clearance interval.
- Changes in the allocation of excess pedestrian time between the WALK and clearance intervals do not appear to affect compliance to the solid DONT WALK interval.
- The frequency of pedestrian behaviors, as defined in this report, does not appear to vary significantly with changes in the allocation of excess pedestrian time.
- The arrival rate of pedestrians at a signalized intersection crosswalk is not uniform, but is higher just prior to and during the WALK interval (see page 72).
- This analysis has resulted in a method of estimating pedestrian-caused right turn vehicle delay from two-way pedestrian volume.

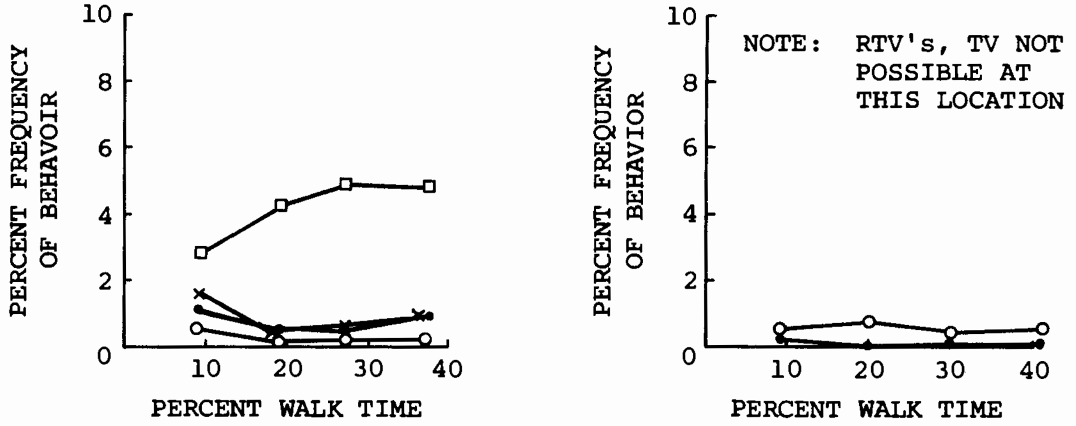
## TABLE 18. RESULTS OF Z TEST ON PEDESTRIAN BEHAVIORS

| Location | Length of WALK Intervals Compared | Z Values | | | | | |
|---|---|---|---|---|---|---|---|
| | | B | RTV | MV (veh.) | MV (ped.) | TV (veh.) | TV (ped.) |
| <u>Washington</u> | | | | | | | |
| 15th & H | 7.2/15.2 | 1.884 | 3.390** | 2.543* | 2.843** | -6.074** | 2.250* |
| | 7.2/21.6 | 2.042* | 2.871** | 2.266* | 3.015** | -8.817** | 3.214** |
| | 7.2/29.6 | 0.820 | 2.084* | 2.757** | 3.666** | -8.580** | 2.246* |
| | 15.2/21.6 | 0.065 | -0.767* | -0.324 | -0.066 | 2.609** | 0.906 |
| | 15.2/29.6 | -1.211 | -1.598 | 0.347 | 0.690 | -2.109* | -0.131 |
| | 21.6/29.6 | -1.356 | -0.897 | 0.668 | 0.804 | -0.608 | -1.096 |
| 20th & M | 7.2/16.0 | 1.701 | - | -0.868 | -0.805 | - | - |
| | 7.2/24.0 | 1.212 | - | 1.252 | 1.391 | - | - |
| | 7.2/32.8 | 1.623 | - | 0.069 | 1.011 | - | - |
| | 16.0/24.0 | -0.587 | - | 2.088* | 2.174* | - | - |
| | 16.0/32.8 | -0.376 | - | 0.920 | 1.901 | - | - |
| | 24.0/32.8 | 0.280 | - | -1.174 | -0.489 | - | - |
| <u>Pheonix/Tempe</u> | | | | | | | |
| Central & Monroe | 7.0/13.0 | -3.397** | -0.772 | -0.102 | 0.286 | -1.296 | -2.441* |
| | 7.0/18.5 | -4.729* | -0.281 | -0.991 | -1.143 | -0.372 | -3.115* |
| | 13.0/18.5 | -1.554 | -0.445 | -0.914 | -1.375 | 0.908 | -0.818 |
| University & Forest | 7.0/21.7 | -1.160 | -0.581 | -1.690 | -1.962 | 0.957 | 1.226 |
| | 7.0/37.8 | -1.236 | 0.064 | - | - | -2.052* | -0.338 |
| | 21.7/37.8 | -0.056 | 0.660 | - | - | -2.903** | -1.582 |
| <u>Buffalo</u> | | | | | | | |
| Pearl & Church | 7.0/14.7 | -1.573 | 1.726 | -1.270 | -2.033* | -0.189 | -0.357 |
| | 7.0/23.1 | -1.327 | 1.744 | -0.810 | -0.984 | -1.946 | -1.813 |
| | 14.7/23.1 | 0.242 | 0.008 | 0.438 | 1.020 | -1.711 | -1.405 |
| Grant & Ferry | 7.2/14.4 | 3.302** | 0.079 | 0.853 | 1.694 | 3.334** | 3.648** |
| | 7.2/21.6 | 1.830 | -1.312 | 2.336* | 1.973* | 5.918** | 5.344** |
| | 14.4/21.6 | -1.454 | -1.429 | 1.461 | 0.321 | 2.642** | 1.864 |

Note: *Indicates significant difference at .05 level

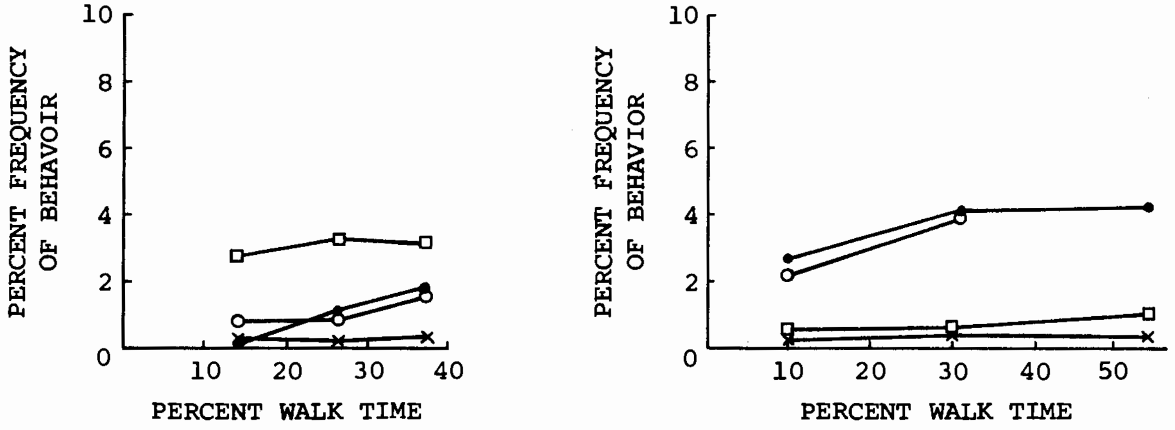
**Indicates significant difference at .01 level

Negative sign indicates that mean % frequency of 2nd alternative is greater than the first. No sign indicates the opposite.



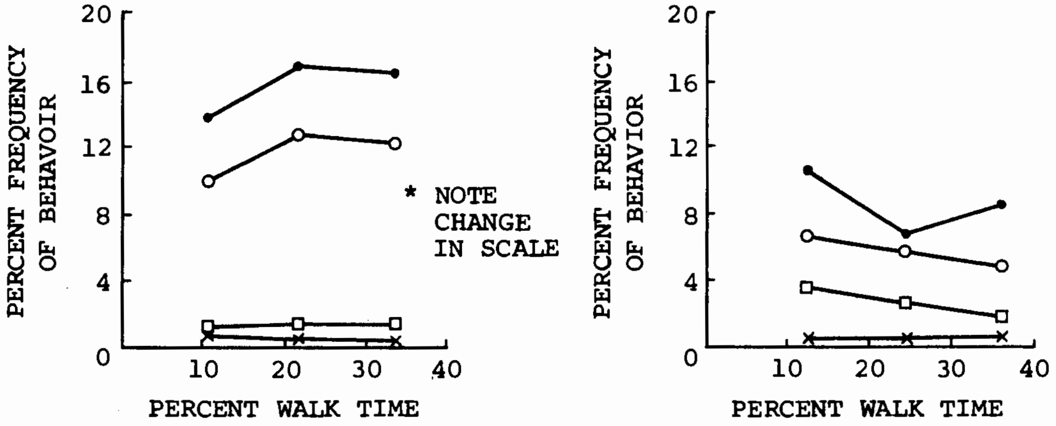
15th & H STREETS

20th & M STREETS



CENTRAL AVE. & MONROE STREET

UNIVERSITY AVE. & FOREST AVE.



PEARL & CHURCH STREETS

GRANT & FERRY STREETS

LEGEND

○ MVM □ TV ÷ 10

● B × RTV

FIGURE 19. GRAPHICAL REPRESENTATION OF PEDESTRIAN BEHAVIORS

TABLE 17. FREQUENCY AND PERCENT FREQUENCY OF PEDESTRIAN BEHAVIOR.

| Location | Length of WALK Intervals (sec.) | Frequency/Percent Frequency | | | | | |
|---|---|---|---|---|---|---|---|
| | | B | RTV | MVM (veh.) | MVM (ped.) | TV (veh.) | TV (ped.) |
| <u>Washington, D. C.</u> | | | | | | | |
| 15th & H | 7.2 | 19/1.1 | 29/1.6 | 14/0.5 | 14/0.8 | 194/24.4 | 194/10.9 |
| | 15.2 | 12/0.5 | 12/0.5 | 4/0.1 | 4/0.1 | 194/40.6 | 194/8.7 |
| | 21.6 | 14/0.5 | 19/0.7 | 5/0.2 | 5/0.2 | 213/49.2 | 213/8.0 |
| | 29.6 | 23/0.8 | 26/0.9 | 3/0.1 | 3/0.1 | 246/47.2 | 246/8.9 |
| 20th & M | 7.2 | 5/0.2 | - | 14/0.5 | 14/0.7 | - | - |
| | 16.0 | 1/0.0 | - | 14/0.7 | 19/0.9 | - | - |
| | 24.0 | 2/0.1 | - | 8/0.3 | 8/0.4 | - | - |
| | 32.8 | 2/0.1 | - | 13/0.5 | 13/0.5 | - | - |
| <u>Pheonix/Tempe</u> | | | | | | | |
| Central & Monroe | 7.0 | 5/0.2 | 8/0.4 | 5/0.9 | 5/0.2 | 115/29/9 | 165/7.8 |
| | 13.0 | 22/1.1 | 5/0.2 | 4/0.9 | 4/0.2 | 203/33.5 | 203/9.9 |
| | 18.5 | 26/1.7 | 5/0.3 | 7/1.6 | 7/0.5 | 167/31.0 | 167/10.8 |
| University & Forest | 7.0 | 11/2.7 | 1/0.2 | 11/2.1 | 11/2.7 | 33/6.2 | 33/8.0 |
| | 21.7 | 17/4.1 | 2/0.4 | 22/4.0 | 22/5.3 | 24/4.8 | 24/5.8 |
| | 27.8 | 19/4.2 | 1/0.2 | - | - | 39/9.7 | 39/8.6 |
| <u>Buffalo</u> | | | | | | | |
| Pearl & Church | 7.0 | 108/13.9 | 6/0.8 | 115/11.0 | 115/14.8 | 61/11.7 | 61/7.9 |
| | 14.7 | 115/16.9 | 1/0.1 | 128/12.8 | 128/18.8 | 57/12.1 | 57/8.4 |
| | 23.1 | 113/11.4 | 1/0.1 | 115/12.1 | 115/16.7 | 73/15.9 | 73/7.7 |
| Grant & Ferry | 7.2 | 110/10.3 | 1/0.1 | 100/6.4 | 100/9.4 | 149/32.0 | 149/14.0 |
| | 14.4 | 77/6.5 | 1/0.1 | 88/5.7 | 88/7.4 | 108/22.4 | 108/7.0 |
| | 21.6 | 89/8.1 | 4/0.4 | 78/4.6 | 78/7.1 | 77/15.7 | 77/6.9 |

characterized by more non-compliant pedestrians. The Grant and Ferry Street crosswalk was the shortest of all, but the higher traffic volume prevented violations from exceeding the rate at Pearl and Church.

Although the violation rates could be explained intuitively, no definite relationship among the variables could be developed. It is evident, however, that apportioning the WALK and clearance times in various ways did not affect the number of pedestrians starting during the solid DONT WALK interval.

## Pedestrian Behaviors

The analysis of pedestrian behaviors is the second way in which the safety aspect of timing changes can be weighed. Volume II of the Final Report explains the rationale behind using behaviors as indicators of accident potential. Table 17 shows the frequency and percent frequency of the four behaviors for each location and timing alternative. Percent frequency for the MvM and TV behaviors was derived using both vehicular volume (through vehicles for MVM and turning vehicles for TV) and pedestrian volume. Both of these are essentially vehicle counts, but pedestrian exposure is the parameter of primary interest. It is not clear if percent frequency derived using vehicle volume is any more valid than using pedestrian volume. Figure 19 portrays the percent frequency graphically to identify trends.

As shown in Figure 19, it is very difficult to identify a cause/effect relationship between the timing changes and changes in behavior. The behaviors do not consistently rise or fall in one direction or another with changes in the allocation of WALK and clearance times.

The areas in which one might expect the most significant changes are the RTV and TV behaviors. It might be expected that the incidence of these would be lower with a longer clearance interval since, theoretically, pedestrians would walk only during the shorter WALK interval. The fact that this did not surface in the data can be readily explained by the low level of compliance to the clearance intervals.

The behavior data was subjected to a series of statistical tests to verify what had been generally observed. The results of a Z test are presented in Table 18. The test revealed significant differences in some of the behaviors in some of the cities, but there complete their crossing in time, even under the minimum clearance alternatives. Part of the reason for this is that most pedestrians begin walking as soon as the WALK interval is displayed. This normally gives them several seconds in addition to the clearance interval in which to complete their crossing. Thus, this extra time can permit even those walking slower than 4.0 feet per second to cross successully. The only pedestrians usually in danger of not crossing in the proper time are the very slow walkers and the moderate speed walkers who start at the end of the WALK interval.

The percentage of pedestrians starting during the prohibitive phase, the solid DONT WALK interval, varies significantly by city and by location (see Table 16). The intersection of 15th and H Streets in Washington has the lowest percentage of all the crosswalks, followed closely by Monroe Street and Central Avenue in Phoenix. The two Buffalo intersections had by far the highest percentages in this category. There are three basic factors to which this result can be attributed. One that probably has an effect but can be only subjectively verified is the previously mentioned "regional attitude" factor. The other two can be more easily related to the results of the data and include the width of the street and the volume of traffic passing the crosswalk during the prohibitive pedestrian phase. The narrower the street and the lower the volume, the greater opportunity a pedestrian will have to cross the street. The crosswalk at 15th and H Streets was not only quite wide but also was heavily traversed by vehicles, explaining the low percentages. The Monroe Street crosswalk in Phoenix was also fairly long but had much less in the way of traffic volume, yielding the slightly higher percentages. The crosswalk at 20th and M Streets in Washington was shorter and produced a greater violation rate even though traffic volumes were quite substantial. The fact that it was a one-way street may have contributed to this. The crosswalk observed at the intersection of University and Forest Avenues in Tempe was also quite short. This, in combination with the very low traffic volumes, produced a significant violation rate, even with the more compliant attitudes which typically exist in that section of the country. The intersection of Pearl and Church Streets in Buffalo encountered the highest violation rate of the solid DONT WALK interval. Although the street was as wide as 20th and M Streets in Washington, D.C., it had a lower traffic volume and was in a region

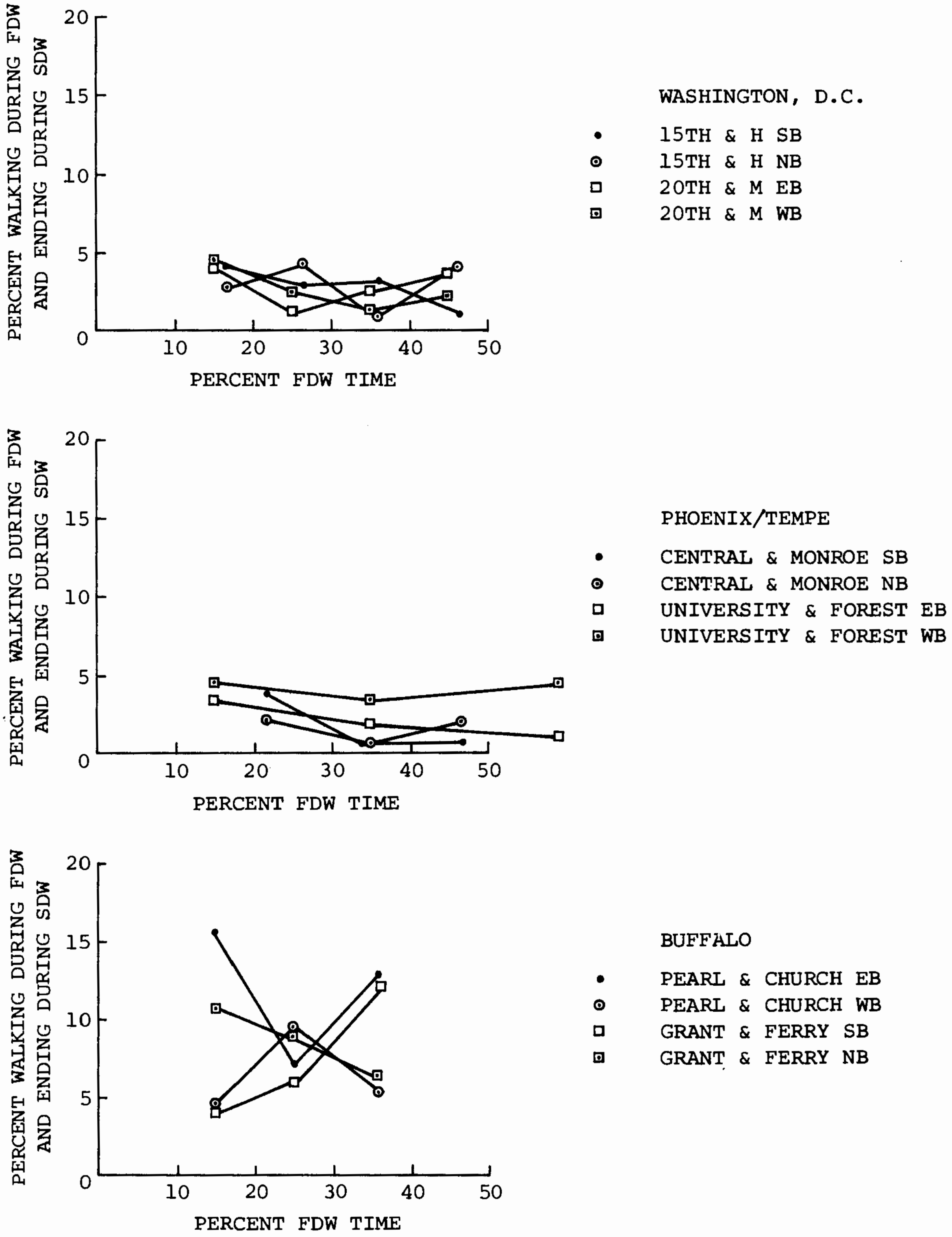


FIGURE 18. EFFECT OF TIMING ON THE NUMBER OF PEDESTRIANS IN THE STREET AT THE END OF THE PHASE.

to the clearance interval up to a certain interval length (specifically, an interval timed close to the minimum clearance) and that longer clearance times encourage noncompliance. The compliance achieved at minimum clearance time is likely to be the maximum which would normally be achieved at the intersection observed.

The reasons for the significant decrease in compliance for clearance intervals longer than the minimum appears to be that the average pedestrian is not "fooled" into thinking that he has less time to cross the street before vehicles on the cross street are released. Pedestrians that have previously used the same crosswalk have come to know quite accurately how much time they will require to safely complete their crossing. Consequently, if they know that there is a long clearance interval, they will not hesitate to begin their crossing during that interval.

Even though the compliance of pedestrians beginning their crossing dropped with increasing clearance times, this in itself does not mean that the hazard to pedestrians increased. A hazard is more likely indicated by the number of pedestrians which are in the street when they are least expected by drivers, that is, during the solid DONT WALK interval. From Table 16, it can be seen that a fairly small percentage of all pedestrians start on the clearance interval but end during the solid DONT WALK intervals. The percentage is below five for all but Buffalo data.

A plot was then made to determine if the allocation of excess pedestrian time had a bearing on the relationship between the percentage of pedestrians beginning to cross during clearance but ending during solid DONT WALK and the times allocated to the clearance interval (Figure 18). The percentage seems to bear no relationship to the allocation of excess pedestrian time. In the Washington, D.C. and Phoenix data, there is a tendency for the percentages for the minimum WALK alternatives to be slightly higher, but this is by no means conclusive. One must rather conclude that pedestrians are just as likely to be caught by the solid DONT WALK interval with minimum clearance as with the minimum WALK alternative. It is highly probable, however, that clearance times of less than minimum would begin to create substantial increases in this percentage.

Hazards to the pedestrian are also indicated by those starting on WALK but finishing during solid DONT WALK and by all those starting on solid DONT WALK. From Table 16, <u>very few persons starting on WALK fail to</u>

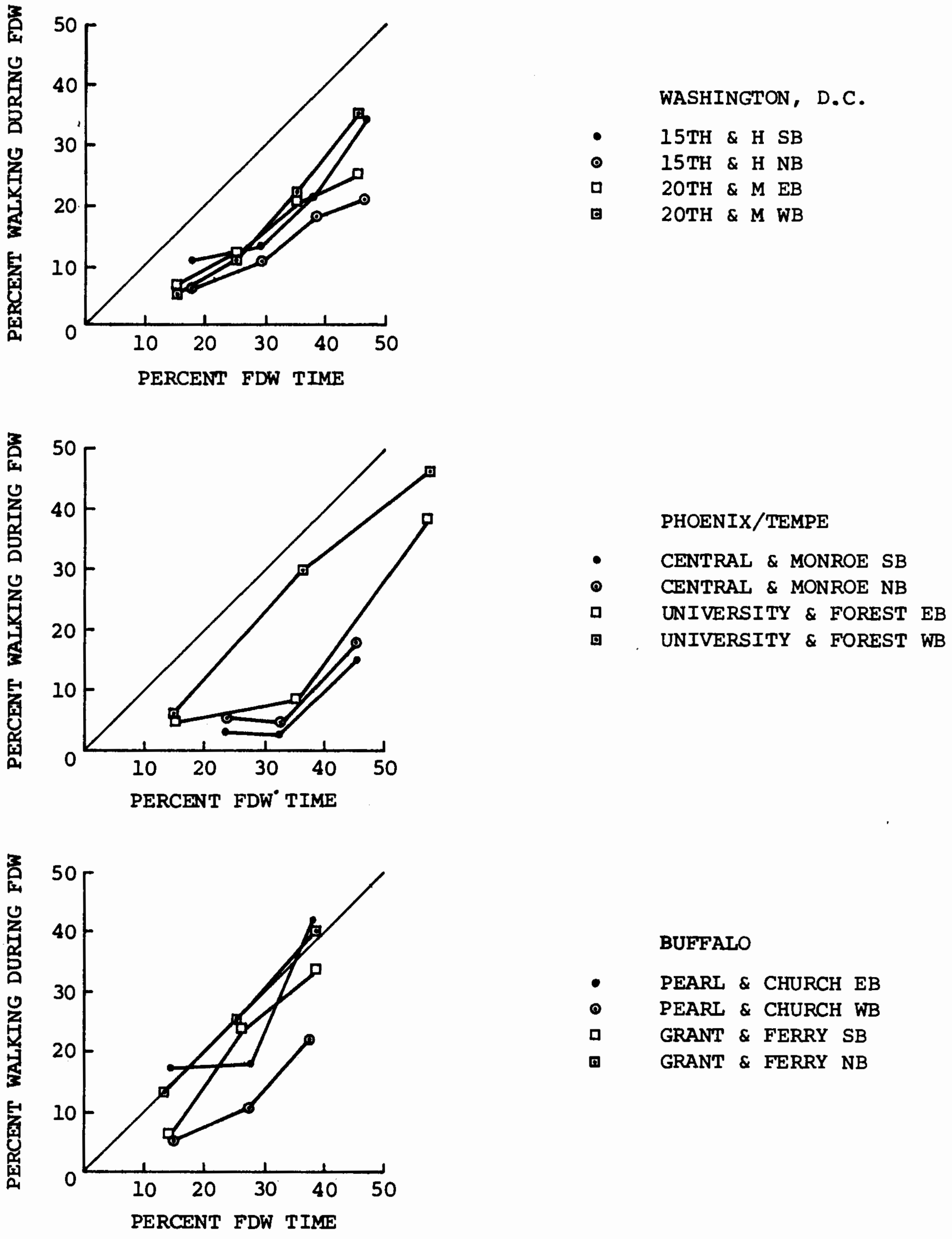


FIGURE 17. RELATIONSHIP BETWEEN COMPLIANCE TO THE CLEARANCE INTERVAL AND SIGNAL TIMING.

# TABLE 16. RESULT OF PEDESTRIAN SIGNAL TIMING COMPLIANCE STUDY.

| Signal Timing Alternatives (in seconds) | | | Compliance of Southbound or Eastbound Peds (Frequency/% Frequency) | | | | | | | Compliance of Northbound or Westbound Peds (Frequency/% Frequency) | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Begin on W | | Begin on FDW | | Begin on SDW | | | Begin on W | | Begin on FDW | | Begin on SDW | | |
| W | FDW | SDW | End on W or FDW | End on SDW | End on FDW | End on SDW | End on SDW | End on W or FDW | Total Volume | End on W or FDW | End on SDW | End on FDW | End on SDW | End on SDW | End on W or FDW | Total Volume |
| **D.C. 15th & H** | | | | | | | | | | | | | | | | |
| 7.2 | 37.6 | 35.2 | 595/65.5 | 0/0.0 | 295/32.5 | 12/1.3 | 4/.4 | 2/.2 | 908 | 685/78.0 | 0/0 | 151/17.2 | 32/3.6 | 4/0.5 | 5/.6 | 878 |
| 15.2 | 29.6 | 35.2 | 892/78.0 | 0/0.0 | 214/18.7 | 32/2.8 | 0/0 | 5/.4 | 1143 | 880/81.9 | 0.0 | 180/16.7 | 11/1.0 | 4/0.4 | 0/0 | 1075 |
| 21.6 | 23.2 | 35.2 | 1148/83.6 | 0/0.0 | 188/13.7 | 34/2.5 | 1/.1 | 2/.1 | 1373 | 1105/86.3 | 0/0 | 106/8.3 | 52/4.1 | 2/0.2 | 16/1.2 | 1281 |
| 29.6 | 15.2 | 35.2 | 1128/87.7 | 0/0.0 | 111/7.9 | 59/4.2 | 1/.1 | 1/.1 | 1400 | 1271/92.2 | 0/0 | 63/4.6 | 37/2.7 | 8/0.6 | 0/0 | 1379 |
| **D.C. 20th & M** | | | | | | | | | | | | | | | | |
| 7.2 | 36.8 | 36.0 | 715/69.2 | 0/0 | 238/23.0 | 34/3.3 | 46/4.5 | 0/0 | 1033 | 667/64.0 | 0/0 | 332/31.9 | 24/2.3 | 13/1.2 | 6/.6 | 1042 |
| 16.0 | 28.0 | 36.0 | 807/77.5 | 0/0 | 188/18.1 | 27/2.6 | 16/1.5 | 3/.3 | 1041 | 820/75/6 | 0/0 | 222/20.5 | 14/1.3 | 16/1.5 | 13/1.2 | 1065 |
| 24.0 | 20.0 | 36.0 | 934/83.0 | 0/0 | 142/12.6 | 16/1.4 | 29/2.6 | 4/.4 | 1125 | 859/82.8 | 0/0 | 109/10.5 | 25/2.4 | 34/3.3 | 10/1.0 | 1037 |
| 32.8 | 11.2 | 36.0 | 1392/91.3 | 2/.1 | 47/3.1 | 57/3.7 | 25/1.6 | 2/.1 | 1525 | 1170/90.0 | 16/1.2 | 15/1.2 | 59/4.5 | 25/1.9 | 15/1.2 | 1300 |
| **Phoenix - Central & Monr** | | | | | | | | | | | | | | | | |
| 7.0 | 23.0 | 20.0 | 975/83.9 | 0/0 | 166/14.3 | 12/1.0 | 3/.3 | 8/.7 | 1164 | 763/81.0 | 0/0 | 144/15.3 | 21/2.2 | 3/.3 | 12/1.2 | 942 |
| 13.0 | 10.0 | 20.0 | 1046/95.2 | 10/.9 | 22/2.0 | 7/.6 | 8/.7 | 6/.6 | 1099 | 880/94.4 | 5/.5 | 32/3.4 | 6/.6 | 4/.4 | 5/.5 | 932 |
| 18.5 | 11.5 | 20.0 | 764/93.3 | 11/1.3 | 10/1.2 | 24/2.9 | 3/.4 | 7/.9 | 819 | 669/92.7 | 4/.6 | 19/2.6 | 16/2.2 | 3/.4 | 11/1.5 | 722 |
| **Tempe - Univ. & Forest** | | | | | | | | | | | | | | | | |
| 7.0 | 39.9 | 23.1 | 94/49.2 | 0/0 | 73/38.2 | 2/1.1 | 11/5.8 | 11/5.8 | 191 | 91/42.9 | 0/0 | 90/42.5 | 10/4.7 | 11/5.2 | 10/4.7 | 212 |
| 21.7 | 25.2 | 23.1 | 163/85.3 | 0/0 | 10/5.2 | 4/2.1 | 3/1.6 | 11/5.8 | 191 | 129/58.6 | 0/0 | 58/26.4 | 8/3.6 | 16/7.3 | 9/4.1 | 220 |
| 37.8 | 9.1 | 23.1 | 187/87.8 | 2/.9 | 3/1.4 | 7/3.3 | 7/3.3 | 7/3.3 | 213 | 192/80.7 | 13/5.5 | 2/.8 | 10/4.2 | 14/5.9 | 7/2.9 | 238 |
| **Buffalo - Pearl & Church** | | | | | | | | | | | | | | | | |
| 7.0 | 26.6 | 36.4 | 103/25.9 | 0/0 | 117/29.4 | 50/12,6 | 80/20.1 | 48/12.1 | 398 | 123/32.6 | 0/0 | 67/17.8 | 21/5.6 | 98/26.0 | 68/18.0 | 377 |
| 14.7 | 18.9 | 36.4 | 123/35.2 | 0/0 | 40/11.5 | 25/7.2 | 105/30.1 | 56/16.1 | 349 | 108/32.6 | 0/0 | 29/8.8 | 31/9.4 | 90/27.2 | 73/22.1 | 331 |
| 23.1 | 10.5 | 36.4 | 166/43.6 | 2/.5 | 8/2.1 | 6/ 1.6 | 88/23.1 | 56/14.7 | 381 | 164/53.4 | 0/0 | 3/1.0 | 15/4.9 | 76/24.8 | 49/16.0 | 307 |
| **Buffalo - Grant & Ferry** | | | | | | | | | | | | | | | | |
| 7.2 | 22.8 | 30.0 | 276/50.3 | 0/0 | 115/20.9 | 68/12.4 | 54/9.8 | 36/6.6 | 549 | 228/44.4 | 2/.4 | 175/34.0 | 32/6.2 | 35/6.8 | 41/8.0 | 514 |
| 14.4 | 15.6 | 30.0 | 347/52.0 | 0/0 | 111/16.6 | 42/6.3 | 53/7.9 | 114/17.1 | 667 | 319/61.6 | 0/0 | 78/15.1 | 47/9.1 | 39/7.5 | 36/6.9 | 518 |
| 21.6 | 8.4 | 30.0 | 434/78.0 | 0/0 | 9/1.6 | 27/4.8 | 53/9.5 | 35/6.3 | 558 | 871/68.2 | 0/0 | 20/3.3 | 58/10.7 | 45/8.3 | 50/9.2 | 544 |

Note: W = WALK interval
FDW = flashing DONT WALK or clearance interval
SDW = solid DONT WALK interval

## Pedestrian Compliance

Table 16 presents the results of the compliance study at the 6 locations. Each four-hour data collection period has been summarized by site and alternative. The compliance data are arranged from the minimum clearance to the minimum WALK alternative to facilitate comparison.

The primary comparison is between the percent compliance for the three or four alternatives at each location. This describes how pedestrians generally reacted to the changes in timing. Looking first at the percentage beginning on the WALK interval, it can be seen that the percentage almost universally decreases with decreasing WALK time. In general, this decrease in the percentage of pedestrians beginning their crossing on the WALK interval resulted in an increase in the percentage beginning on the clearance interval.

It is not surprising that the percentage crossing during the clearance interval should increase since the opportunity to cross during that time also increases. One would hope, however, that the increase would not be directly proportional to the increase in time, for this would indicate minimal compliance to the WALK interval.

To test the extent of the increase in the percentage of pedestrians walking during the clearance interval, plots were made of that percentage versus the percent of cycle time allocated to the clearance interval (Figure 17). The percentage of pedestrians includes those completing their crossing during any of the three intervals. In addition, each figure is split by direction to permit comparisons in compliance to be made in this way as well.

A 45 degree line with its intercept at the origin represents the condition of equal percentages. Points falling on this line indicate that pedestrians virtually ignore the clearance interval and walk whenever they please. From Figure 17 one can see that only the Buffalo data contains points which are on or above that line. Some of the Buffalo data and all of the data from Phoenix and Washington, D.C. fall below the line, indicating at least some degree of compliance. It should be noted, however, that the percentage walking during the clearance interval increases at approximately the same rate as the percentage of increase in time allocated to that interval. It appears that the slopes of the lines connecting the data points approximate that of the 45 degree line but that the intercept along the X axis is shifted to the right. This is a very good indication that pedestrians generally comply

## TABLE 15. DELAY EFFECT OF THE MINIMUM WALK ALTERNATIVE RELATIVE TO THE MINIMUM CLEARANCE ALTERNATIVE.

| Pedestrian Volume Per Cycle | Increase In Ped Delay per cycle (person-sec.) | Decrease in vehicle delay for various veh. volumes per cycle (delay in person-sec.)* | | | | Increase in total delay for various veh. volumes per cycle under early WALK termination | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 2 | 4 | 6 | 8 | 2 | 4 | 6 | 8 |
| 0 | -- | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 2 | 31 | 0 | 0 | 0 | 0 | 31 | 31 | 31 | 31 |
| 5 | 75 | 0 | 2 | 9 | 18 | 75 | 73 | 66 | 57 |
| 10 | 148 | 0 | 9 | 24 | 39 | 148 | 139 | 124 | 109 |
| 20 | 299 | 5 | 23 | 41 | 231** | 294 | 276 | 258 | 68 |

*NOTE: Person delay assumes an average vehicle occupancy of 1.5 persons per vehicle.

**This significant jump in vehicle delay arises out of the inability of the 7th and 8th vehicles to make their turn during this cycle. They are assumed to make their turn during the following cycle.

and the differences between the two would be obtained to produce the delay comparison between the two alternatives.

### Overall Delay Results

Table 15 shows the results of the delay computations and summarizes the increase in total person delay for the minimum WALK alternative compared to the minimum clearance alternative. Even for the largest vehicle and pedestrian volumes, the reduction in vehicle delay with minimum WALK falls short of counteracting the increased pedestrian delay. It should be noted, however, that at the highest vehicle and pedestrian volumes, the vehicle delay value increases significantly (see Table 15 for 20 pedestrians and 8 vehicles per cycle). This occurs because not all the vehicles can make their turn during the first cycle, due to the magnitude of the pedestrian interference. Consequently, they are forced to wait for the next cycle, significantly adding to vehicle delay.

In summary of the delay analysis, allocating excess pedestrian time to the clearance interval rather than to the WALK interval will significantly increase total intersection delay. The increases caused to pedestrian delay will always outweigh any decreases in vehicle delay unless right turn vehicle volumes and pedestrian volumes are high (over 8 vehicles and 20 pedestrians per cycle for the example used here). Where pedestrian compliance to this longer clearance interval is low, the increase in delay will be lower, but the very purpose of allocating time to the clearance interval will have been defeated. A discussion of pedestrian compliance to clearance intervals longer than the minimum are discussed in the next section.

One additional element which might be considered in any analysis of overall intersection person-delay is the relative cost of delay to pedestrians and vehicles. One may argue that it is important to give preference to the group which undergoes greater loss because of delay. The delays used here have not been weighted by any type of cost factors. This could be easily done, however, from data compiled in references 4 and 6. However, it is doubtful whether this additional factor would alter any of the conclusions stated in the paragraph above.

## TABLE 14. SAMPLE CALCULATION FOR VEHICLE RIGHT TURN DELAY WITH PEDESTRIAN VOLUME OF 10 PER CYCLE.

|  | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| Vehicle Volume per cycle (all right turn) | $t_2$ for each vehicle with no ped interference | $t_2$ for each vehicle with 10 peds per cycle (from Fig. 14) | Delay for each vehicle (Col. 2 minus col. 1) | Cumulative Vehicle Delay | Cumulative Person Delay (Col. 4 x 1.5) |
| 1 | 5 | 13 | 8 | 8 | 12 |
| 2 | 8 | 19 | 11 | 19 | 29 |
| 3 | 11 | 24 | 12 | 32 | 48 |
| 4 | 14 | 29 | 15 | 47 | 71 |
| 5 | 17 | 33 | 16 | 63 | 95 |
| 6 | 20 | 36 | 16 | 79 | 119 |
| 7 | 23 | 39 | 16 | 95 | 143 |
| 8 | 26 | 42* | 16 | 111 | 167 |

*Note: This last vehicle may or may not make it through in a 40-second cycle depending on the driver's aggressiveness.

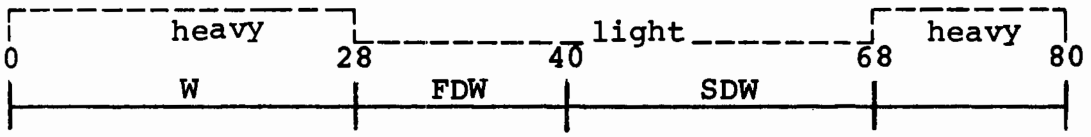
pedestrian-caused delay, or at t = 12. Using the t = 12 curve from Figure 14 would give the second vehicle an additional 3 seconds delay. The third vehicle would now arrive 11 seconds later than normal or at t = 18. This vehicle would incur about 2 seconds of delay. This delay plus the 3-second headway would then be added to determine the arrival time of the fourth vehicle and so on. Total delay for a given volume is the sum of delays for each individual vehicle up to that volume. Thus, for the queue of 6 vehicles, the total vehicle delay would be 79 seconds as indicated in the sample calculation in Table 14. An occupancy value of 1.5 persons per vehicle was then used to translate vehicle-seconds of delay into person-seconds of delay for vehicles.

The decrease in vehicle delay for the alternative terminating the WALK interval at 7 seconds would accrue toward the end of the phase. Given the 52 foot street width and perfect pedestrian compliance, there would theoretically be no pedestrians remaining in the street after about 18 seconds into the phase. Therefore, delay for all times after this interval was defined as zero for this alternative. For the minimum clearance alternative, pedestrians can begin their crossing later in the interval, possibly increasing right turn delay.

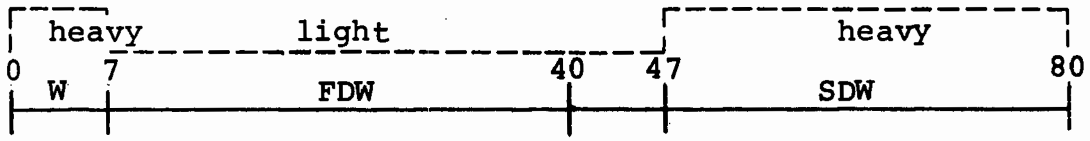
Calculation of Pedestrian Delay

The next step was the calculation of pedestrian delay. The two basic assumptions included perfect pedestrian compliance and the arrival rates described in Figure 16. Pedestrian delay was computed by multiplying the number of pedestrians arriving during one of the two DONT WALK intervals by the average time between their arrival and the next WALK interval. For example, with the minimum clearance alternative and 20 pedestrians per cycle, 13.33 pedestrians would arrive on the average during the heavy arrival period with the remainder arriving during the light arrival period. The average delay for the 6.67 pedestrians would be the mean of the two extreme delays times, that is $\frac{52 + 12}{2}$ = 32 seconds. Multiplying this by the 6.67 pedestrians yields 213 person-seconds of delay. For the heavy arrival period the delay would be the number of delayed arrivals (12/40 x 13.33) times the average delay $\left[\frac{12 + 0}{2}\right]$ = 6.0 sec. or 24 person-seconds. Pedestrians arriving during the WALK interval would encounter zero delay so that the total delay would be 213 + 24 or 237 person-seconds for the minimum clearance alternative. A similar value would be derived for the minimum WALK alternative

MINIMUM CLEARANCE ALTERNATIVE



MINIMUM WALK ALTERNATIVE



Note: light arrival rate = 0.5. x heavy arrival rate

where W = WALK interval

FDW = flashing DONT WALK interval

SDW = solid DONT WALK interval

Numbers indicate time from the beginning of the cycle.

FIGURE 16. ASSUMED PEDESTRIAN ARRIVAL DISTRIBUTIONS.

The end of the period of heavy arrivals usually seems to come at the end of the WALK interval unless there is a high violation rate of the flashing DONT WALK. Thus, the arrival rate for two adjacent crosswalks would be complementary if volumes are of the same magnitude. It was found that the average arrival rate for the heavy arrival period was approximately twice the arrival rate of the light arrival period. For the example used here with the 80-second cycle, the assumed light and heavy arrival periods would be as shown in Figure 16.

## Calculation of Vehicle Delay

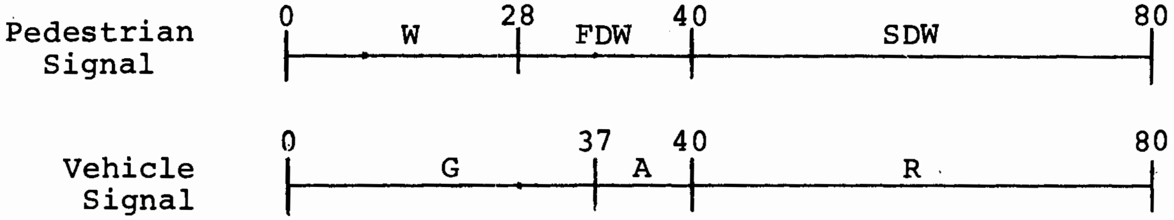
Since the overall delay results were expected to be in favor of extending the WALK interval to the maximum, the assumptions were made conservatively in favor of the opposite alternative with respect to vehicles. Assumptions included:

- all vehicles in right lane turn right
- 3-second average right turn vehicle headways at saturation flow with no pedestrian interference (this value is close to the actual right turn headways measured in this project).
- all vehicles considered in the delay calculation are in the right lane queue when the signal phase begins.

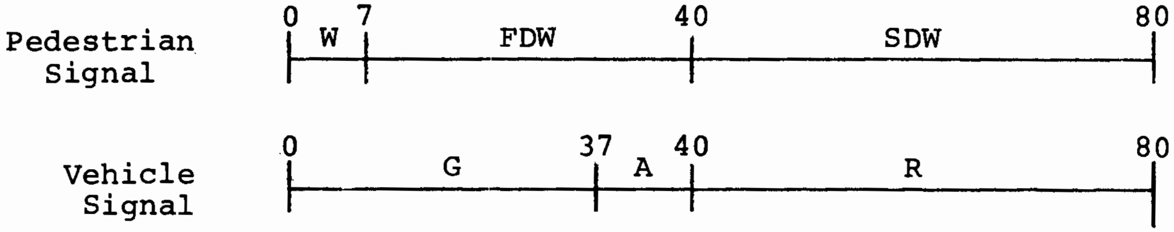
A range of both pedestrian and vehicular volumes were considered. Pedestrian volumes included an average of 2, 5, 10, and 20 persons crossing per cycle. For the 80-second cycle used, the highest value translates into an approximate hourly volume of 900 pedestrians. Vehicle volumes used were 2, 4, 6 and 8 per cycle, translating to hourly volumes between 90 and 260 vehicles per hour.

The delay to vehicles was calculated by estimating from the short crosswalk curves in Figure 14 the delay to each vehicle as it attempted to make its turn. The arrival time and expected delay of each successive vehicle could be determined from the knowledge of the number of previous vehicles and the previous delay. For example, assuming 6 vehicles in a queue, all of which turn right, and 10 pedestrians during the cycle, delay would be computed as follows. The first vehicle arriving at t = 0 would expect to be delayed by 8 seconds (see Figure 14). Assuming 4 seconds to complete the turn, this means that the second vehicle would arrive 8 seconds later than if there were no

MINIMUM CLEARANCE ALTERNATIVE
(excess time allocated to WALK interval)



MINIMUM WALK ALTERNATIVE
(excess time allocated to clearance interval)



where G = Green indication
A = Amber indication
R = Red indication
W = WALK interval
FDW = flashing DONT WALK interval
SDW = solid DONT WALK interval
Numbers indicate time from the beginning of the cycle.

FIGURE 15. VEHICLE AND PEDESTRIAN PHASES TESTED FOR THE ALLOCATION OF EXCESS PEDESTRIAN TIME.

## Timing Alternatives Considered

The vehicular and pedestrian phases for the two alternatives of excess pedestrian time allocation being compared are shown on Figure 15. For the purpose of further reference these are termed the "minimum WALK" and "minimum clearance" alternatives. A street width (crosswalk length) of 52 feet and an 80-second cycle with a 50-50 split were assumed. It is possible to split the excess time between the WALK and clearance intervals in different proportions. However, the resulting pedestrian delay will be somewhat proportional to the percentage allocated to each. It was felt that the best means of analysis would be the examination of the extremes, which are represented by the timing diagrams in Figure 15.

## Pedestrian Arrival Rates

A number of simplifying assumptions were necessary before evaluating the delay impact for each of the timing schemes in Figure 15. One important assumption dealt with the arrival of pedestrians approaching the crosswalk. The studies performed on pedestrian arrivals revealed that the arrival rate is heavier during the period just prior to and during the WALK interval. The arrival rate is relatively light following the beginning of the clearance interval. The reason for this occurence is that pedestrians wishing to cross to the diagonally opposite corner have an option of which crosswalk to use. Wishing to minimize their delay, they will naturally select the direction which is displaying, or will most quickly display, the WALK signal. Thus, the only pedestrians who will arrive at a crosswalk shortly after the WALK interval ends are those who do not wish to eventually arrive at the diagonal curb. It was found in this study that the period of heavy arrivals normally begins shortly after the time that the flashing DONT WALK is displayed on the perpendicular crosswalk. Complicating this, however, is the group of pedestrians that arrive at the crosswalk after having been released from the opposite end of the perpendicular crosswalk. This tends to create a "hump" in the arrival distribution as many pedestrians in that group may also want to use the crosswalk in question to complete a crossing to the diagonal curb. The location of this hump in the distribution of arrivals will be dependent on the length of the perpendicular crossing. For the purpose of this study the hump was absorbed into the rest of the distribution and assumed not to exist.

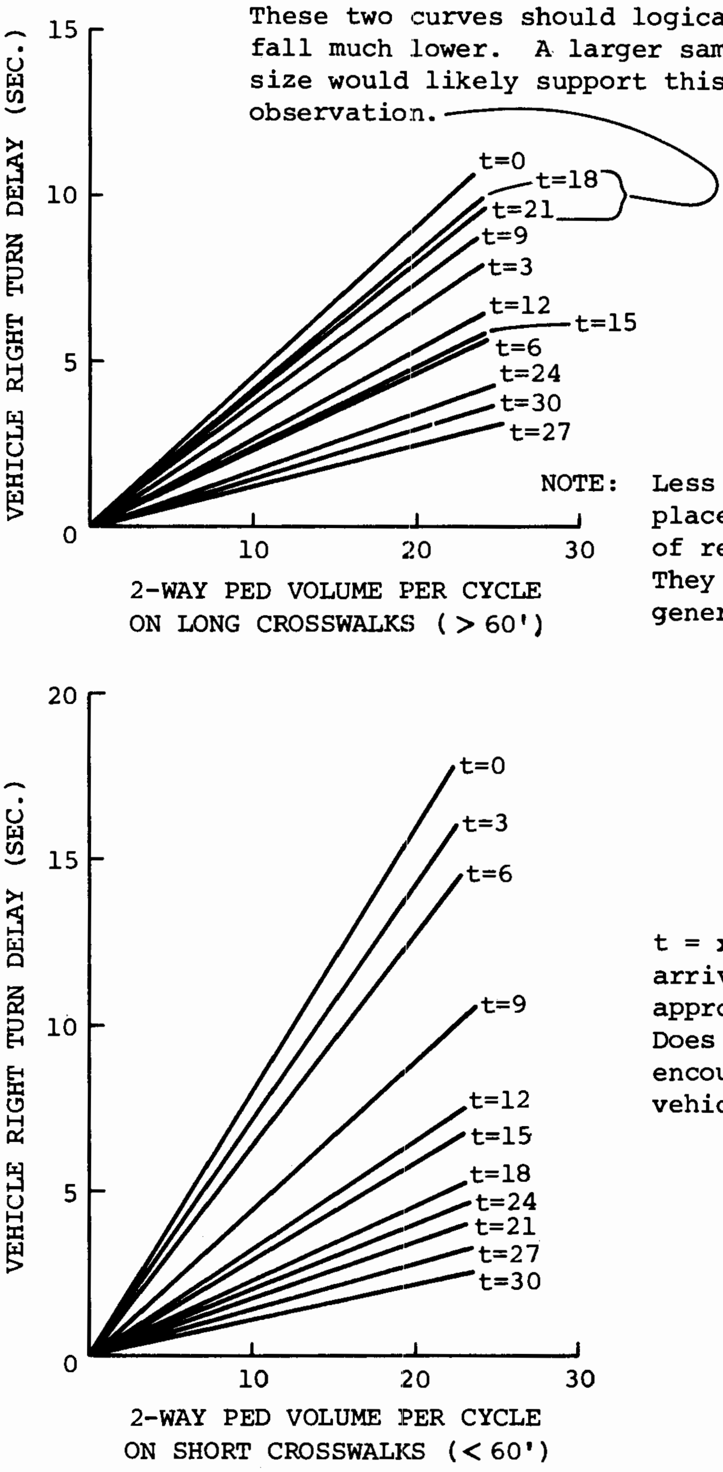
Even though the data for the pedestrian volume ranges observed in this study supported linearity of the delay curves, it is anticipated that at some volume level the slope of the curve begins to decrease. This change probably takes place at a volume level somewhere above 20 pedestrians per cycle. The number of locations and times during which this level is attained is few so that the curves will be applicable to a vast majority of locations. It is also felt that the curves are applicable to signalized intersections without pedestrian signal displays even though the data was taken only at locations with the displays provided. This conclusion is supported by the fact that the distribution of pedestrians walking in a crosswalk with or without pedestrian displays will not differ during the beginning of the phase and will only differ slightly toward the end of the phase. A difference will occur toward the end of the phase because pedestrians on a crosswalk without pedestrian displays tend to begin crossing up until they see the yellow vehicle indication whereas those on a crosswalk with the displays generally cease beginning to cross when the pedestrian clearance interval is shown,usually several seconds before the yellow vehicle indication. However, vehicle right turn delays during this latter part of the phase are usually small, resulting in little difference in delay with or without the pedestrian displays.

As already stated, the curves for long crosswalks are less valid than for the short crosswalks because of fewer data points. The t = 0 curve, however, is based on adequate data and forms a good comparison between long and short crosswalks. Delays for t = 0 are significantly less for long crosswalks than for short. This substantiates the previous hypothesis that vehicles turning into long crosswalks will usually incur less delay since they have a gap between the passing of opposing pedestrian platoons. The remainder of the long crosswalk data offer only general trends in delay differences between long and short crosswalks and should not be used to obtain definite delay values. In general, it appears that overall delay for short crosswalks is greater than for those which are long. The gap between platoons and the fact that vehicles turning into wide streets have more space available in which to avoid pedestrians may explain this phenomenon. Thus, the remaining delay analysis will be built around the short crosswalk data which seems to form the "worst case" with respect to right turn delay.

## TABLE 13. EQUATIONS AND CORRELATION COEFFICIENTS OF VEHICLE RIGHT TURN DELAY CURVES.

| Vehicle Arrival Time Category | Slope | y Intercept | r | Sample Size |
|---|---|---|---|---|
| <u>Long Crosswalks</u> | | | | |
| t = 0 | .304 | 3.135 | .821 | 110 |
| 3 | .237 | 0.706 | .899 | 28 |
| 6 | .168 | -.303 | .725 | 30 |
| 9 | .322 | 1.061 | .866 | 40 |
| 12 | .068 | 2.386 | .267 | 34 |
| 15 | .339 | -.958 | .937 | 22 |
| 18 | .366 | 1.028 | .841 | 17 |
| 21 | .522 | -.851 | .692 | 16 |
| 24 | .181 | -.162 | .708 | 10 |
| 27 | .064 | .581 | .515 | 12 |
| 30 | -.104 | 3.883 | -.422 | 20 |
| <u>Short Crosswalks</u> | | | | |
| t = 0 | .786 | .116 | .978 | 400 |
| 3 | .816 | .817 | .974 | 123 |
| 6 | .601 | .846 | .974 | 141 |
| 9 | .390 | .478 | .894 | 142 |
| 12 | .308 | .093 | .842 | 148 |
| 15 | .325 | -.105 | .825 | 162 |
| 18 | .104 | 1.137 | .979 | 134 |
| 21 | .127 | 0.629 | .782 | 134 |
| 24 | .143 | 0.927 | .640 | 126 |
| 27 | .085 | 0.456 | .855 | 127 |
| 30 | .010 | 0.860 | .712 | 109 |

Note: Less confidence can be placed in long crosswalk data because of reduced sample size (see text).



These two curves should logically fall much lower. A larger sample size would likely support this observation.

NOTE: Less confidence can be placed in this data because of reduced sample size. They are included here for general comparison only.

t = x refers to time of arrival of vehicle at approach crosswalk. Does not include delay encountered by preceding vehicles.

FIGURE 14. RIGHT TURN DELAY CURVES FOR LONG AND SHORT CROSSWALKS.

# TABLE 12. AVERAGE VEHICLE RIGHT TURN DELAY VALUES FOR LONG CROSSWALKS.

| Vehicle Arrival Time Category | Pedestrian Volume Range | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1-5 Peds. | 6-10 | 11-15 | 16-20 | 21-25 | 26-30 | 31-35 | 36-40 |
| t = 0 | 47/4.0 | 23/4.2 | 17/7.39 | 10/13.9 | 2/12.0 | 2/7.0 | 8/11.0 | 1/15.0 |
| t = 3 | 11/0.5 | 8/3.8 | 3/5.0 | -- | 2/7.5 | 2/6.5 | 2/7.0 | -- |
| t = 6 | 13/0.5 | 12/0.6 | 1/2.0 | 1/0.0 | 1/9.0 | 1/3.0 | 1/5.0 | -- |
| t = 9 | 14/2.1 | 15/4.1 | 4/2.5 | 1/2.0 | 1/9.0 | 3/13.6 | 2/9.5 | -- |
| t = 12 | 10/0.5 | 10/4.2 | 9/5.0 | 3/3.6 | -- | -- | 2/1.0 | -- |
| t = 15 | 8/0.4 | 6/1.8 | 3/2.6 | -- | 1/2.0 | 1/12.0 | 3/10.6 | -- |
| t = 18 | 4/1.8 | 4/2.8 | 3/10.0 | 2/7.0 | 2/6.0 | -- | 2/14.0 | -- |
| t = 21 | 2/2.0 | 5/1.6 | 4/9.3 | 3/3.3 | 1/22.0 | -- | 1/14.0 | -- |
| t = 24 | 1/0.0 | 4/2.0 | 2/2.0 | 2/1.0 | -- | 1/7.0 | -- | -- |
| t = 27 | -- | 7/1.0 | 2/2.0 | 1/0.0 | 1/4.0 | -- | 1/2.0 | -- |
| t = 30 | 1/1.0 | 7/4.9 | 7/1.0 | 2/0.5 | -- | 2/3.0 | 1/0.0 | |

KEY: Number of data points/average delay for each pedestrian volume range.

## TABLE 11. AVERAGE VEHICLE RIGHT TURN DELAY VALUES FOR SHORT CROSSWALKS.

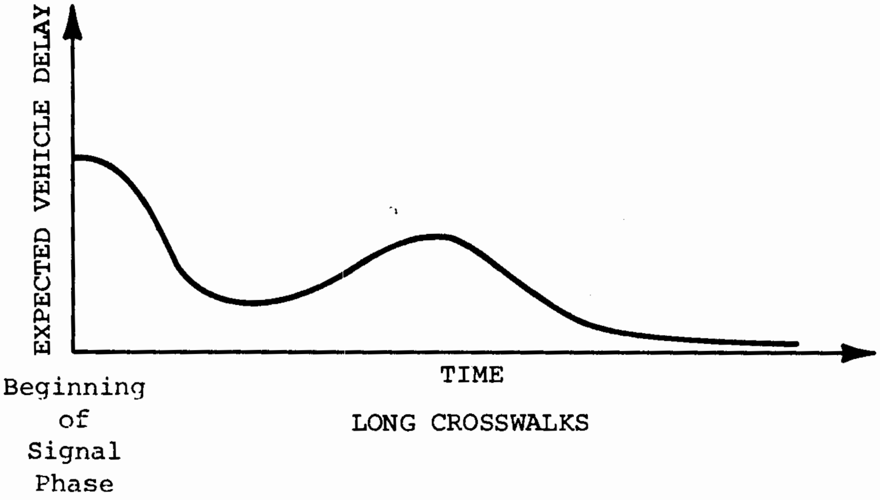
| Vehicle Arrival Time Category | Pedestrian Volume Range | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1-5 | 6-10 | 11-15 | 16-20 | 21-25 | 26-30 | 31-35 | 36-40 | 41-45 |
| t = 0 | 126/2.9 | 121/5.8 | 72/11.2 | 40/16.4 | 33/21.1 | 6/17.5 | 1/34.0 | | |
| t = 3 | 53/1.9 | 42/5.6 | 15/7.8 | 4/10.8 | 5/21.8 | 1/22.0 | 1/28.0 | | |
| t = 6 | 66/2.3 | 42/4.1 | 23/8.2 | 6/13.7 | 2/24.5 | 1/20.0 | 1/18.0 | | |
| t = 9 | 63/1.5 | 45/3.4 | 21/7.8 | 12/5.9 | 1/5.0 | | | | |
| t = 12 | 72/0.8 | 48/3.0 | 17/5.5 | 8/2.4 | 3/17.0 | | | | |
| t = 15 | 60/1.6 | 53/0.9 | 33/5.1 | 12/10.5 | 4/12.5 | | | | |
| t = 18 | 42/1.4 | 54/2.1 | 23/2.7 | 9/3.5 | 6/3.5 | | | | |
| t = 21 | 35/1.1 | 58/1.3 | 24/4.6 | 13/3.8 | 2/1.0 | 2/4.5 | | | |
| t = 24 | 28/1.2 | 56/2.5 | 23/3.7 | 15/2.3 | 4/12.8 | | | | |
| t = 27 | 28/1.0 | 45/1.2 | 26/2.7 | 15/3.3 | 7/4.3 | 4/6.8 | 1/1.0 | | |
| t = 30 | 18/1.7 | 41/1.0 | 29/3.2 | 11/3.7 | 6/3.2 | 2/1.5 | 1/10.0 | | |

**KEY:** Number of data points/average delay for each pedestrian volume volume range.

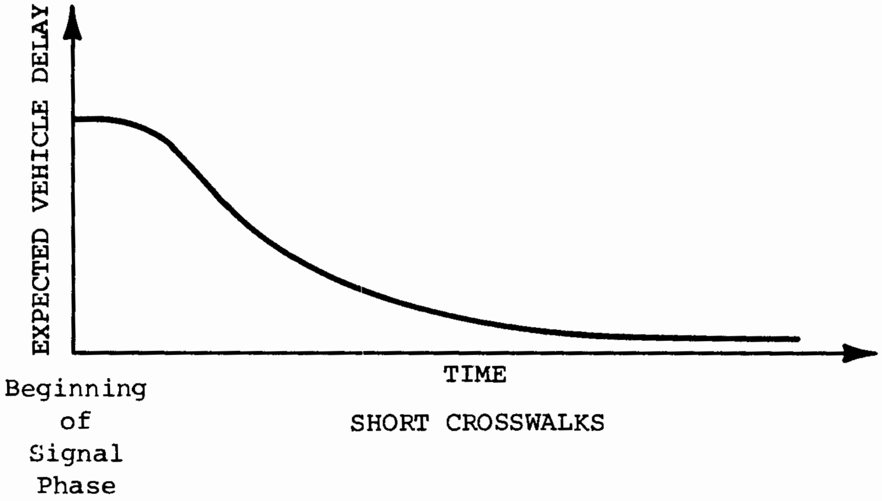
The data for each 3-second time increment from t = 0 to t = 30 and each crosswalk length category were then separately plotted, resulting in 11 pairs of figures. The most data was available for the lower $t_1$ values since vehicles were nearly always present at this time. The scatter in the data was very wide, which is to be expected for this type of behavior. For instance, some vehicles, by anticipating the green, may make their turn before any pedestrians can block their path while other vehicles may wait for the pedestrians to pass. Driver behavior was found to vary widely in this regard. However, this study is not so concerned with how orderly the delays are but what the <u>average</u> value of delay is over a period of time. Because this was the case, delays for ranges of pedestrian volumes were averaged in anticipation that the relationship could be more clearly defined. All delay values were averaged for groups of 5 pedestrians for each crosswalk length category (Tables 11 and 12 ). Adequate data for the short crosswalk category were obtained, but data for long crosswalks was somewhat deficient for all but the lower $t_1$ values. Less confidence can thus be placed on the long crosswalk data. These data were then plotted graphically and curves were fit to those points. The entire series of curves with the averaged data points are shown in Appendix C and a composite display of curves for all time increments is shown in Figure 14.

A high degree of correlation was achieved with a linear curve fit for each set of data on short crosswalks. The relationship between right turn delay and pedestrian volume is especially good for those vehicle arrival time values with the most data. Table 13 shows the equation of each line and its associated correlation coefficient. The lines in Figure 14 have been adjusted slightly based on the most reasonable configuration for the entire family of curves. One requirement for this adjustment was that each line begin at the origin, since at zero pedestrian volume there could be no pedestrian-caused vehicle right turn delay. Consequently, each line retained its general configuration but was rotated to conform with this criterion.

The curves depicting delay on short crosswalks progress in a fairly orderly manner from high delay to low delay as time into the phase increases. From t = 18 to t = 30 there is very little variation in delay. This gives good evidence as to the validity of the curves since t = 15 to t = 18 marks the point where the major platoons of pedestrians will have completed their crossing, after which pedestrians will cross only intermittently.

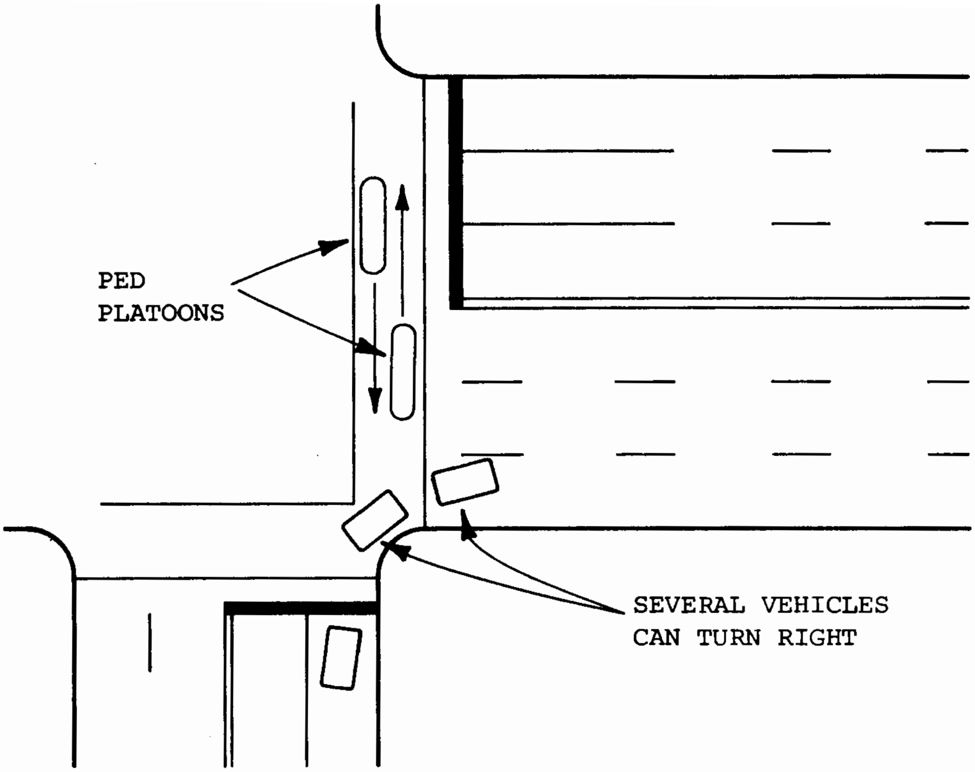


LONG CROSSWALKS

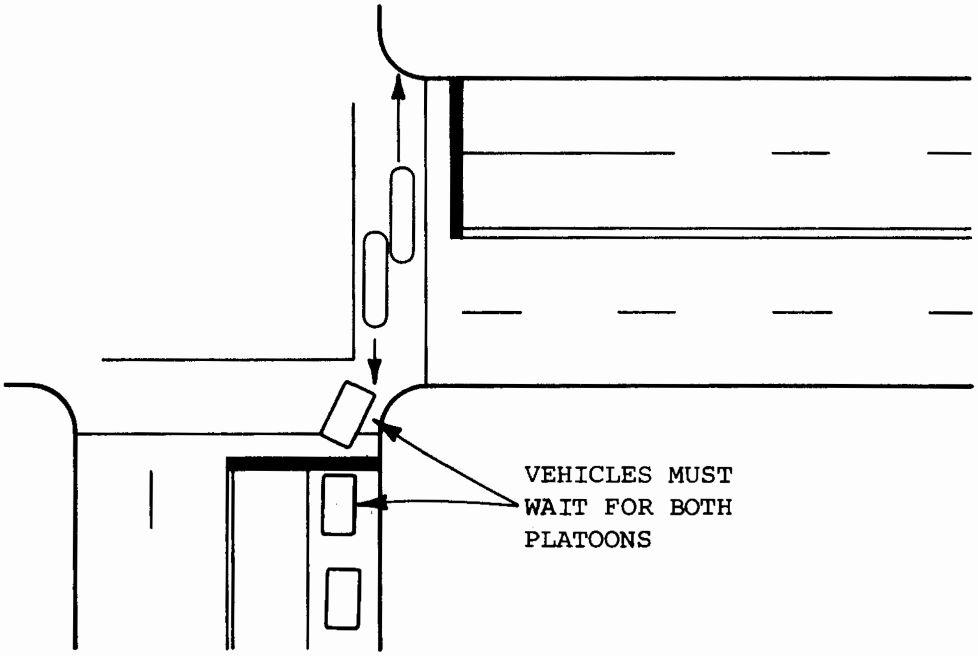


SHORT CROSSWALKS

FIGURE 13. GENERAL DISTRIBUTION OF RIGHT TURN DELAYS FOR VEHICLES CROSSING LONG AND SHORT CROSSWALKS.



LONG CROSSWALKS



SHORT CROSSWALKS

FIGURE 12. GAPS BETWEEN PEDESTRIAN PLATOONS
ON LONG AND SHORT CROSSWALKS.

crossed point $t_2$ at 28 seconds as shown but to have encountered no delay. This means that the 4th vehicle would have had a $t_1$ value of 24 if this were the case. Assuming a time headway for right turning vehicles of 3 seconds (this was found to be a typical value in this research) the 5th vehicle could have arrived no sooner than 3 seconds after this 24-second mark for the 4th vehicle or at $t = 27$. The value of 27 is now determined to be the pseudo $t_1$ value for the 5th vehicle. Since the difference between $t_1$ and $t_2$ is now only 4 seconds, the time required for an undelayed turn, it is evident that the 5th vehicle incurred no delay beyond that which was also encountered by preceding vehicles. In other words, it is likely that no pedestrians passed in front of the 5th vehicle after the 4th vehicle had passed.

The next step was to classify the data points according to $t_1$ values. Values from $t_1 = 0$ to $t_1 = 30$ were grouped together by 3-second increments. Each increment actually included one value on each side of the multiples of 3 seconds. For instance, the value of $t_1 = 6$ included values of $t_1 = 5$ and $t_1 = 7$ while the $t_1 = 9$ value also include $t_1 = 8$ and $t_1 = 10$.

In addition to the above classification, points were also stratified by length of crosswalk. The hypothesis was that vehicle right turn delay would vary depending on the length of time pedestrians were in the street and on the time gap between the passing of opposing pedestrian flows across the vehicle paths of travel. For example, on long crosswalks (wide streets) there will usually be time enough for one or more vehicles to make their turn behind the pedestrians in the near-side platoon and in front of those in the approaching far-side platoon. (See Figure 12). On a narrow street, there is less likely to be a gap between the two platoons since they are much closer to each other at the beginning of green. However, pedestrians on wider streets require a longer crossing time. Vehicles later in the phase may thus encounter less delay for narrow streets since most of the pedestrians have already completed their crossing and are out of the path of vehicles. The distribution of delays might expect to take on a form as shown in Figure 13 for wide and narrow streets. This figure was not based on any numerical analysis but is presented only to illustrate the above point. The stratification of data by crosswalk length was quite difficult because pedestrian volumes also influence the length of the gap between opposing platoons. However, it appeared that under moderate pedestrian volumes a width of approximately 60 feet marked the division line between no vehicles being able to pass between the platoon and at least one vehicle being able to do so.

### Conclusions and Recommendations

The only locations where changes in pedestrian signal timing on a time-of-day basis could be made effectively would be where the signal controller uses a separate off-peak timing plan which does not include one of the three pedestrian peak periods (morning, noon and evening). A volume range of at least 20-30 pedestrians per cycle during midday and perhaps 1-5 pedestrians per cycle at night would also be required. Under these conditions, it would be desirable to use a longer pedestrian clearance during the midday period. A longer or shorter WALK interval might be provided based on the rationale previously described on page 92. Each local traffic engineer would have to evaluate his own set of circumstances for using time-of-day changes.

Based on the above analysis, it appears that changing pedestrian signal timing by time-of-day has little potential for practical application. Signal timing is fairly insensitive to fluctuations in pedestrian flow characteristics since timing would be changed by no more than a few seconds regardless of the variation in pedestrian volume. The confusion which may be caused by attempting such changes may offset any advantages that they may have.

## CORRECTION FACTORS FOR THE HIGHWAY CAPACITY MANUAL

### Study Approach

The "Intersection Capacity" chapter of the *Highway Capacity Manual* (12) is certainly the most widely used and probably one of the most controversial chapters in the *Manual*. Most of the values and factors involved have received much debate as to their accuracy. It is evident from this debate that the development of accurate factors for universal application is extremely difficult, if not impossible.

It was in this context that the issue of capacity correction factors for pedestrians was addressed. The results of this study were not expected to culminate in specific correction factors but were expected to suggest a methodology from which the factors might eventually evolve. One major reason why the development of factors would not be feasible at this time is that the effect of pedestrians on the factors in the *Manual* is uncertain. No specific pedestrian studies were included in the data base used for the *Manual*. Data sheets used for the some 1,600 intersection approaches observed included boxes to be checked for various levels of pedestrian levels labeled "none", "few", "moderate", and "large" numbers of pedestrians. A vast majority of observers judged pedestrian volumes to be in the "few" range. No numerical values of pedestrian volumes were stated.

The factor in which pedestrian activity is primarily taken into account is the "metropolitan area location" factor. This factor was intended to account for all variables that had not been individually accounted for by one of the other factors. Pedestrians were viewed to be one small part of the metropolitan area location factor. Vehicular side friction (parking maneuvers, etc.) and general differences in driver behavior were also viewed to be responsible for the differences among the CBD, fringe area, outlying business district, and residential area factors.

Since no quantitative values of the pedestrian influence are contained in the Capacity Manual, it would be very difficult to extract its effect on capacity. This would have to be done before any clearly defined pedestrian factor could be separately introduced. It was felt that a pedestrian factor would best be derived along with a restructuring of the entire intersection chapter of the Manual, should that occur.

The remainder of this section describes a methodology which could be applied to the development of such a pedestrian correction factor. The methodology is limited to the analysis of the right lane only and does not address the entire approach. It must again be emphasized that this analysis is not intended to suggest the factors to be used. A much more comprehensive analysis will be required to develop factors which will adequately fit into the Capacity Manual. It is expected that the methodology described herein will also be applicable to analyses other than the Highway Capacity Manual. The "critical lane" method may hold particular promise for its application.

## Data Collection

No additional data were collected beyond that used for the previous studies involving vehicular delay. The delay curves derived previously (Figure 14) have been used to form the base for the analysis performed here. Specifically, the delay curves for short crosswalks (less than 60 feet) have been used to develop the capacity factors.

## Data Analysis

The data analysis section consists of the proposed methodology and a numerical example of its application. The casual factors considered in this methodology included vehicle volume in the right lane, percent right turns in that lane, two-way pedestrian volume per cycle and width of the exit leg of the street. For the purpose of this analysis, a cycle length of 80 seconds and a 50-50 split were assumed. It is also important to note that the methodology was initially designed to develop factors for level of service E (capacity) only. Factors for the other levels of service can be derived using adjustment factors based on data in the Capacity Manual.

First in the methodology, consider that for each distinct number of vehicles per cycle in the right lane on one approach and for each percentage of right turn vehicles in that lane, there is a certain probability that there will be 0, 1, 2... or n right turning vehicles. For each of these possibilities, there are a number of possible arrival patterns. To illustrate, for a volume of four vehicles per cycle there are $2^4$ or 16 possible combinations of right turn and through vehicles. These have the probabilities of occurrence shown in Table 23 for right lane right turn percentages of 0, 25, 50, 75, and 100. Five vehicles per cycle would have $2^5$ combinations and so on. These probabilities assume an infinite population of vehicles.

To derive capacity, one would ideally compute the number of vehicles that could be served for each arrival pattern. This can be done using the short crosswalk data in Figure 14 and a procedure similar to that previously shown in Table 14. For example, where the first four vehicles turn right, it is possible to serve 5 vehicles for the timing assumed above and 20 pedestrians per cycle. If there are no right turning vehicles toward the beginning of a phase, it will be possible to serve perhaps 10 or 12 vehicles. Once the service rate for all possible arrival patterns has been computed, each would be assigned a probability of occurrence from tables similar to Table 22. Because it is not likely that these probabilities will sum to one, they must be normalized so that this is the case. The overall capacity would then be the sum of the products of each service rate and normalized probability.

Because the above procedure is much too complex to be carried out here, a simpler procedure was devised based on the assumption that no vehicle delay occurs after the fourth vehicle. Although this is not always true, it gives

## TABLE 22. PROBABILITIES OF OCCURRENCE OF RIGHT TURN VEHICLES FOR VARIOUS TURN PERCENTAGES AND 4 VEHICLES PER CYCLE

| Sequence of Right Turning Vehicles | Percent Right Turns | | | | |
|---|---|---|---|---|---|
| | 0 | 25 | 50 | 75 | 100 |
| 0 | 1.0 | .317 | .0625 | .005 | 0.0 |
| 1 | 0.0 | .105 | .0625 | .012 | 0.0 |
| 2 | 0.0 | .105 | .0625 | .012 | 0.0 |
| 3 | 0.0 | .105 | .0625 | .012 | 0.0 |
| 4 | 0.0 | .105 | .0625 | .012 | 0.0 |
| 1,2 | 0.0 | .035 | .0625 | .035 | 0.0 |
| 1,3 | 0.0 | .035 | .0625 | .035 | 0.0 |
| 1,4 | 0.0 | .035 | .0625 | .035 | 0.0 |
| 2,3 | 0.0 | .035 | .0625 | .035 | 0.0 |
| 2,4 | 0.0 | .035 | .0625 | .035 | 0.0 |
| 3,4 | 0.0 | .035 | .0625 | .035 | 0.0 |
| 1,2,3 | 0.0 | .012 | .0625 | .105 | 0.0 |
| 1,2,4 | 0.0 | .012 | .0625 | .105 | 0.0 |
| 1,3,4 | 0.0 | .012 | .0625 | .105 | 0.0 |
| 2,3,4 | 0.0 | .012 | .0625 | .105 | 0.0 |
| 1,2,3,4 | 0.0 | .005 | .0625 | .317 | 1.00 |

a reasonable estimate of what the capacity for various turning percentages and pedestrian volumes might be.

To do this, a table was constructed of delay times for each of the arrival patterns possible under four vehicles per cycle. Delay time is defined as the time differential between when the last vehicle in the queue would pass through the intersection with no pedestrians present during the cycle and when that same vehicle would pass through the intersection after the queue has encountered pedestrian delay. This yields an estimate of the "lost capacity" of the right turn lane for each of the combinations of through and right turn vehicles. Table 23, derived from the delay curves, shows the delay time for per cycle pedestrian volumes of 5, 10, 15, and 20 and vehicle volumes of four per cycle. It was assumed that none of the through vehicles had sufficient room to by-pass the delayed right turning vehicle. If by-pass room did exist, the delay time would be reduced.

Each of the delay times were then multiplied by the corresponding probability of that arrival pattern. An example of this procedure for 20 pedestrians per cycle is given in Table 24. The result is an expected delay time for each pedestrian volume and turning percentage (Table 25).

The next step is to subtract the expected delay time from the available green time and divide by the average time headway for each turning vehicle to estimate the expected capacity per cycle for each right turn percentage and pedestrian volume. Average headways were assumed to be 2.25 seconds for through vehicles and 3.0 seconds for turning vehicles. This must then be converted into vehicles per hour of green. For 20 pedestrians per cycle and 25 percent right turns the following would be done:

$$
\begin{array}{rl}
37 & \text{sec. available time} \\
- \underline{10.4} & \text{sec. delay time} \\
26.6 & \text{sec. actual movement time}
\end{array}
$$

$$\frac{26.6 \text{ sec.}}{(2.44 \text{ sec. avg. headway for } 25\% \text{ turns})} = \text{10.9 vehicles per cycle expected capacity}$$

10.0 veh./cycle x 3600 sec./hr. - 37 sec. green/cycle
= 1069 veh./hr. green

TABLE 23. DELAY TIME PER CYCLE FOR 16 COMBINATIONS OF VEHICLE SEQUENCE

| Sequence of Right Turn Vehicles | Delay time (in seconds) for various pedestrian volumes per cycle. | | | |
|---|---|---|---|---|
| | 5 | 10 | 15 | 20 |
| 0 | 0 | 0 | 0 | 0 |
| 1 | 4 | 8 | 12 | 16 |
| 2 | 4 | 7 | 11 | 14 |
| 3 | 3 | 6 | 9 | 13 |
| 4 | 2 | 4 | 7 | 9 |
| 1,2 | 6 | 11 | 15 | 19 |
| 1,3 | 6 | 11 | 15 | 19 |
| 1,4 | 5 | 10 | 15 | 19 |
| 2,3 | 6 | 10 | 14 | 18 |
| 2,4 | 6 | 10 | 14 | 17 |
| 3,4 | 5 | 9 | 12 | 17 |
| 1,2,3 | 8 | 13 | 18 | 22 |
| 1,2,4 | 7 | 13 | 17 | 21 |
| 1,3,4 | 7 | 13 | 17 | 21 |
| 2,3,4 | 7 | 12 | 17 | 21 |
| 1,2,3,4 | 9 | 15 | 20 | 24 |

TABLE 24. EXPECTED DELAY TIME FOR 20 PEDESTRIANS PER CYCLE AND 25 PERCENT RIGHT TURNS.

| Sequence of Right Turn Vehicles | Delay Time (x) | Probability of Occurrence (p) | (x) (p) |
|---|---|---|---|
| 0 | 0 | .317 | 0.0 |
| 1 | 16 | .105 | 1.68 |
| 2 | 14 | .105 | 1.47 |
| 3 | 13 | .105 | 1.37 |
| 4 | 9 | .105 | 0.95 |
| 1,2 | 19 | .035 | 0.67 |
| 1,3 | 19 | .035 | 0.67 |
| 1,4 | 19 | .035 | 0.67 |
| 2,3 | 18 | .035 | 0.63 |
| 2,4 | 17 | .035 | 0.60 |
| 3,4 | 17 | .035 | 0.60 |
| 1,2,3 | 22 | .012 | 0.26 |
| 1,2,4 | 21 | .012 | 0.25 |
| 1,3,4 | 21 | .012 | 0.25 |
| 2,3,4 | 21 | .012 | 0.25 |
| 1,2,3,4 | 24 | .005 | 0.12 |

$\sum = 10.44$

## TABLE 25. EXPECTED DELAY TIME PER CYCLE

| Percent Right Turns | Delay time (in seconds) for various pedestrian volumes per cycle. | | | | |
|---|---|---|---|---|---|
| | 0 Peds | 5 | 10 | 15 | 20 |
| 0 | 0 | 0 | 0 | 0 | 0 |
| 25 | 0 | 2.9 | 5.5 | 8.0 | 10.4 |
| 50 | 0 | 5.3 | 9.5 | 13.3 | 16.9 |
| 75 | 0 | 7.2 | 12.5 | 17.0 | 21.0 |
| 100 | 0 | 9.0 | 15.0 | 20.0 | 24.0 |

Table 26 gives the capacities in vehicles per hour of green from the estimated values of delay time in Table 25. The per cycle pedestrian volumes can be translated into hourly volumes, if desired, by multiplying by the number of cycles per hour.

Plotting the capacity curves of equal pedestrian volume graphically displays the pattern of decreasing capacity with increasing pedestrian volume and right turn percentages (Figure 26 ). Although these were obtained using the assumption of no delay after the fourth vehicle, they should closely approximate the true values where the exit leg is less than 60 feet wide (a constraint of the delay curves used in Figure 14). Multiplying the volumes at level of service E shown in Figure 26 by .83 will yield an estimate of volume at service level D. Multiplying by .67 will estimate levels A, B, and C. These factors were derived from the table on page 139 of the Capacity Manual.

The inclusion of the data into the overall capacity analysis could be approached from two directions, both of which require significant presumptions. First, the right lane may be analyzed entirely separate from the through lanes as is presently done for special turning lanes in the Capacity Manual. This requires the knowledge of through and right turn movements in the right lane, data which is usually unavailable. It will usually be known when there is an exclusive right turn lane, however, in which case these curves can be directly applied.

The other method, more difficult but more directly applicable to the Capacity Manual, is the inclusion of the right lane volumes and turn percentages into the total flow of traffic. This holds the distinct advantage of using data which is normally available. Factors similar to the right and left turn factors presently used could then be derived using 10 percent right turns as the base condition. However, the method by which the curves are expanded from only the right lane to all lanes will necessitate much additional research.

Another data set which would be useful to the traffic engineer consists of the capacity of exclusive right turn lanes for a given phase length and pedestrian volume (Table 28). It has been assumed in this table that at least 2 vehicles can turn even under the most restricted conditions. From this table, it can be determined what the timing requirements for an exclusive right turn lane might be or if queues will develop under various levels of

TABLE 26. RIGHT LANE CAPACITY IN VEHICLES PER HOUR GREEN

| Percent Right Turns In Right Lane | Pedestrian Volume Per Cycle | | | | |
|---|---|---|---|---|---|
| | 0 | 5 | 10 | 15 | 20 |
| 0 | 1600 | 1600 | 1600 | 1600 | 1600 |
| 25 | 1480 | 1360 | 1260 | 1160 | 1060 |
| 50 | 1370 | 1170 | 1020 | 880 | 740 |
| 75 | 1280 | 1030 | 850 | 690 | 550 |
| 100 | 1200 | 930 | 710 | 550 | 420 |

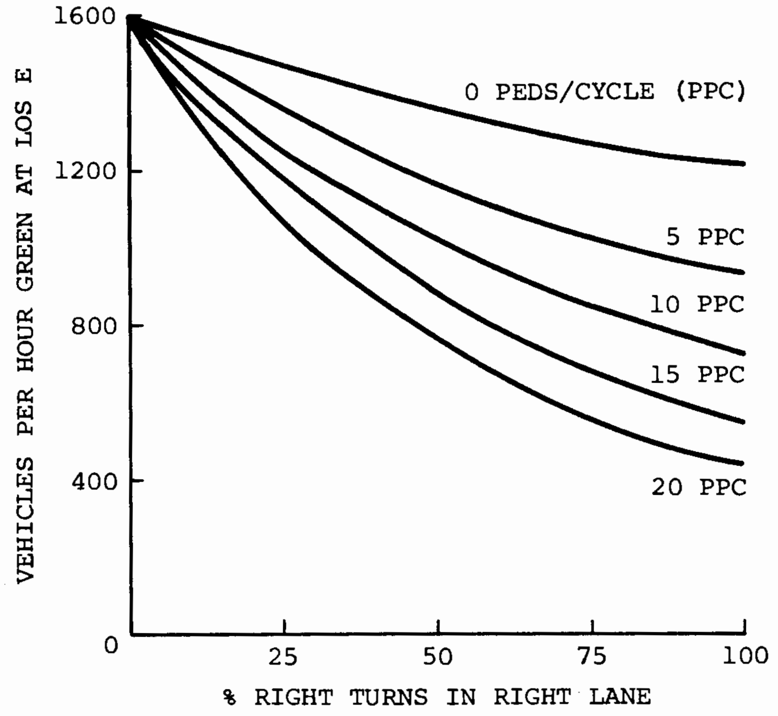


FIGURE 26. CAPACITY OF RIGHT LANE FOR VARIOUS TURN PERCENTAGES AND PEDESTRIAN VOLUMES.

TABLE 27. CAPACITY PER CYCLE OF AN EXCLUSIVE RIGHT TURN LANE.

| Green Phase Time (sec) | Pedestrian Volume Per Cycle | | | | |
|---|---|---|---|---|---|
| | 0 | 2 | 5 | 10 | 20 |
| 20 | 6 | 5 | 4 | 3 | 2 |
| 25 | 8 | 6 | 5 | 4 | 2 |
| 30 | 10 | 7 | 6 | 5 | 3 |
| 35 | 11 | 9 | 8 | 6 | 4 |
| 40 | 13 | 12 | 10 | 7 | 5 |
| 45 | 15 | 13 | 11 | 9 | 6 |
| 50 | 16 | 15 | 13 | 11 | 8 |
| 55 | 18 | 17 | 15 | 12 | 10 |
| 60 | 20 | 18 | 16 | 14 | 11 |

Note: This table is applicable only where the exit crosswalk is less than 60 feet long.

vehicle demand. This table has already been referenced in the procedures for determining capacity of late release and scramble timing.

## Conclusions and Recommendations

Clearly, a method for developing pedestrian factors for the Highway Capacity Manual is achievable. Given the proper data, it is likely that factors equally or more accurate than most of those presently in the Manual can be obtained. However, additional data and more rigorous analysis methods (i.e. computer) will be required to effectively accomplish this task. Such an effort is beyond the economic and time constraints of this study and would best take place along with the complete restructuring of the Manual if that should occur.

The primary problem with the use of pedestrian factors, given that they can be developed, lies in the data which will be required by the traffic engineer for their application. These data must include a two-way pedestrian count, length of the crosswalk, cycle length or an estimate of cycle length, and any unusual conditions or compliance problems encountered.

## GENERAL OBSERVATIONS ON PEDESTRIAN FLOW CHARACTERISTICS

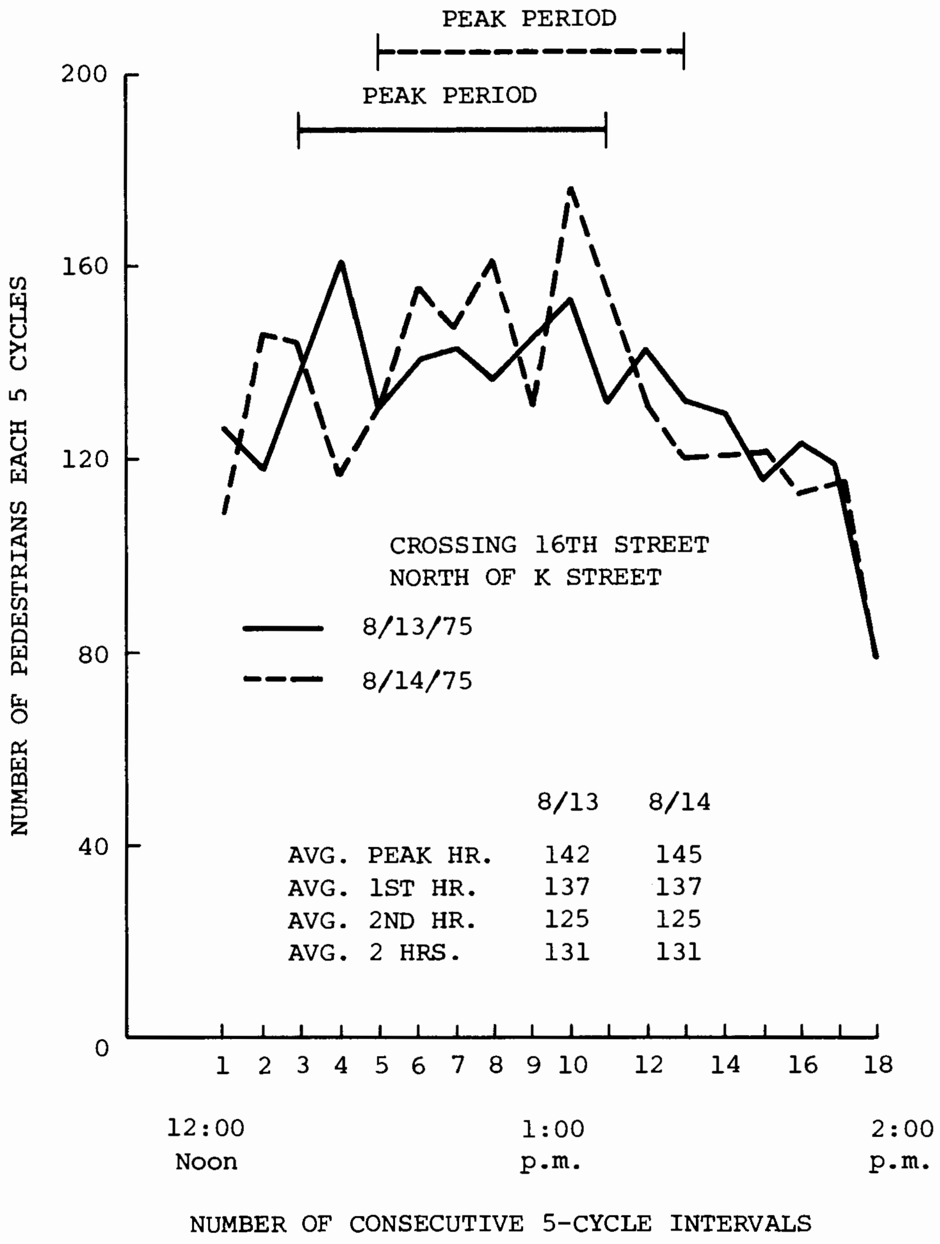
During the course of this task of the pedestrian safety research, a significant amount of volume data were gathered from various sources. An analysis of these data has resulted in several noteworthy observations.

First, it was observed from data in Washington, D.C. that pedestrian peak hours can vary widely from one location to another. While peak hours in the CBD seem to occur during the a.m. and p.m. peak traffic hours and particularly during the lunch hour, peak hours for locations in other parts of the city vary throughout the day. At most locations outside the CBD, volumes are lower and are more evenly distributed throughout the day. The peak hour characteristics at all locations are highly dependent on the types of development in the immediate vicinity of the intersection. Peak hours must usually be selected by actual on-site investigation.

Another interesting observation involved the difference in pedestrian volumes on different crosswalks at the same intersections. Pedestrian count data obtained from the District of Columbia showed that volumes on one or two crosswalks can be far higher than volumes on the other crosswalks. It was common in this data for volumes on certain crosswalks at one intersection to be higher than others by a factor of 4 or more over the period of a day. This indicates that if one desires to know the highest volumes at an intersection, he must be selective not only in the peak hour, but also in the crosswalk or crosswalks observed. These may be important considerations where no pedestrian volume data exist and where extensive counting cannot be done. It is also important when decisions are made to prohibit pedestrians from using one or more crossings at critical intersections.

The solution to the lack of data and personnel to collect it rests with short pedestrian counts. It was found in an analysis of the Washington, D.C. data that a 12 minute sample (20 percent on an hourly volume approximation) would provide an accuracy of 10 percent with less than 70 percent confidence for the fairly high volume levels observed. A more accurate representation of average hourly volume when a short count is used is obtained when samples are taken periodically throughout the hour, for instance on alternate cycles, every third cycle, etc. Sampling on alternate cycles was found to bring accuracy into the range of 5 percent of the hourly count with a high level of confidence. Such samples tend to dampen the effect of short peaks within the hour such as those which caused such a large error with the 20 percent continuous sample. These short peaks were found to exist quite frequently during the heavy volume pedestrian periods observed. A sample of short term peaking characteristics, classified by 5 80-second cycle increments from the Washington, D.C. data is shown in Figure 27. An example of 24 hour pedestrian volume characteristics has been previously documented in reference 9. Hourly volumes over shorter periods are typically available from those cities which perform pedestrian counts.

Practically speaking, counting personnel required for a typical 4-legged intersection could be halved if alternate cycle sampling is used. The observer would merely rotate systematically around the intersection, counting one crosswalk each phase. If a short vehicle count is also acceptable, arrangements could be made to coordinate the vehicle and pedestrian counting efforts so as to make the most efficient use of field crew time. If pedestrian volumes are light, the observer may be able to count both vehicles and pedestrians simultaneously in this manner, requiring only one person for the entire count.



| | 8/13 | 8/14 |
|---|---|---|
| AVG. PEAK HR. | 142 | 145 |
| AVG. 1ST HR. | 137 | 137 |
| AVG. 2ND HR. | 125 | 125 |
| AVG. 2 HRS. | 131 | 131 |

FIGURE 27. SHORT TERM PEDESTRIAN PEAKING CHARACTERISTICS FOR 5-CYCLE - (400 SEC.) INTERVALS.

It was also discovered that sampling on one day with no adverse weather conditions should give adequate results. At the location observed for two consecutive days (16th and K Streets in Washington, D.C.), the 5-cycle counts were remarkably close, being within less than 1 percent each hour. In view of the above results, a short pedestrian count should be sufficient for obtaining the volume data required for use of timing procedures described herein as well as for the examination of the pedestrian warrant for signal installations as described in the MUTCD. In the case of the latter, a short count would be required for each hour over an 8-hour period. In this regard, it would certainly be desirable to develop an inexpensive pedestrian counter.

# REFERENCES

1. Manual on Uniform Traffic Control Devices, U.S. Department of Transportation, 1971.

2. Robertson, H.D., W.G. Berger and R.F. Pain, Urban Intersection Improvements for Pedestrian Safety - VOL. II - Identification of Safety and Operational Problems at Intersections, prepared for Federal Highway Administration, May 1975.

3. Hulscher, F. R., Traffic Signal Facilities for Blind Pedestrians, Department of Motor Transport, N.S.W., August, 1976.

4. Pushkarev, B.S. and J. M. Zupan, Urban Space for Pedestrians, MIT Press, Cambridge, Massachusetts, 1975.

5. Hoel, Lester A., "Pedestrian Travel Rates in Central Business Districts", Traffic Engineering, January, 1968, pp. 10-13.

6. Prokopy, J.C., A Manual for Planning Pedestrian Facilities, Federal Highway Administration, June, 1974.

7. Butler, E.C., "Denver's New Traffic Control System Includes Pedestrian Separation Period", Traffic Digest and Review, April, 1953.

8. Dier, R. D., "Pedestrian 'Scramble' Control", Traffic Engineering, January, 1959, pp. 21-22, 35.

9. Dunivar, Robert E., Study of Pedestrian Usage of Roadway Mediums at Signalized Crosswalks, Department of Public Works, Metropolitan Dade County, Florida, 1964.

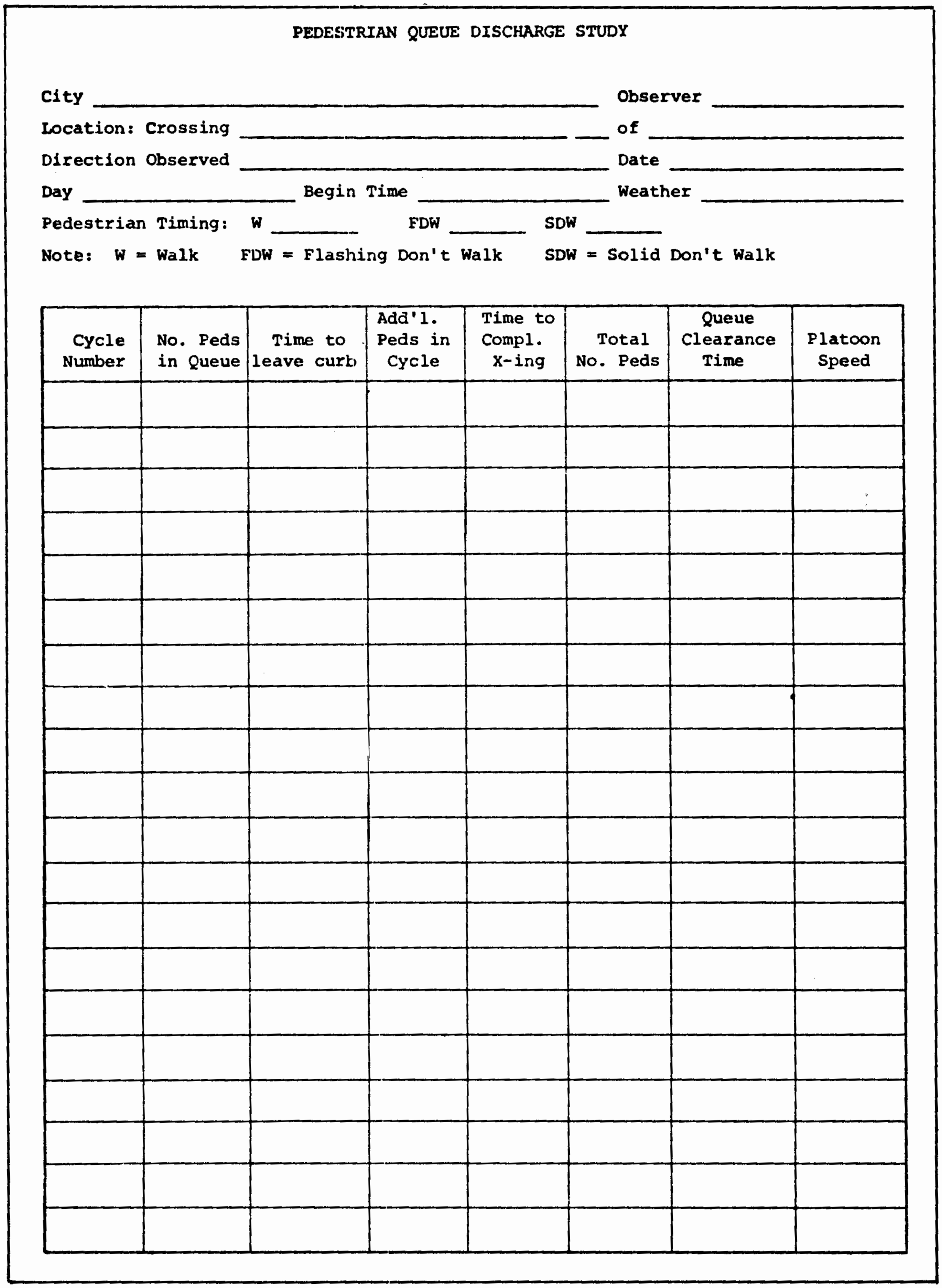
10. Cameron, R. M., "Mechanical Measurement of Pedestrian Volumes", Transportation Research Record 498, Transportation Research Board, 1974, pp. 13-19.

11. Department of Highways and Traffic, Washington, D.C.

12. Highway Capacity Manual 1965, HRB Special Report 87, Washington, D. C., Highway Research Board, 1965.

# APPENDIX A

## FIELD STUDY FORMS



**PEDESTRIAN QUEUE DISCHARGE STUDY**

City ______________________________ Observer ____________

Location: Crossing ____________________ __ of ________________

Direction Observed ______________________ Date ______________

Day _____________ Begin Time ____________ Weather ____________

Pedestrian Timing: W _____ FDW _____ SDW _____

Note: W = Walk FDW = Flashing Don't Walk SDW = Solid Don't Walk

| Cycle Number | No. Peds in Queue | Time to leave curb | Add'l. Peds in Cycle | Time to Compl. X-ing | Total No. Peds | Queue Clearance Time | Platoon Speed |
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FIGURE 28. FORM USED FOR PEDESTRIAN QUEUE DISCHARGE AND PLATOON SPEED STUDIES.

# FIELD STUDY FORM FOR WALK INTERVAL EXTENSION

Location: ____________________

Direction Observed: ____________ Observer ____________

Date ____________ Time ____________ Weather ____________

| Cycle Number | Discharge Time | Cycle Number | Discharge Time | Cycle Number | Discharge Time |
|---|---|---|---|---|---|
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Note: Observe largest queue each cycle in either direction for peak hour. Materials: stopwatch, forms, clipboard, pencil.

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Discharge Time: | 1-7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | Total |
| No. Observations: | | | | | | | | | | |

95% x Total =

Set timing for value which will accommodate at least 95% of total queue.

FIGURE 29. FIELD STUDY FORM FOR DETERMINING LENGTH OF WALK INTERVAL EXTENSIONS.

## ALLOCATION OF EXCESS PEDESTRIAN TIME

(Compliance with Pedestrian Signals)

City ______________________________ Observer ____________________

Location: Crossing ______________________ of ________________________

Direction Observed ______________________ Date ________________________

Day _____________ Begin Time ______________ Weather _________________

Timing Alternative: W ________ FDW ________ SDW ________

Note: W = Walk FDW = Flashing Don't Walk SDW = Solid Don't Walk

| | PEDESTRIAN ACTIVITY TIME INTERVAL | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Begin on WALK | | Begin on FDW | | Begin on SDW | | COMMENT | |
| Cycle Number | End on W or FDW | End On SDW | End On W or FDW | End On SDW | End On SDW | End on W or FDW | | Total No. of Peds |
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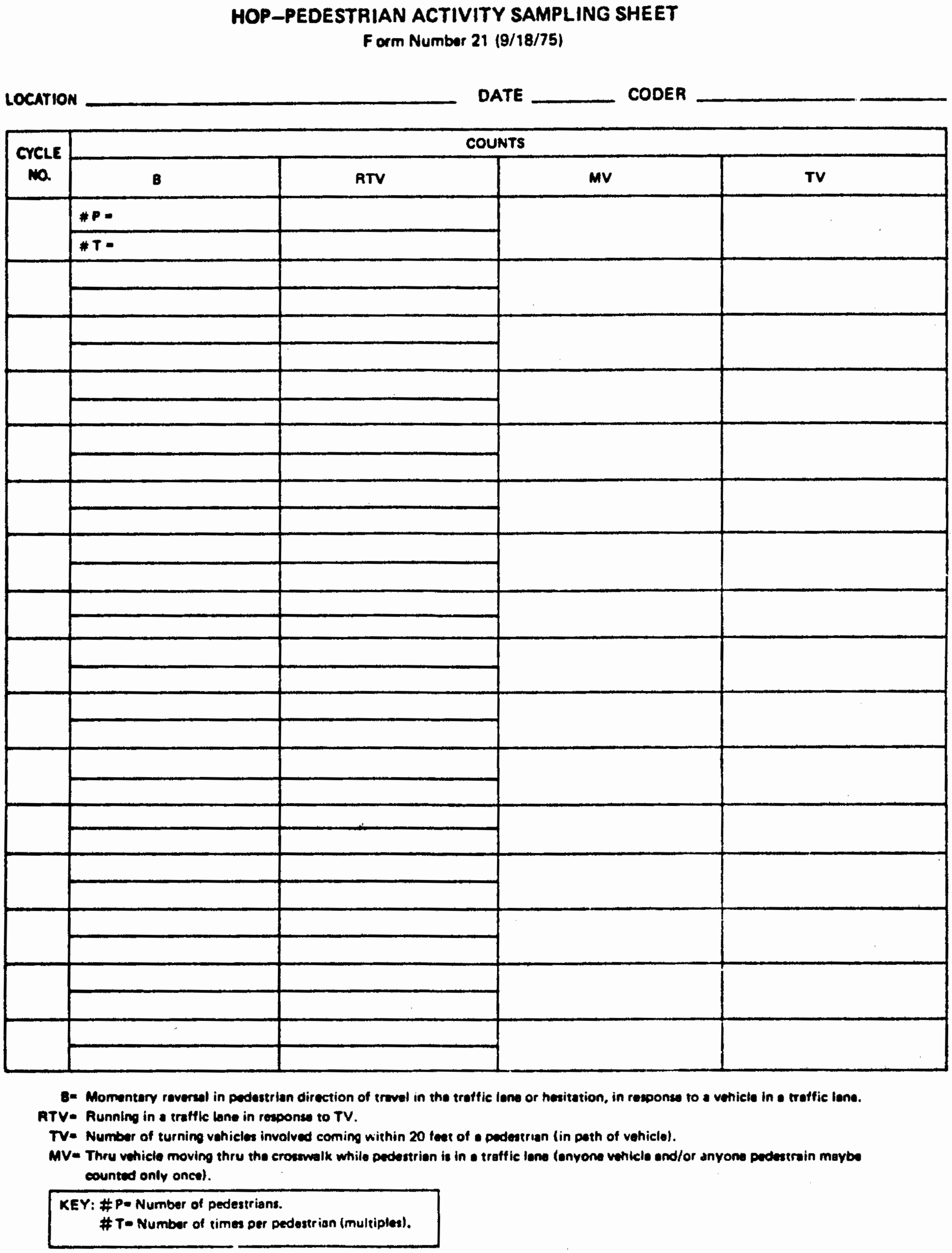
| TOTALS | | | | | | | | |

FIGURE 30. FIELD STUDY FORM USED FOR ALLOCATION OF EXCESS PEDESTRIAN TIME.



**HOP–PEDESTRIAN ACTIVITY SAMPLING SHEET**
Form Number 21 (9/18/75)

LOCATION ____________________ DATE ________ CODER ____________________

| CYCLE NO. | COUNTS | | | |
|---|---|---|---|---|
| | B | RTV | MV | TV |
| | #P = | | | |
| | #T = | | | |
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B= Momentary reversal in pedestrian direction of travel in the traffic lane or hesitation, in response to a vehicle in a traffic lane.
RTV= Running in a traffic lane in response to TV.
TV= Number of turning vehicles involved coming within 20 feet of a pedestrian (in path of vehicle).
MV= Thru vehicle moving thru the crosswalk while pedestrian is in a traffic lane (anyone vehicle and/or anyone pedestrain maybe counted only once).

KEY: #P= Number of pedestrians.
#T= Number of times per pedestrian (multiples).

FIGURE 31. FIELD STUDY FORM USED FOR PEDESTRIAN BEHAVIORS.

## LATE RELEASE FIELD STUDY
## (Compliance with Pedestrian Signals)

LOCATION______________________________

SIDE OBSERVED____________________ DATE__________

DAY________________ BEGIN TIME________________

CYCLE LENGTH________________

| CYCLE | PEDESTRIAN ACTIVITY | | | | | | Total Number of Pedestrians |
|---|---|---|---|---|---|---|---|
| | 1. Begin on "WALK" | 2. Begin on Clearance | | 3. Begin on Solid Don't Walk | | | |
| | | | | Vehicle Signal Red | | veh. turn arrow | |
| | End on Walk or Clearance | 2a. End on Clearance | 2b. End on SDW | 3a. End on SDW | 3b. End on W or FDW | 3c. End cn W or FDW | |
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FIGURE 32. FORM USED FOR LATE RELEASE COMPLIANCE STUDY.

# APPENDIX B

## CUMULATIVE DISTRIBUTIONS OF PEDESTRIAN PLATOON SPEEDS

CROSSING M STREET AND RHODE ISLAND AVENUE EAST OF CONNECTICUT AVENUE

AUGUST 12, 1975 12:00-2:00 P.M.

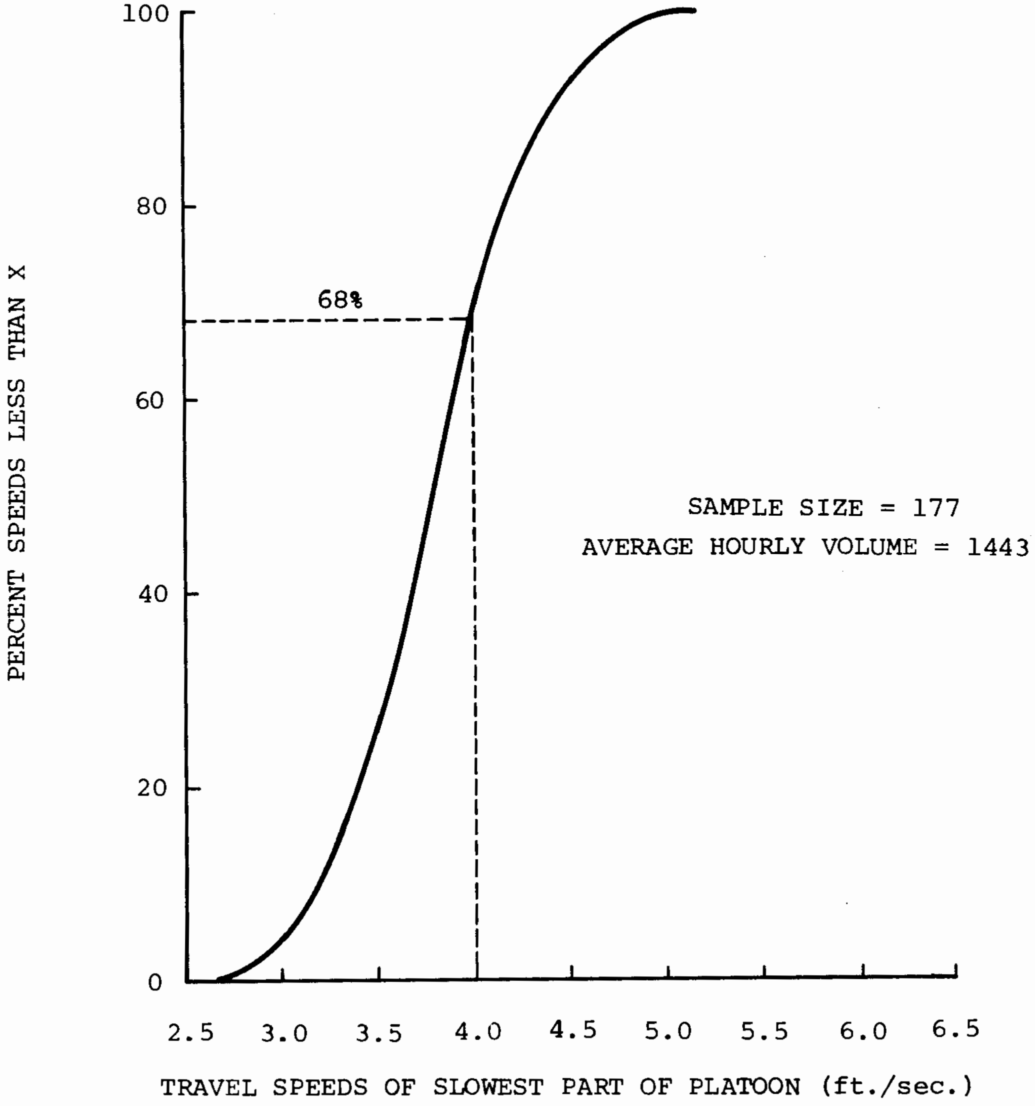


FIGURE 33. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS, WASHINGTON, D.C., M STREET AND CONNECTICUT AVENUE.

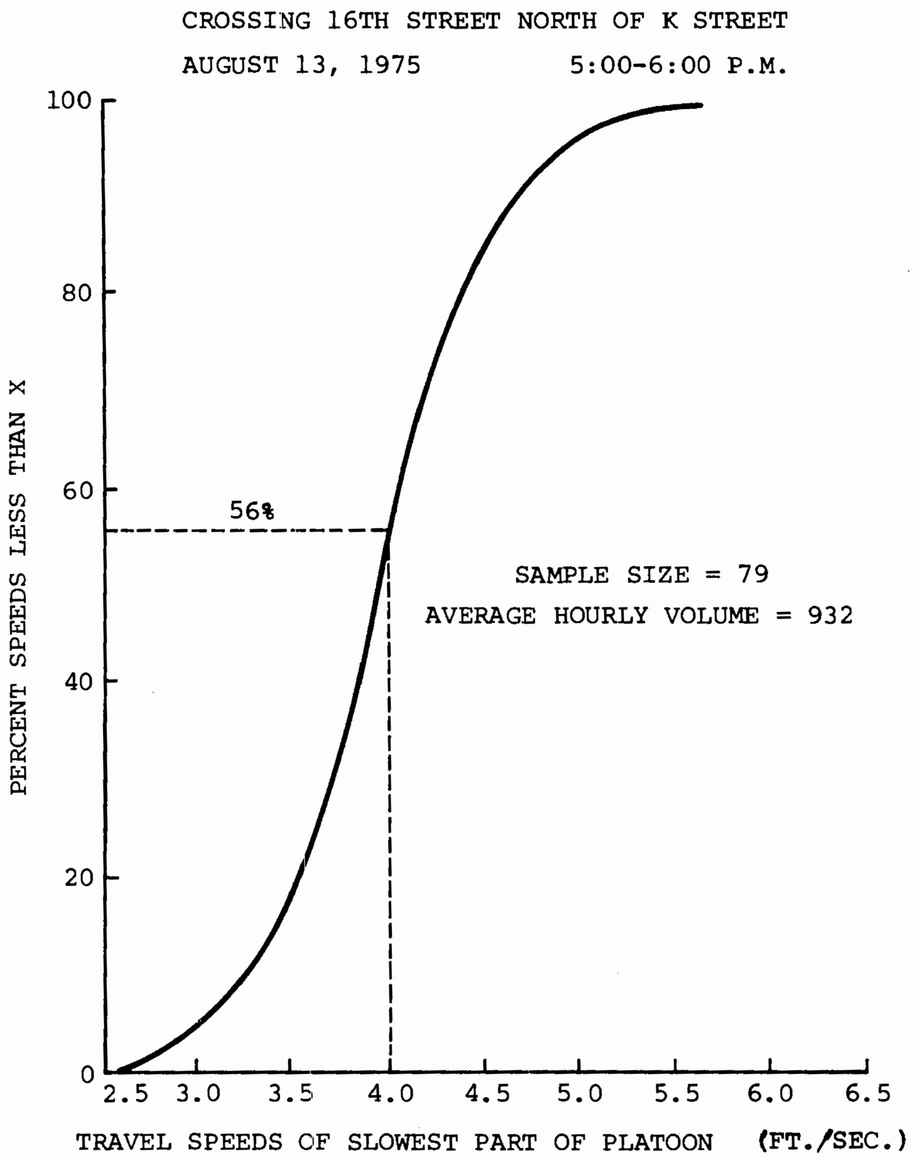


FIGURE 34. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS, WASHINGTON, D.C., 16TH AND K STREETS, EVENING PEAK.

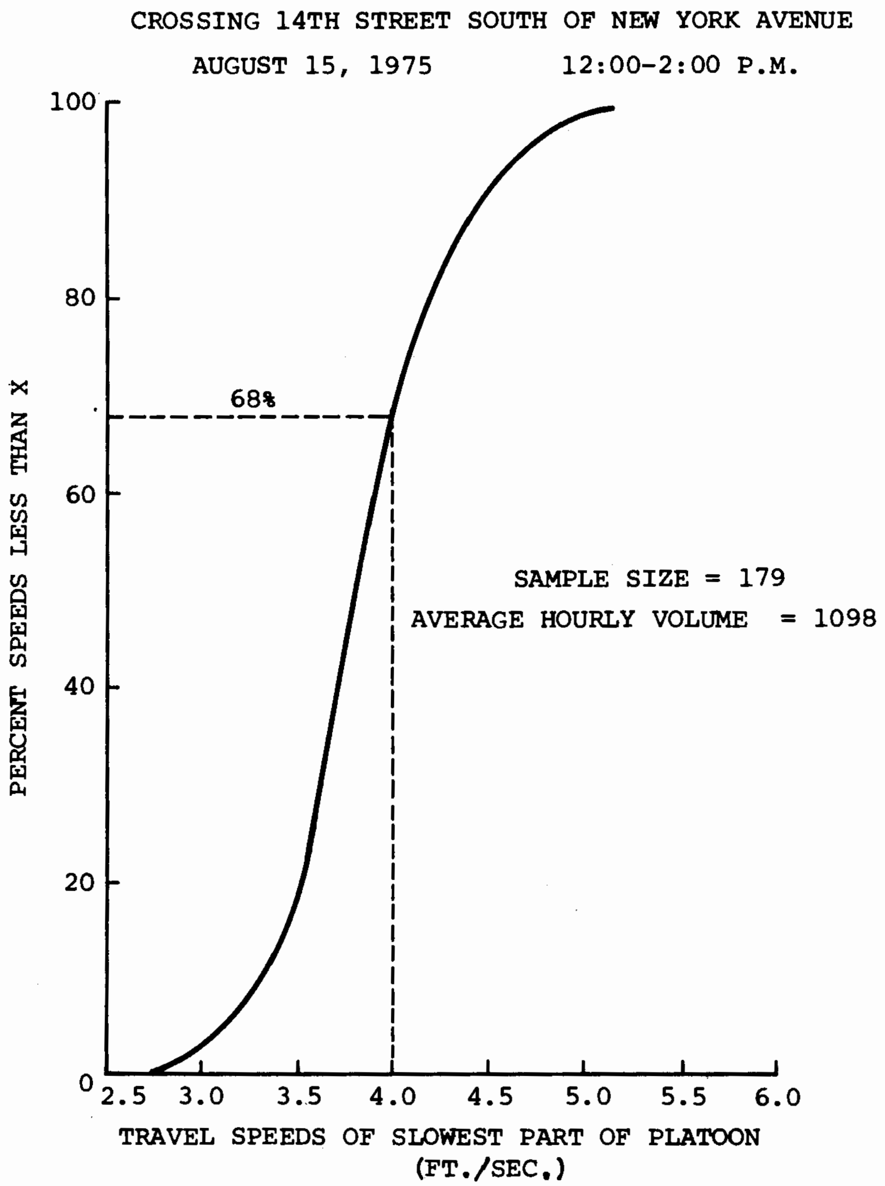


FIGURE 35. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS, WASHINGTON D.C., 14TH STREET AND NEW YORK AVENUE.

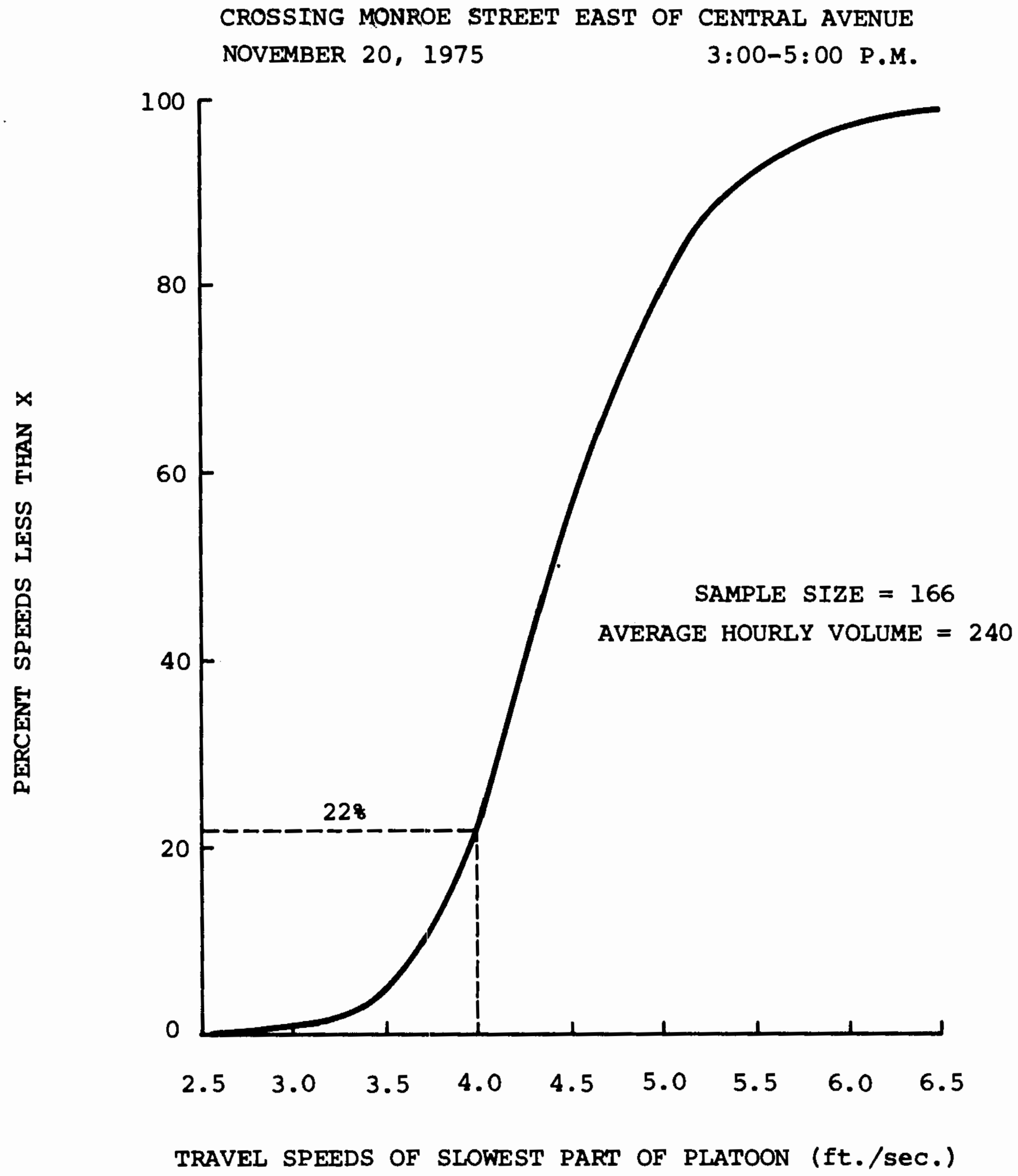


FIGURE 36. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS IN PHOENIX.

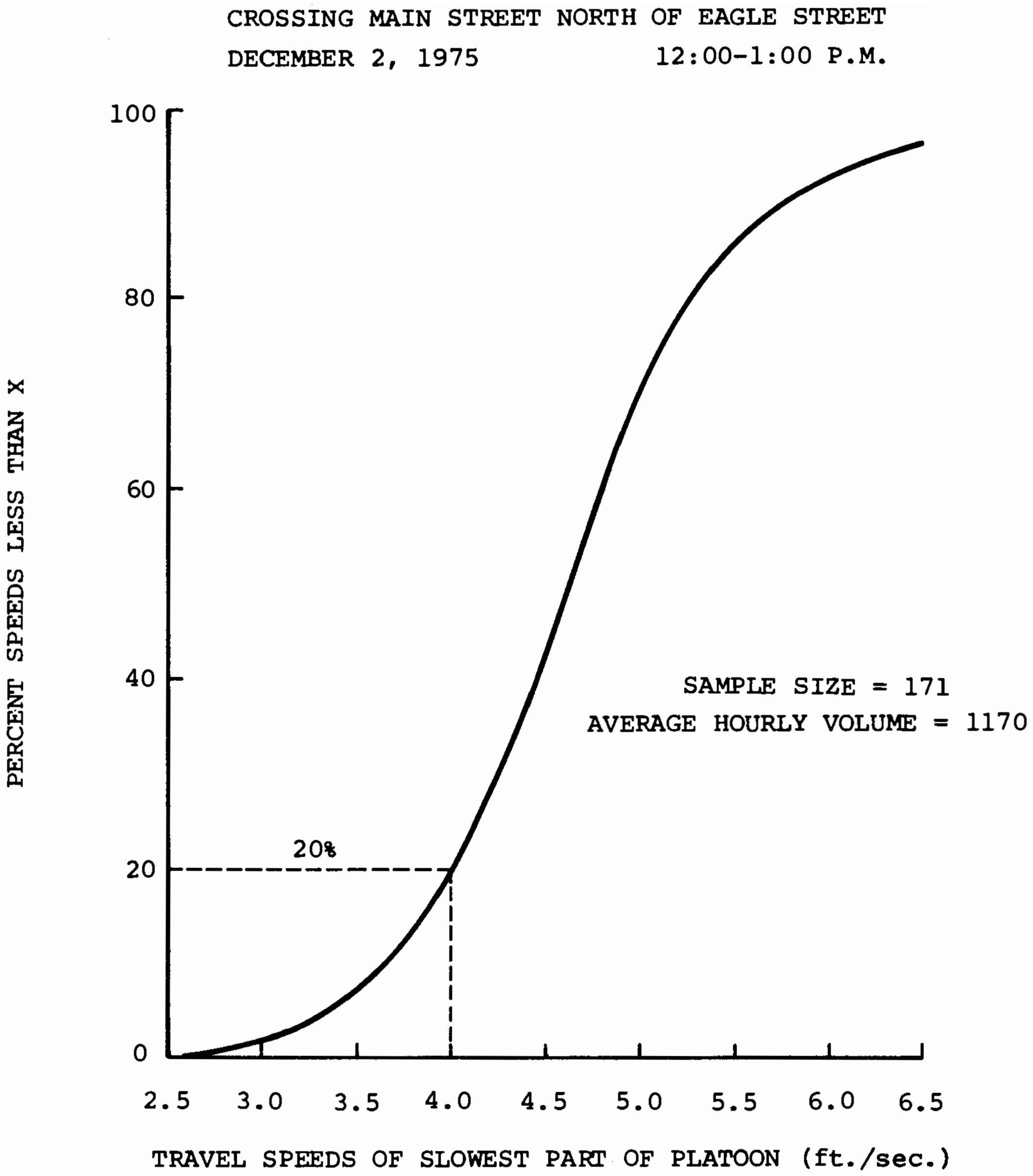


FIGURE 37. CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON IN BUFFALO.

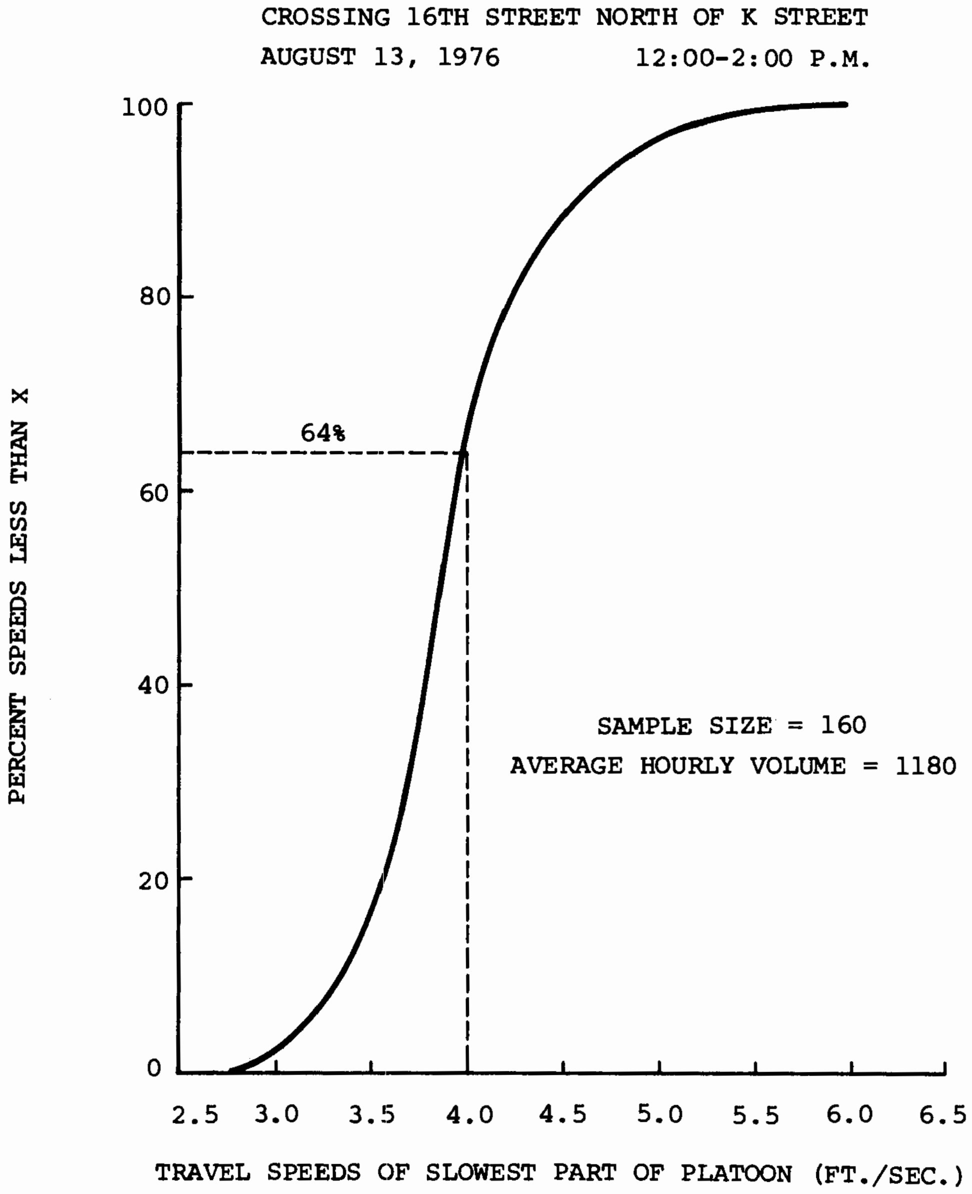


FIGURE 38 . CUMULATIVE DISTRIBUTION OF PEDESTRIAN PLATOON SPEEDS, WASHINGTON D. C. 16TH AND K STREETS, MIDDAY.

# APPENDIX C

## VEHICLE RIGHT TURN DELAY CURVES

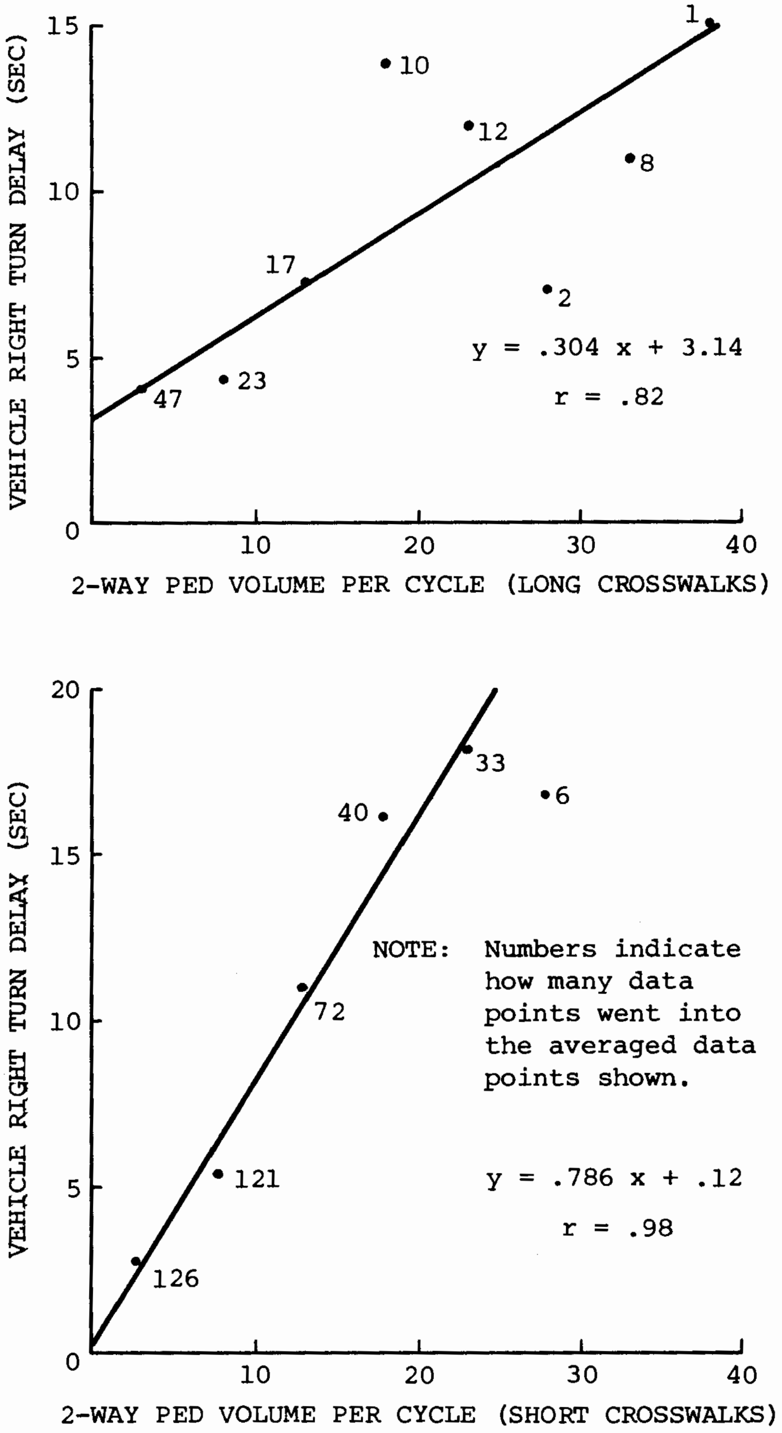


FIGURE 39. RIGHT TURN DELAY CURVES FOR t = 0.

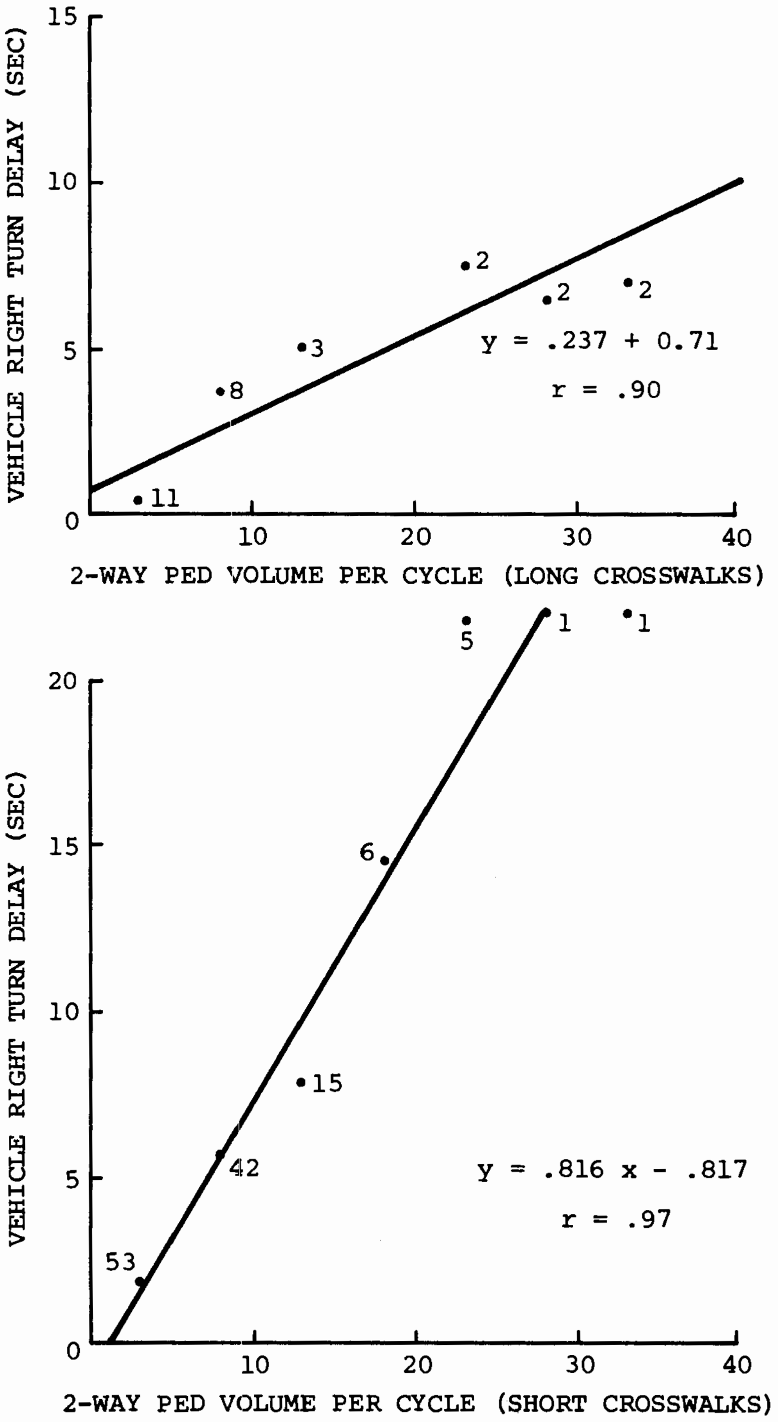


FIGURE 40. RIGHT TURN DELAY CURVES FOR t = 3.

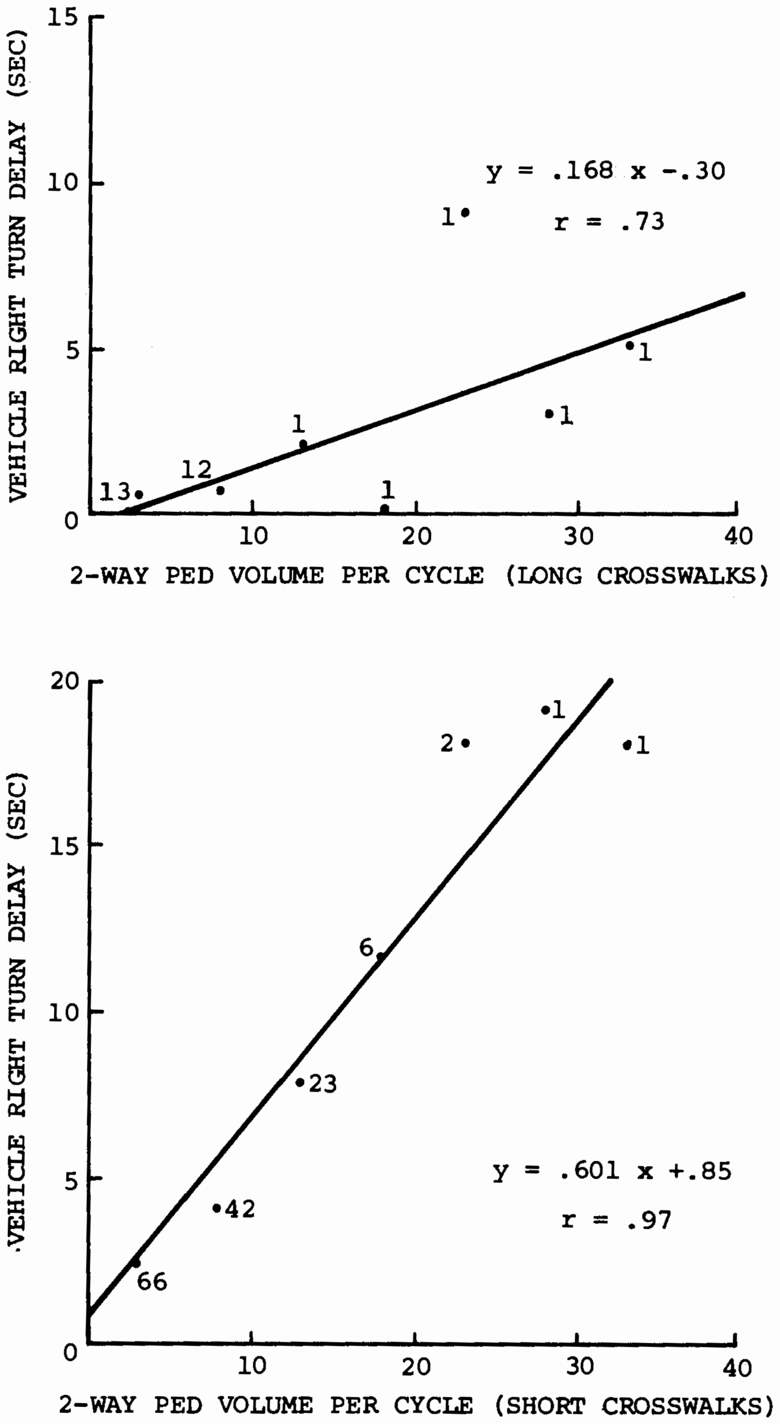


FIGURE 41. RIGHT TURN DELAY CURVES FOR t = 6.

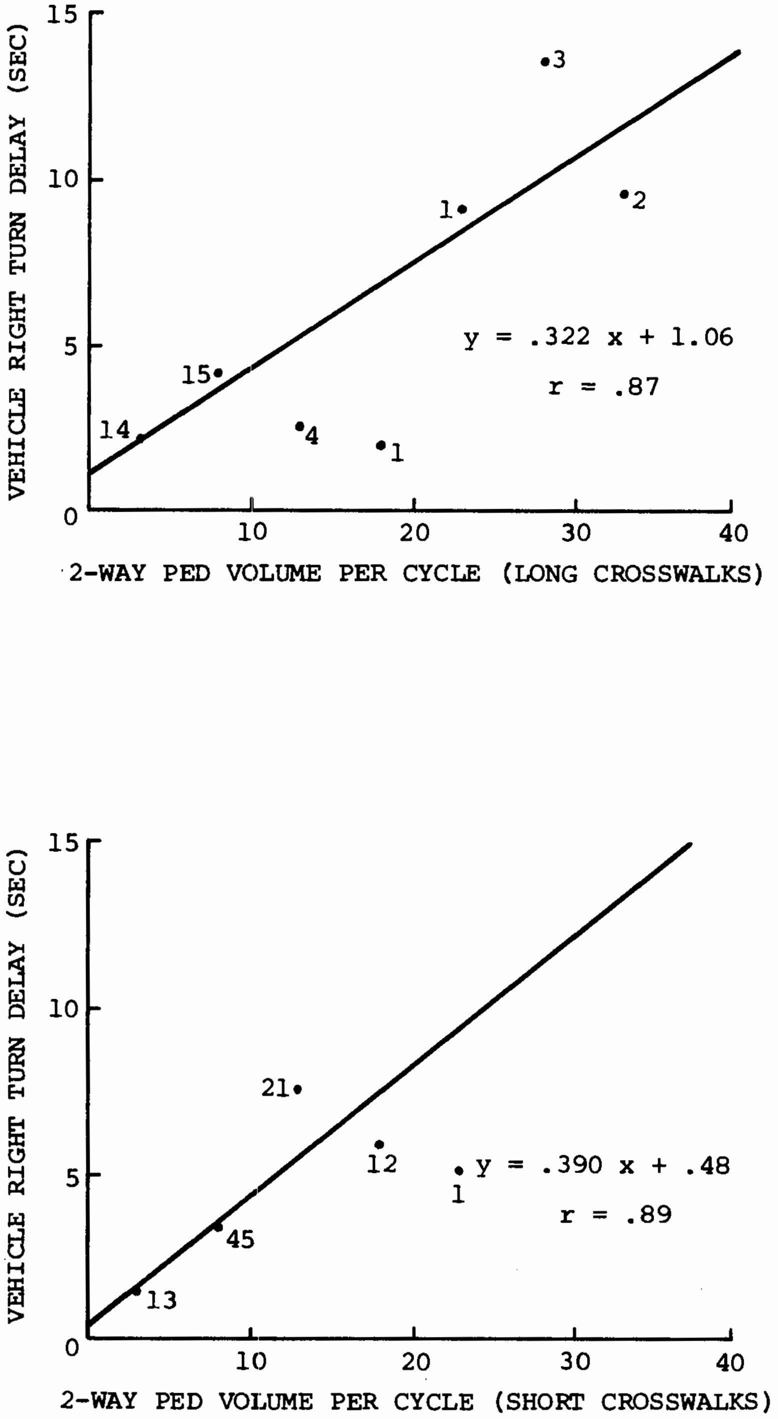


FIGURE 42. RIGHT TURN DELAY CURVES FOR t = 9.

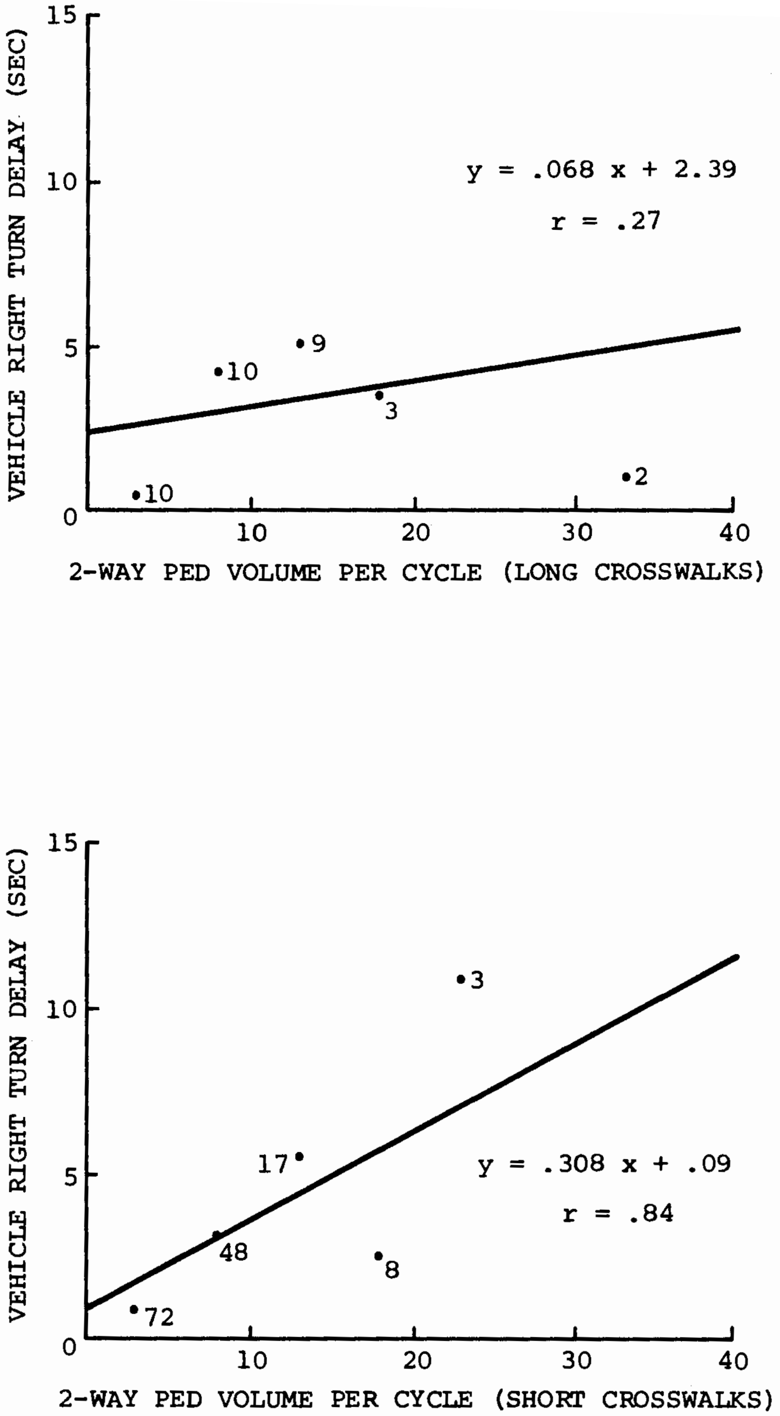
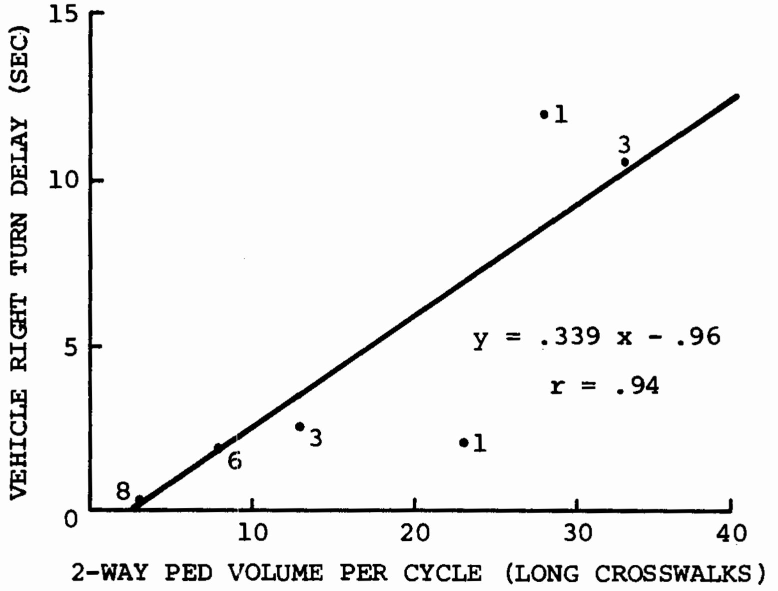


FIGURE 43. RIGHT TURN DELAY CURVES FOR t = 12.



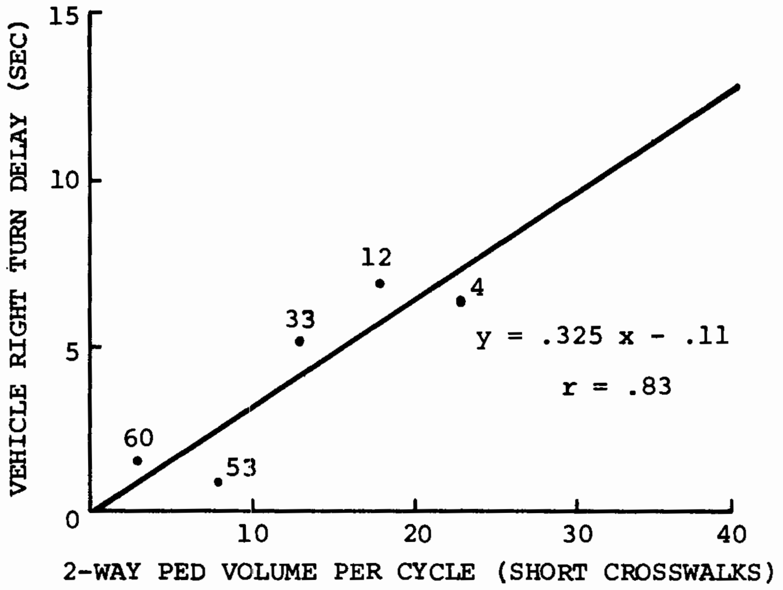
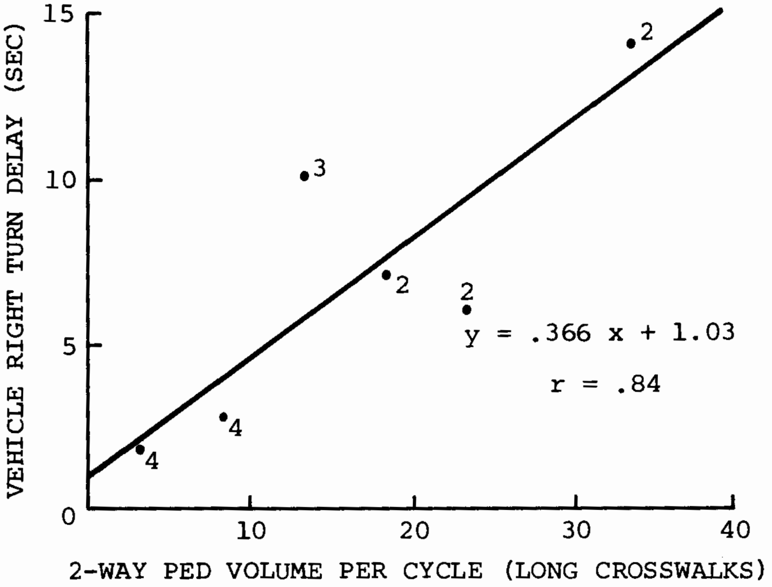


FIGURE 44. RIGHT TURN DELAY CURVES FOR t = 15.



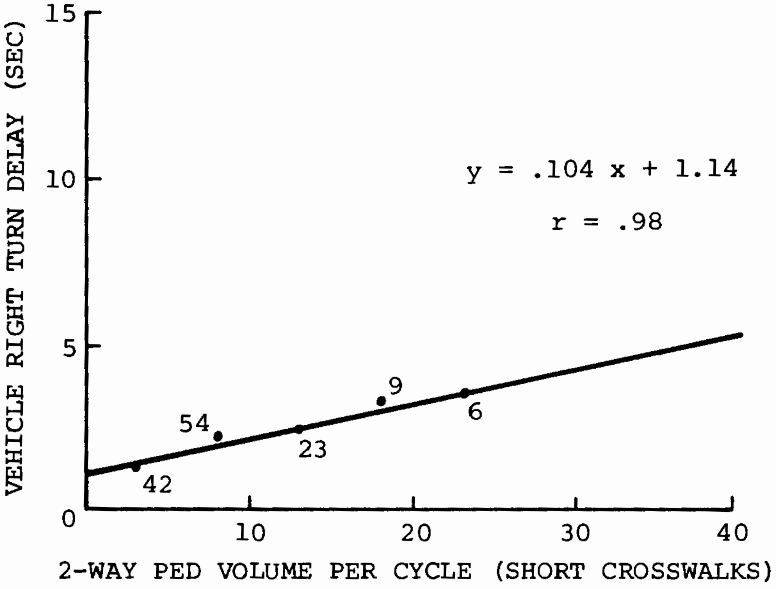
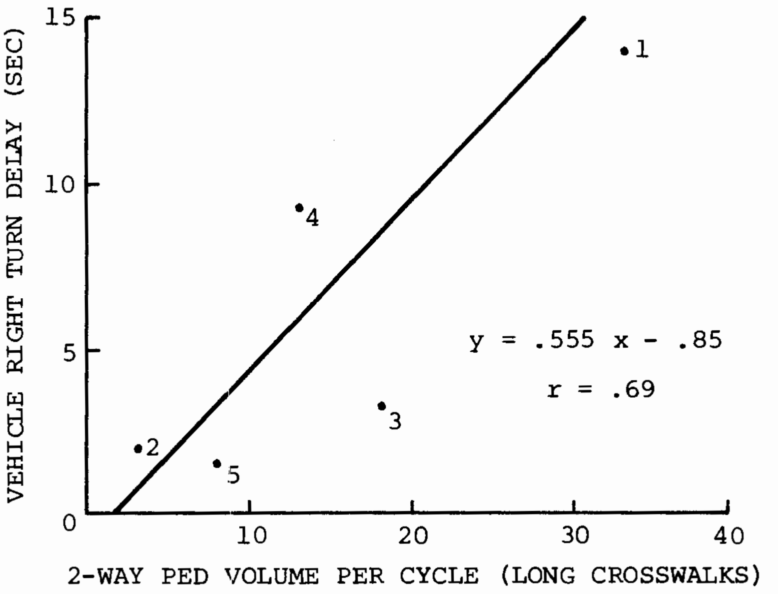


FIGURE 45. RIGHT TURN DELAY CURVES FOR t = 18.



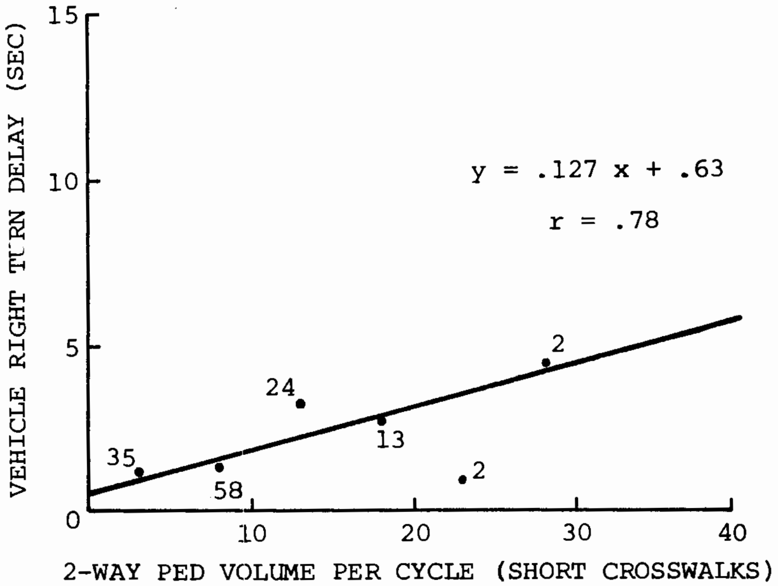


FIGURE 46. RIGHT TURN DELAY CURVES FOR t = 21.

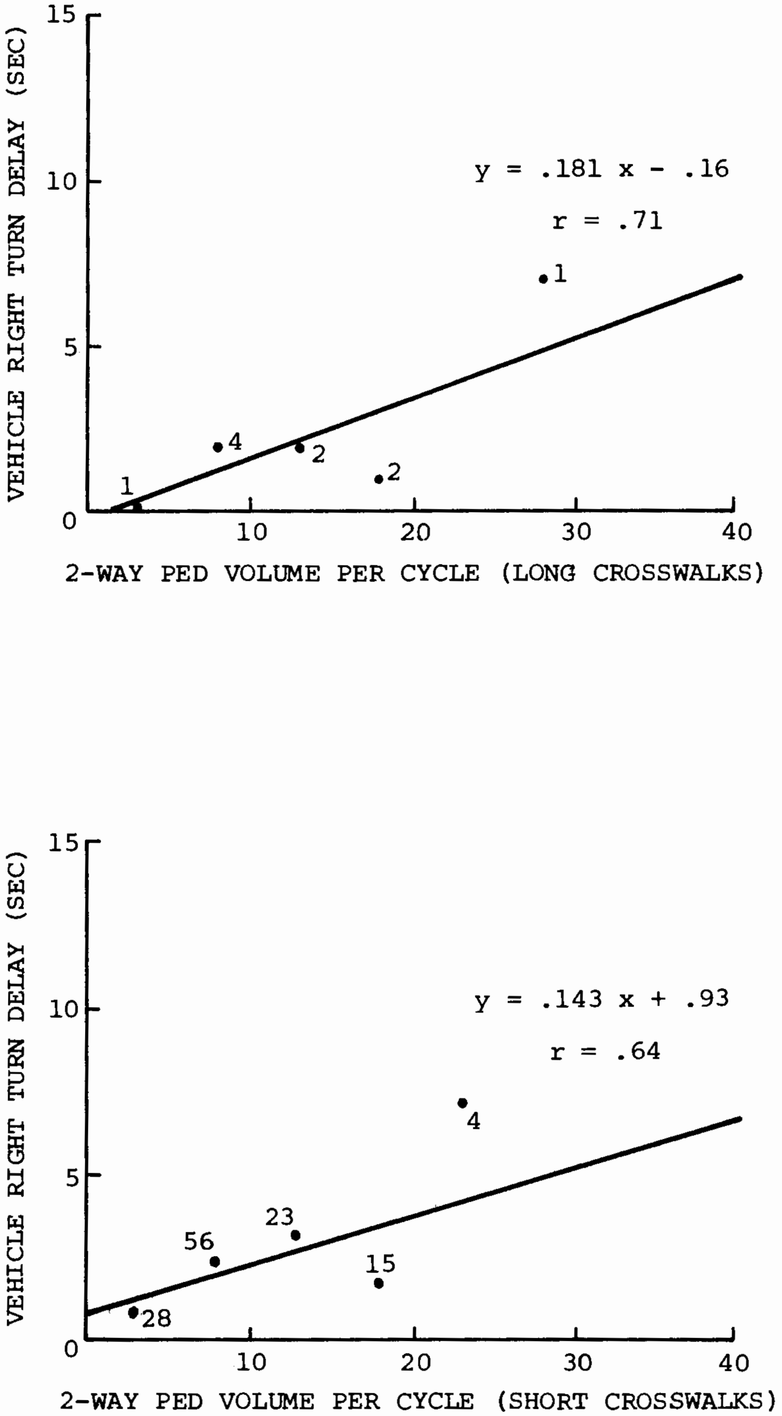


FIGURE 47. RIGHT TURN DELAY CURVES FOR t = 24.

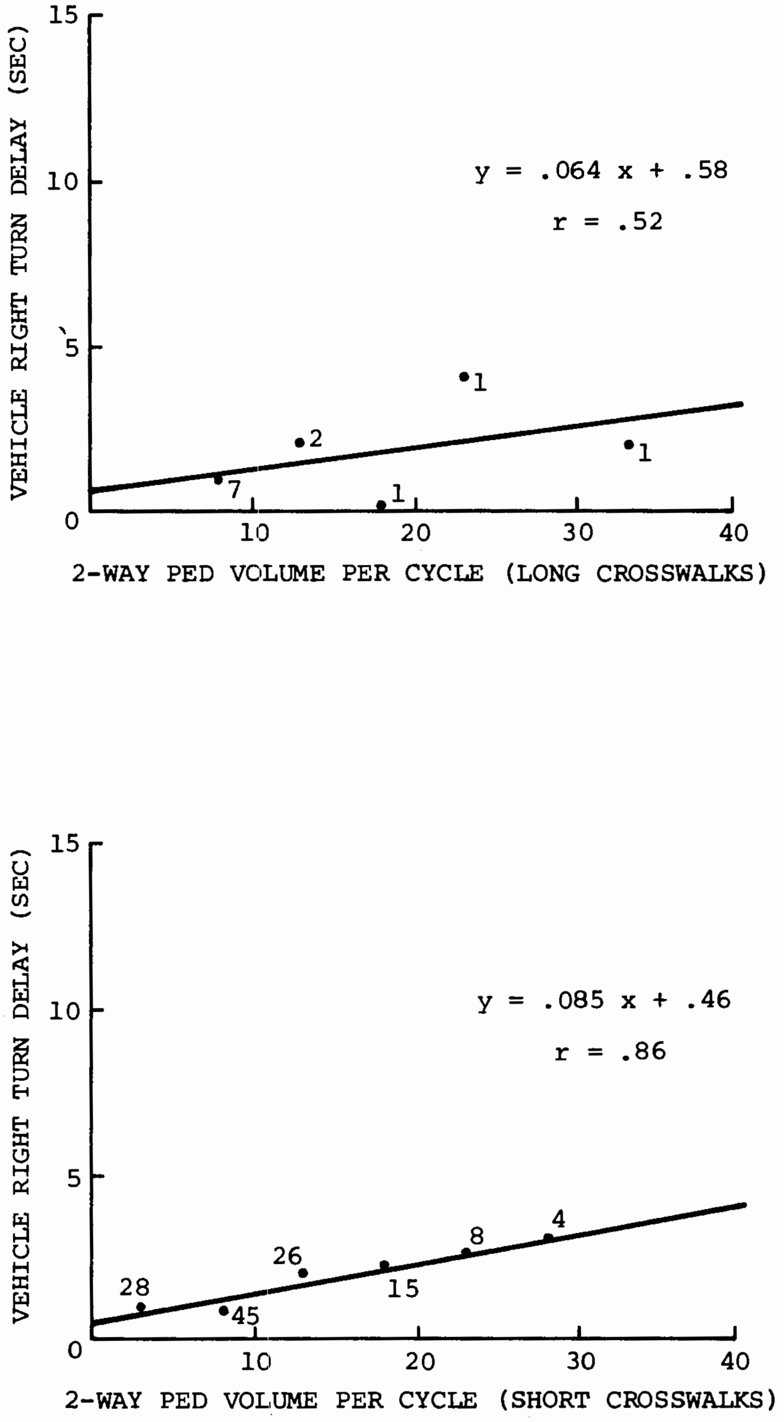


FIGURE 48. RIGHT TURN DELAY CURVES FOR t = 27.

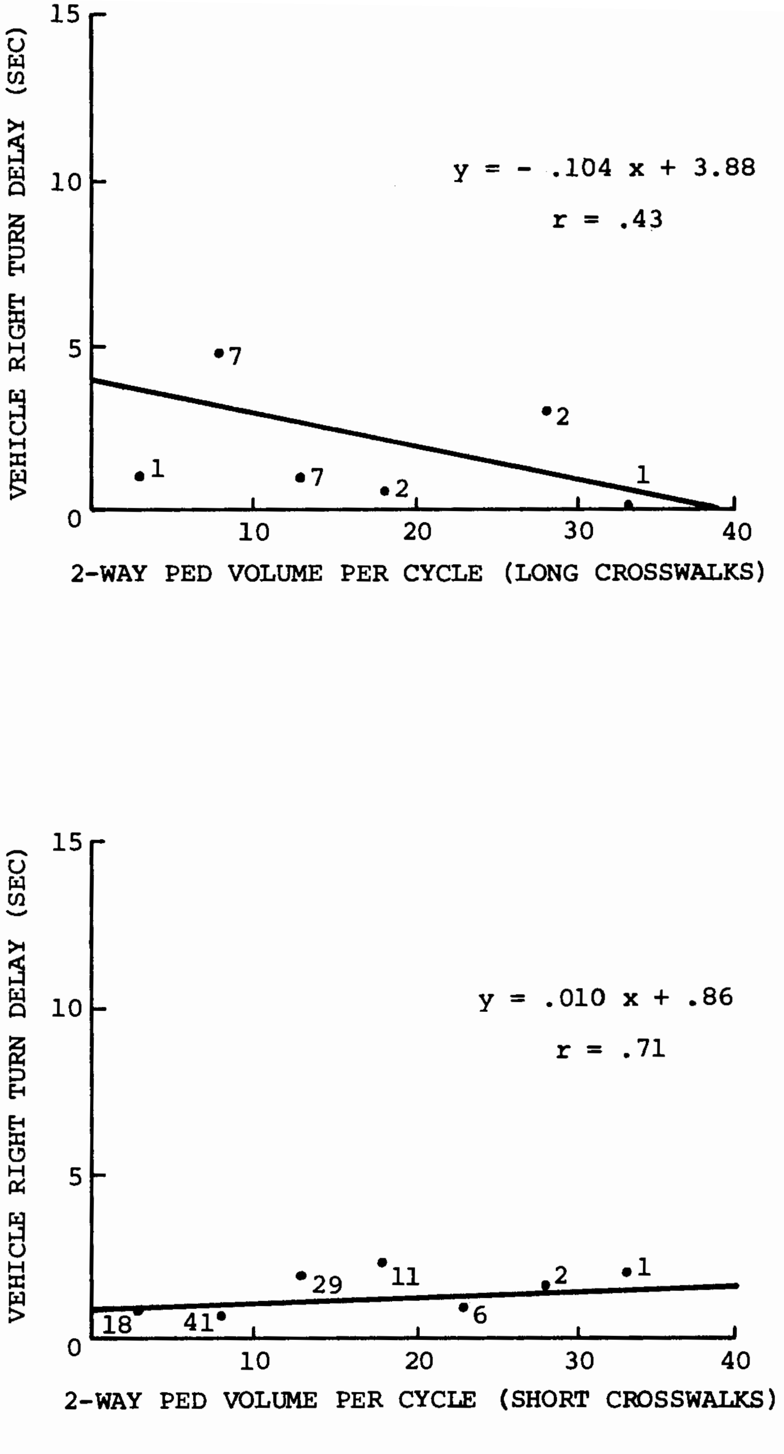


FIGURE 49. RIGHT TURN DELAY CURVES FOR t = 30.

*U.S. GOVERNMENT PRINTING OFFICE : 1978 O-724-859/1271

# FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

## *FCP Category Descriptions*

### 1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

### 2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

### 3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

### 4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

### 5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

### 6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

### 7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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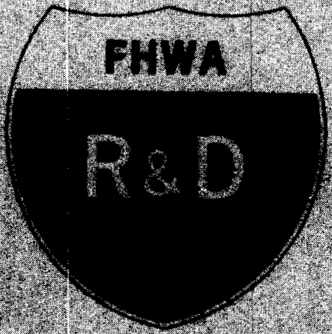
* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.


FHWA
R&D