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OCATING DETECTORS FOR ADVANCED TRAFFIC ONTROL STRATEGIES

echnical Report

R. D. Henry, S. A. Smith, and J. M. Bruggeman

September 1975
Interim Report

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This report will be of interest to both the research engineer and the traffic operations engineer who are concerned with network computerized traffic signal systems. The report provides the studies conducted to determine the detector requirements for the second and third generation traffic signal control strategies for the UTCS system in Washington, D. C. It provides the basic background information which was used to develop the handbook.

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16. Abstract

This report presents the results of a study to develop procedures for determining locations of detectors for advanced traffic control strategies. The project is a part of the continuing research for the Urban Traffic Control System/Bus Priority System (UTCS/BPS) in Washington, D.C. The work included defining criteria for surveillance data for 1st, 2nd, and 3rd generation UTCS strategies, identifying links requiring detectors, developing detector placement criteria within the link, preparing detector recommendations specific to the UTCS network, and developing detector placement guidelines for other locations. It was found that 3rd generation requires detectors on essentially all links in the UTCS network; 2nd generation requires at least every other link; and 1st generation approximately every fourth link. Critical intersections require data from each approach link. Within a link, a critical lane can usually be identified and that lane best represents demand. One detector, approximately 210 feet upstream of the intersection, tended to be adequate for volume, occupancy, and speed. This volume covers the details of the study and another volume contains the handbook information. The other volume is:

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INTRODUCTION

This report covers the work performed under Task D. "Locating Detectors for Advanced Traffic Control Strategies" of the project for "Evaluation of UTCS/BPS Control Strategies." The project reports include this volume, which is the technical report, and a second volume, which is a handbook for applying the methods developed for this project in other jurisdictions involved in designing a computer based traffic signal system. The project had the following as its stated objective:

> "...determine the optimum number and location of vehicle detectors on the approach to traffic signalized intersections in the UTCS area to provide the traffic surveillance information required to implement the advanced traffic signal control strategies."

The work, then, had the goal of developing specific plans for enhanced detection in the UTCS network sections that are to be used for testing the second and third generation strategies. These strategies are designed to directly respond to measured and predicted traffic conditions through on-line optimization and special control of congestion at and around critical intersections. A secondary goal was also considered throughout the conduct of the research project. This secondary goal was to report those procedures which appeared most transferable to the global issue of locating detectors for computer based traffic control systems. Near the completion of the project it became apparent that this secondary goal was achievable and that the "handbook" approach would facilitate distribution of the information. For this reason, a separate volume was developed to provide a handbook for detector placement. Details of field studies and analysis procedures are provided in the appendix. The information relating to UTCS site specific issues and research activities which were not deemed transferable are presented herein.

BACKGROUND

The UTCS/BPS project is a major research project of the Federal Highway Administration. The overall project is designed to develop and evaluate traffic signal control strategies which may be implemented using a computer based system. The project activities center around real-world application in a portion of Washington, D.C. The overall UTCS program is summarized in the report "The Urban Traffic Control System in Washington, D.C." prepared by the Department of Transportation, Federal Highway Administration and dated September, 1974. The features of the three basic strategies which are being developed, implemented, and evaluated are summarized in Table 1.

This detector locating study was initiated to identify changes or enhancements to the surveillance system which may be required in the operation of the more advanced forms of control--the second and third generation strategies. The study was also designed to resolve certain basic detection issues which have been identified as a part of other UTCS/BPS activities and other research and implementation work.

STUDY AREA

The second and third generation control strategies are to be implemented in two of the four sections of the UTCS network. These sections--Sections 1 and 3--are highlighted on Figure 1 which shows the UTCS area in Washington, D.C. Section 1 is primarily an arterial street which passes through a commercial area of Washington (Georgetown). The street carries heavy commuter traffic as well as serving the commercial-shopping activities. Section 3 includes major portions of the high density private office section of downtown Washington, D.C. and commercial activities. Limited summer visitor traffic impacts the southern and eastern portions of the section. Section 1 includes 11 signalized intersections and Section 3 has 45.

SUMMARY AND FINDINGS

The following is a summary of the project effort in identifying links in the UTCS network which require detectorization and in locating detectors within the links when so identified. Basic findings from the effort are also summarized. The research team feels that many of the generalized procedures and findings are applicable to other signal system projects currently being designed or implemented. The handbook noted earlier describes the more straight forward procedures which may serve as a reference when determining detection requirements.

Links Requiring Detectorization

Two separate efforts were undertaken to identify links which required detectorization. The first effort involved identifying intersections that should be controlled under a critical intersection mode (CIC) -- all approach links to these intersections require dectorizing. The second effort was directed at identifying other links which are important to system control. The efforts related

Table 1. Features of UTCS/BPS strategies.

FEATURE	FIRST GENERATION	SECOND GENERATION	THIRD GENERATION
Optimization	Off-Line	On-Line	On-Line
Frequency of Update	15 Mintues	5 Minutes	3-6 Minutes
No. of Timing Patterns	Up to 40 (7 used)	Unlimited	Unlimited
Traffic Prediction	No	Yes	Yes
Critical Intersection Control	Adjusts Split	Adjusts Split and Offset	Adjusts Split, Offset, and Cycle
Hierarchies of Control	Pattern Selection	Pattern Computation	Congested, Medium Flow
Fixed Cycle Length	Within Each Section	Within Variable Groups of Intersections	No Fixed Cycle Length

Source: "The Urban Traffic Control System in Washington, D.C.," U.S. Department of Transportation, Federal Highway Administration, September, 1974.

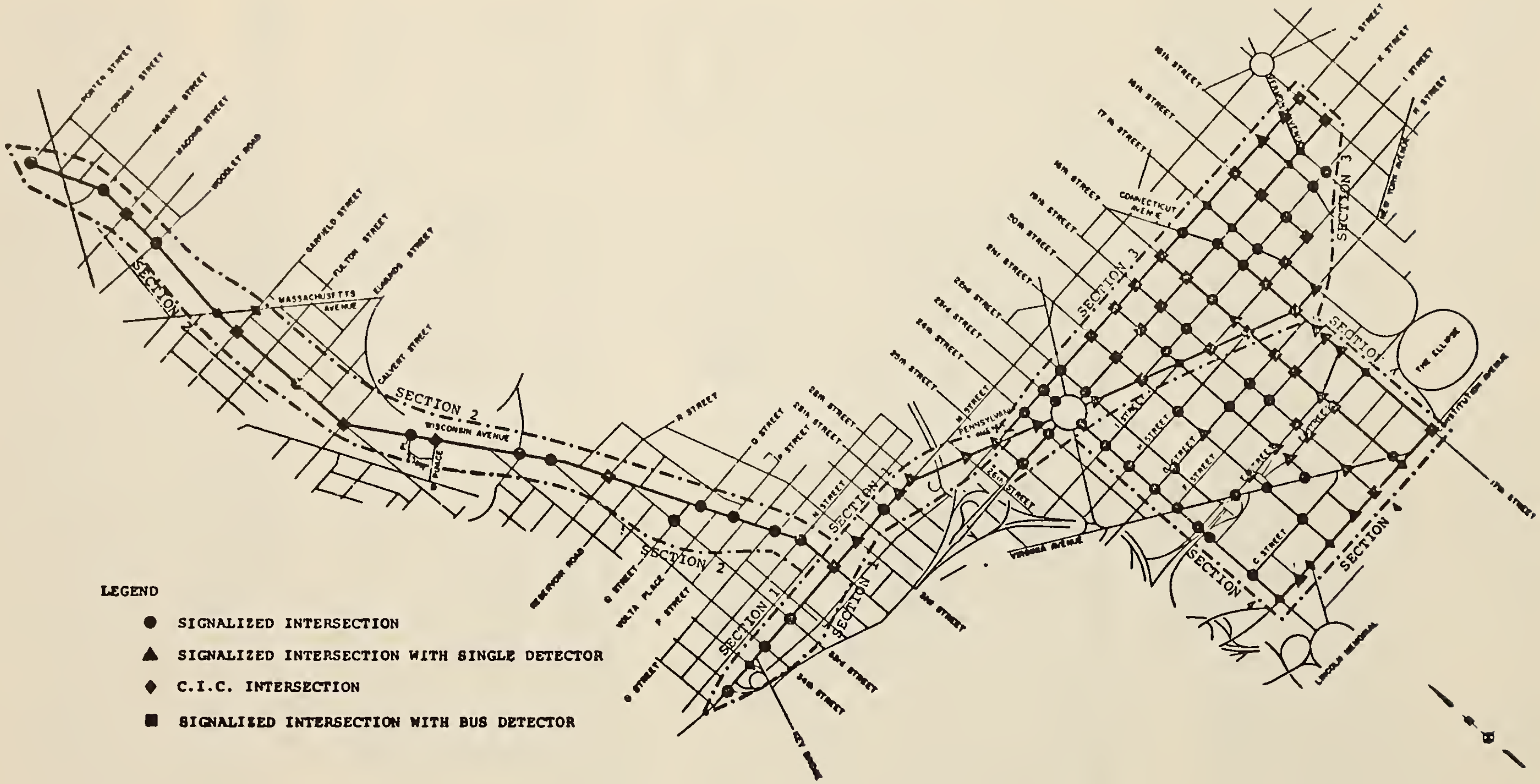

Figure 1. UTCS network map including signal locations and section boundaries.

to first through third generation UTCS strategies. As a point of note, Section 3 of the study area has an extensive surveillance system as part of its initial implementation program.

Local Control Related Requirements

Four different approaches to identifying local control requirements were used. These were existing CIC criteria, volume/capacity measures, analysis of system measures of effectiveness (MOE's), and cycle failure measures.

Under the existing CIC criteria (primarily one of engineering judgement by observing heavy fluctuating demand on two or three conflicting phases) 2 of 11 and 28 of 45 intersections in Sections 1 and 3 respectively had been selected for CIC control. The criteria appears to have identified the vast majority of locations requiring CIC in the ITCS network. The procedure is not, however, transferable and may have selected intersections which were not really susceptible to improvement with a first generation CIC algorithm.

Using simple volume/capacity analyses, an attempt was made to refine the CIC selection procedure. The method did not prove effective as a sole selection method in the UTCS network. The method tended to understate the problem at intersections because of interference of downstream congestion, cross-street blockage, and pedestrian interference. The method does appear applicable to jurisdictions with a less congested core area and as a "first pass" for all areas.

The analysis of MOE's is unique to an area that has an operating surveillance system such as the UTCS network. The procedure involved comparison of volume and occupancy MOE's for conflicting approaches and quantifying the "lack of fit" patterns. The procedure proved to be too sensitive to differences and was impacted by equipment failures. It is not transferable to most locations.

The cycle failures measure involved field observation of "cycle" failures and "queue" failures at all intersections. The intersections were then ranked by a form of summed percentage of failures. The higher the percentage of failures, the more critical the intersection. This procedure did provide an apparently sound basis for quantifying the need for CIC control and is transferable to other locations. The procedure is relatively simple to use.

The research effort indicated that two additional intersections in Section 1 should have CIC control -- M Street at 33rd and 34th Streets. Five additional locations in Section 3 were also identified. As a point of emphasis, however, it was apparent from the review of data in Section 3 that the majority of the intersections are operating in an unstable condition and could require CIC. Further, because of the concept of queue management inherent in third generation operation (control of queue buildup and spread of congestion) and because of the number of CIC locations, all intersections will be involved and must be detectorized.

System Control Related Requirements

The last point, all intersections in Section 3 will need surveillance for the third generation QMC/CIC operation, supersedes the need to investigate individual links. It did not, however, negate the need to investigate link requirements for the other control strategies or for developing procedures to be used in Section 1 for developing surveillance needs. Four approaches were considered -- identification of unimportant links, link MOE comparisons, offset benefit analysis, and review of primary/secondary volume ratios.

The analysis of unimportant links hinges on finding those links whose demand never exceeds the minimum green time to be apportioned to the phase to which the link is assigned. This means that within a reasonable range of cycle lengths, the safe pedestrian minimum will always exceed the vehicular demand. Several approach links were identified as "umimportant" using this technique, which is transferable. These links were along the arterial in Section 1.

Using the existing UTCS surveillance system's MOE's statistical comparisons of data from adjacent links were made. Tests of goodness of fit were used to compare data for one, two and three links beyond a given detector. This test was repeated throughout the study area. The results indicated that, on streets with relatively uniform flow, detectors should be placed approximately every four blocks for traffic responsive control. Two block spacing can be used for the more responsive second generation control strategy. The procedure is dependent upon surveillance data, however, the results appear to be transferable.

The offset benefit procedure involved quantifying the need to maintain an offset for a given link. The procedure considered the arrival patterns at links within the area and the splits available at downstream intersections. It is a ratio of the time required to clear the primary volume and the green time available at the downstream intersection. A ranking of the links was developed to indicate importance for detectorization for second and third generation strategies The procedure is transferable to other locations.

Third generation control uses primary volume in the optimization algorithm. Primary volume is the major flow from the upstream intersection and is considered as the flow for which offsets should be designed. The research team reviewed primary flow measures in the network to determine its requirement for surveillance. The work indicated that primary volume cannot be determined by present detector processing with historical calibration and meet the accuracy goals given later in this report. Two alternative schemes are noted -- both relating to real time measurement of primary volume. The links requiring surveillance for primary flow similar to those identified by the offset benefit methods.

Findings

The following reflect the summary of findings from the above work.

- Two intersections in Section 1 and five in Section 3 should be added to CIC control for first and second generation strategies.
- All intersections in Section 3 require surveillance for the third generation QMC/CIC control mode.
- Detectors are required every 2 to 4 blocks for traffic responsive control especially for the advanced strategies.
- Historical data does not provide the information needed for primary volume and for the more advanced strategies.
- Transferable procedures assisting in the identification for links requiring detectorization have been developed. They are: identifying CIC locations with volume/capacity analyses and with cycle failure studies, and assessing other link requirements based on "umimportance", offset benefit, and distance between surveillance points.

Detector Placement Within a Link

Detector placement within a link is discussed in three areas, longitudinal placement, critical lane analysis, and sink/source impact. The longitudinal placement analysis considers the number and placement of detectors in a lane on a link. The critical lane analysis addresses the question of number of lanes to be detectorized. The sink/source investigation examines the effects of sink/sources on the two earlier analyses.

Longitudinal Comparisons

The UTCS system currently uses two or three detectors per lane for the more sophisticated surveillance locations. The Q1 detector is placed approximately 35 feet from the downstream stop line, the Q2 210 feet, and the Q3 approximately 325 feet. Data from the three detectors are averaged for most measures. The longitudinal comparison study reviewed this practice in light of additions, deletions, and continuance of the practice at other locations. A note of emphasis -- some aspects of the analysis could not be divorced from the algorithms which process the data.

To examine the detector placement question, both automated and manual studies were conducted. Temporary loops were installed at 62 locations and connected to the UTCS communications system. Raw history tapes of pulse lengths from the temporary and permanent detectors were collected and processed. The data were analyzed using the UTCS statistical processor and were also compared to field data. The information was reviewed with respect ot observed data versus surveillance data, surveillance data by detector position, speed comparisons, and effectiveness of loops extending over several lanes.

The longitudinal issue, as addressed by comparing observed values with detector values, concentrated on determining if the surveillance data provided an accurate representation of actual volumes. Comparisons were made at all the temporary loops covering ten links in detail. Other links were examined to confirm results. The comparisons indicated a high correlation between observed and measured values. The cycle by cycle comparisons indicated that demand at the Q2 location was well within the error range of 1-3 vehicles per cycle. A detector tendency to overcount was noted, with the average overcount being less than one vehicle per cycle. The overcounting was directly related to having one vehicle impact two loops. The overcounting was worst at the Q3 location where lane changing is prevalent.

A major effort was expended in comparing information at the Q1, Q2, and Q3 locations. Detector counts were compared against one another and against observed value. Automated analysis routines were used for the majority of comparisons. The results of this work represents the most significant findings. The volumes at Q1, Q2, and Q3 are significantly different far more often than not. Further, the average is different from observed values. This relates to driver characteristics along the link. At the Q3 location, drivers are still making many lane changes and are impacted by turns onto the link. In some cases the Q3 values relate more to upstream characteristics than to downstream demand. At the Q1 location, values reflect signal timing and intersection operation instead of demand. The Q1 detector was installed to "count out" for the queue algorithms. The research effort found little or no correlation between observed and computed queue. A new algorithm is needed and is being developed under other UTCS contracts. Unless found essential in this development work, the research team feels that one detector at approximately the Q2 location is more representative of traffic demand than the averaged three detectors.

Speed comparisons were also made as this measure is used in the advanced strategies. Again, it was found that the averaged "speed" did not represent free flow speed. A bias occurred because of vehicles stopped over the Q1 detector and others clipping the edge of the Q3 detector. The Q2 detector data were most representative of free flow speed. It is necessary, however, to edit incoming data and not use unreasonable values. Speed loses the "free flow" connotation when congestion occurs and historical values may be required.

Studies were also conducted to assess the effectiveness of one loop covering several lanes versus several loops, each covering a single lane. The studies related to the issue of greater lane coverage without corresponding increases in detector costs. The results of the study indicated that the multilane loops did not provide data of sufficient accuracy to be used by the advanced algorithms.

Critical Lane Analysis

The UTCS system approach has been to define the lane which exhibits the greatest demand on the link and concentrate detection in that lane. Studies were conducted to determine if the lane could be identified and if, in fact, one lane could be used to provide the necessary surveillance data. Four procedures for identifying and comparing critical lanes were developed and tested. The critical lane data were examined for time of day patterns, the accuracy of critical lane volume measures, and the relationship of primary volume in the critical lane compared to total approach.

Studies were taken throughout the network to determine critical lane demand on a cycle by cycle basis and to observe characteristics by time of day. Although some patterns shifts were noted -- particularly on wide one-way streets with changing parking restrictions and on major two way streets with part-time turn restrictions -- the majority of the patterns held constant throughout the day. The shifts which did occur were generally related to cycle by cycle counts and not overall values.

To relate shifts which do occur and to observe normal variation in demand, cycle by cycle, lane by lane studies were conducted on representative links in the network. The observations were analyzed to assess the impact of the variations to the accuracy goals of the various levels of control. It was generally found that critical lanes could be identified such that they always exhibited a demand that was within the 1-3 vehicle per cycle condition. That is, even if a second lane were "critical" for a cycle, it exceeded normally critical lane value by less than three vehicles. At those locations where this is not true, a second lane must be detectorized so that the accuracy goal is achieved.

Observations of primary flow (as needed by the third generation algorithm) related to critical lane versus total approach were also conducted. The studies also investigated the variability of the ratios on a cycle by cycle basis. The research effort indicated that the critical lane ratios were representative of total approach primary flow ratios.

Sink/Source Analyses

The effects of major sink/sources on the observed traffic measures were investigated. These sink/sources (namely major garages) were examined in light of their impact on total approach and critical lane measures. Their impact on longitudinal placement of detectors was also observed. The investigation related to special surveillance which might be needed to provide the required accuracies on links which contained one or more major parking facilities.

Consideration was given to the impact of lane changing and the simple loss or gain related to turning movements into the parking facility. In general it was found that lane changing was not a major issue -- especially when considering detectors at the Q2 locations. Many more changes were found to occur because of double parking, passenger drop-offs, etc. It appeared that changes occurred more in advance of the upstream intersection than at the sink/source. The only cases where substantial lane changing occurred was where there were queues on-street trying to enter the garages. This did not occur frequently.

Turning movement counts were taken at fourteen major parking facilities in Section 3 of the UTCS network. It was found that the volume changes (primarily those during the a.m. peak) were such that, on a cycle by cycle basis, a significant difference occurred. These changes, as high as ten vehicles per cycle, occurred generally from the curb lane.

When related to critical lane values, the sink/source impact takes on a different perspective. Lane by lane studies for both entering and exiting vehicles and the total link flow were made. It was found that, as noted above, most entering vehicles would not impact downstream demand in that they were in the non-critical curb lane. Exiting vehicles were found to move almost immediately into the lane that they continued in to the downstream signal. The effects of sink/source were not such that they moved the accuracy of traffic measures, in the critical lane out of the accuracy range. Assuming detection at a Q2 location, the observation became even more conclusive. If the source is upstream of the detector, the detector should be at least 50 feet beyond the driveway.

Findings

The following findings are noted by the research team regarding placement within a link.

- Detectors for UTCS should be placed at the Q2 location for volume, occupancy and speed measures (over 200 feet upstream and a minimum of approximately 100 feet downstream). Location for queue is dependent upon the new algorithms -- the current procedure does not meet accuracy goals.
- The single detector located at the Q2 position more closely reflects observed traffic measures than do the averaged three values.
- A multilane detector does not accurately provide measures for UTCS.
- Primary flow must be measured if the short term accuracy goals are to be met.
- A single lane detection system, i.e. critical lane, reflects demand for green time at an intersection.
- The critical lane is generally constant (second lane from right) for the cycle by cycle measures. At questionable locations studies are requred and it may be necessary to place detectors in the second most critical lane.
- Sink/sources do not appear to significantly impact critical lane measures and do not generally require special detection.

REPORT ORGANIZATION

As noted, this report is prepared in two volumes, with this volume being the technical report and the second volume being the handbook. In addition to the brief introductory chapter, this report contains the following chapters.

Work Plan Overview

This chapter presents a brief overview of the work plan and methodology followed in completing the project. The initial issues are also noted in this section. Certain field procedures unique to the UTCS study area are also noted and described in detail in Appendix A.

Existing Detection System

For reference and perspective, the existing detection system and surveillance processing is described. The rationale for initial detector placement is also discussed.

Computer Processing Techniques

Special purpose computer processing techniques were used for gathering and analyzing data for this project. The programs and their application are discussed in this chapter and a users guide is contained in Appendix B.

Requirements of UTCS Control Strategies

This chapter discusses the various control strategies and their requirements for surveillance information. The requirements are discussed both for areawide control and the alternatives of critical intersection control. The design goals for the data accuracies are also noted and a summary set of needs identified.

Identifying Links Requiring Surveillance

This chapter addresses the question of which links in the system require surveillance. Two perspectives are presented, one for links approaching intersections needing critical intersection control and another relating to other important links needing surveillance.

Detector Placement Within a Link

Given a set of decisions on which links require surveillance, this chapter discusses placement within a link. Consideration is given to both longitudinal and latitudinal placement as relates to the various items being measured and/or computed.

Summary and Conclusions

This chapter presents an overview of the project results and discusses general conclusions which may be drawn. The procedures which appear most transferable are also noted with references to Volume II. of this report.

UTCS/BPS Detector Recommendations

The site specific recommendations for Sections 1 and 3 of the UTCS network are described in this chapter.

WORK PLAN OVERVIEW

INTRODUCTION

The following is a discussion of the work plan followed for the project. The work plan overview is concluded with a discussion of the issues, or points of emphasis, that guided the final work efforts. This work plan overview is supported by the handbook and by Appendix A - Discussion of Procedures.

DATA COLLECTION

Existing data in the form of turning movement counts and intersection pavement marking drawings were obtained from the Washington, D.C. Department of Highways and Traffic. Additional traffic data was obtained from the UTCS project group. This 1974 data was originally used to calibrate the UTCS-1 simulation model. The data was converted to a basic intersection turning movement format. This existing data was supplemented with geometric survey data for each intersection in Sections 1 and 3. This survey data included the lane usage and turning movement controls for each approach in the network. This data was combined with the count data to obtain an intersection capacity measure.

Additional manual data collected for the detector placement task included: turning movement counts, classification counts, lane discharge flow rates, and stratified volume counts by critical lane and total approach, and by primary and secondary flows.

DETECTOR PULSE TAPE AND TEMPORARY LOOPS

A large amount of data collected during this phase was done utilizing the UTCS computer and communications systems. This data was recorded on a "Raw Data" tape which held approximately 30 minutes of real-time input. The detectors and communications gear from links not used in the test, such as cross street instrumentation, were used to transmit data from temporary loops taped to the pavement at test locations on the links. The layout, locations, and procedures for the temporary loops are noted in the Appendix A.

The UTCS/BPS system has the capability of generating a history of the pulses from all detectors at 1/32nd of a second intervals. The pulses, consisting of a simple "on-off" status, are written onto a magnetic tape. Additional data, such as the advance pulse and the A-phase green return, are produced as well.

The pulse tape processing is handled by one of the BPS routines using the BPS computer and its associated tape drive. Thus, some form of BPS control must be "up" to generate the data. The system can be "up" with the controllers off-line or with the computer controlling intersections. The latter was the preferred approach when practical because the 15-minute MOE summary tape could be produced at the same time, providing additional data and a check on accuracy.

The research team developed an independent pulse tape processing program to utilize the surveillance system for the detector placement study. This processing capability was developed for use on government furnished computer equipment. The program is described in the COMPUTER PROCESSING chapter.

The software produces both printed reports and output data sets. The latter was used more directly by the existing UTCS post-processor, developed for the evaluation project.

Concurrently with the generation of the pulse tapes, field studies were conducted to obtain equivalent measures from conventional traffic engineering practices. Data sets were produced from these coded data and, with minimal manipulation, converted to post-processor inputs and compared against alternative surveillance measures.

DETECTOR PLACEMENT ISSUES

To focus the detector placement research on the specific requirements of the individual control algorithms, a set of issues were identified early in the project. These issues were reviewed and priorities were assigned with the issues having the most relevance to the current UTCS research given the highest emphasis. Because major issues provided the structure of the work effort, they are discussed below. The first three issues are global in nature and apply equally well to any algorithm that requires real-time surveillance. The remaining issues, while frequently being applicable to more than one control strategy, are specific parameters oriented, and therefore, are relevant only to the algorithms that require these parameters. The discussion of the direct relation between issues and the UTCS algorithms are discussed in a subsequent chapter.

Impact of Selecting a Critical Lane Detection Scheme

Given that critical lane detection provides appropriate measures, the identification of the critical lane becomes quite important. The problem is first to develop an unambiguous definition of "critical lane", then to measure the traffic patterns on each lane of the link over time to ascertain which is, in fact, the critical lane. Of great importance, if shifts in critical lanes are apparent, is the regularity of such shifts and whether they can be related to general parameters such as time-of-day or if some form of cross-lane monitoring is required.

Impact of Lateral and Longitudinal Detector Placement

Detector placement along the link for single-lane monitoring may give widely different surveillance measures under certain circumstances such as heavy turning movements, effective channelization, and mid-block disturbances. The effects are compounded when selecting between single and multiple detector configurations. Similarly, the lateral placement within a lane can cause missed pulses or double pulses, depending upon the situation. In addition, this phenomenon is likely to be highly sensitive to time-of-day and traffic volume levels. A few factors that affect the lateral placement include pavement markings, lane width, marginal friction, illegal maneuvers, and traffic composition.

Sink/Source Impacts

Sink/source impacts are closely related to the issue of lateral and longitudinal placement noted above. Some special characteristics, such as movements across detectors in the vicinity of sources and sinks and the lane distribution of sink/source vehicles, are considered with respect to detector placement.

Detector Location Impact on Speed Measures

Speed, as used in the various algorithms, is in a sense an idealized "free flow speed." A primary situation is the necessity of actually measuring such a quantity on-line, as opposed to using either a hypothesized value or a simple historical measure. Unfortunately, this issue strikes at the evaluation of the algorithm itself and was outside the scope of this study. Since a form of "free flow speed" is desired and this parameter is only defined in the context of an algorithm, several methods are assessed to determine the impact of detector placement. The most promising appeared to be the measurement of the speed of the first car in a platoon assuming that it is not seriously impeded downstream. The proper detector placement for such a measurement and the development parameters defining a time measurement "window" is developed.

Primary Volume Detectorization

Primary volume has been identified as critical to third generation performance. Unfortunately, direct detectorization to ascertain primary volume is an impossible task. Therefore, the main issue emerges to be whether simple relationships can be developed with total volume, or with some approximate method such as a history file. The variation of such relationships over the short time periods for third generation operation is an important issue that is addressed. The question is related to the critical lane detection scheme in that the primary volume relationships for the critical lane may be different from the values for the total approach.

Queue Detection

Queue, as defined within the context of UTCS, is simply the number of vehicles within the detectorized portion of a lane. Thus, the measure serves both as a demand for service upon the downstream intersection and the capacity of the receiving link for the upstream signal. The impact of lane changing on queue estimation is quite severe regardless of the detection plan or interpretation technique. It is most critical, however, when considered with a count-in/count-out algorithm where accumulated errors might be quite severe if the counts of the pair of detectors have a definite bias.

Non-Detectorized Links

It is thought that conditions may exist that would render any detectorization scheme ineffective. Where traffic volumes are extremely low, for instance, local timing would be dictated by pedestrian crossing requirements and there would be no need for CIC detections. Similarly, low volumes are normally characterized as being extremely erratic and hence, a low volume link would be a poor selection for a traffic responsive pattern selection detector.

EXISTING DETECTION SYSTEM

As an initial phase of this research project, a detailed review of the UTCS documentation was made to determine the rationale behind the existing detector placement. The basic sources of information were two reports, "Advanced Control Technology in Urban Traffic Control System," Volume 1, Sperry Systems Management Division, October 1969, and "Urban Traffic Control and Bus Priority System," Volume 1 - Design and Installation, Sperry Systems Management Division, November 1972, Report No. FHWA-RD-73-9.

INITIAL HARDWARE SELECTION

Because the Fine Arts Commission of the District of Columbia prohibits permanently installed devices requiring mast arms greater than six feet in length, overhead-mounted ultrasonic and radar detectors were excluded from consideration. The sonic detector mounted in a side fire configuration (on a curbside pole) could not be used because it cannot observe a uniquely identifiable lane. It was concluded, therefore, that only magnetic or inductive loop detectors could be used with UTCS. In choosing between magnetic and loop detectors the major factor was the inherent measurement accuracy obtainable from the unit. The quantitative accuracy of the magnetic detector is not as high as that of the loop detector due to the somewhat ill-defined and variable character of its field of sensitivity. The characteristics of the loop detector were satisfactory, and therefore the loop detector was selected for use in the UTCS test site.

DETECTOR PLACEMENT CRITERIA

The development of the detector placement criteria was closely related to the specific algorithm that uses the traffic surveillance data as input. Seven traffic parameters are used by the system; volume, occupancy, queue length, stops, delay, average speed, and travel time. These seven parameters were selected not only for their sensitivity to traffic movement, but also because their input requirements could be met with loop detectors.

The original detector locations were determined by developing general guidelines and then using data and observations at each link and intersection to modify the guidelines for local conditions.

Decisions on link instrumentation were made by classifying links into one of four categories:[1]

1. Grid area: A region which possesses a substantial number of links with high occupancy, density, and queue lengths for important parts of the day and in particular during the peak hours. In a grid area, representative links were instrumented to obtain volume, occupancy, and speed measures.

2. Major arterial: A major volume-carrying street crossed by many streets which carry substantially lower volumes. On arteries, the representative links were instrumented on the major streets to obtain volume, speed, and occupancy measures.

3. Critical intersection: The crossing of two or more links carrying volumes through an intersection which is at or near saturation for substantial periods and where, as a result, short term adjustment of split becomes critical. Each link entering a CIC location was instrumented to obtain all seven UTCS traffic parameters.

4. Special sources: The source locations which are significant monitor points for demand flows, such as approaches from bridges, tunnels, freeways, etc. Special source links were instrumented for volume only in the major direction of flow.

Because significant cost savings result if a representative lane can be selected for each signalized approach to measure the dominant characteristics of the traffic flow, a critical lane detectorization technique was employed. For example, if a straight-through movement is characterized by long queues and slow speeds in one lane, that lane would be selected as a basis for both control decisions and MOE studies. Where turning movements are specificially signalized, the appropriate lane most representative of this movement was detectorized to provide traffic-responsive capability for this movement. The selection of lanes to be instrumented thus involved study and observation of each of the

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[1] Source: Basic definitions included in Report No. FHWA-RD-73-9.

links. Where it was difficult to make a clear cut selection, at least one detector was sited in a parallel lane so that a comparison of traffic parameter measurements between the two lanes could be used to confirm the initial decision.

Both reports mentioned earlier stated that the number of detectors to use on an instrumented link is influenced by the competing requirements to obtain traffic parameter data and to minimize costs. Theoretically, the greater the number of detectors used, the more accurate the determination of the parameters. In practice, the reports concluded that satisfactory performance can be achieved with a limited number of detectors. This is so because meaningful traffic parameters are computed over relatively long periods, that is, over controller cycles and 15-minute periods. Smoothing of the detector outputs as well as smoothing of the computed parameters on a cycle-by-cycle basis results in considerable averaging of errors due to instrumentation and to unpredictable events such as lane changes, speed variations within a lane, lane blockages, pedestrian friction, garage friction, etc.

In summary, the reports concluded that one lane instrumented with a single detector could yield with sufficient accuracy the desired traffic parameters, including volume, speed, and occupancy. The reports further concluded that lanes instrumented with two or three detectors would, in addition, yield an accurate measurement of queue, stops, and delay and accuracy would be less sensitive to the effects of the unpredictable traffic events enumerated above. In the final analysis, the number of detectors utilized in a lane depended on the traffic and geometric characteristics, and the type of control that was to be implemented at the associated intersection.

The guidelines used for detector placement within a link were as follows. A typical UTCS detector plan is shown on Figure 2.

- Two detectors on short links and three detectors on long links were used to compute queue length. A distance of 175 feet between detectors and 35 feet between the stop line and the first detector was chosen to achieve good accuracy and good lane coverage. Long links which are also subject to long queues are instrumented with a third detector at a distance of 60-200 feet from the second detector depending on the block length. In the majority of cases in UTCS the distance is 155 feet.

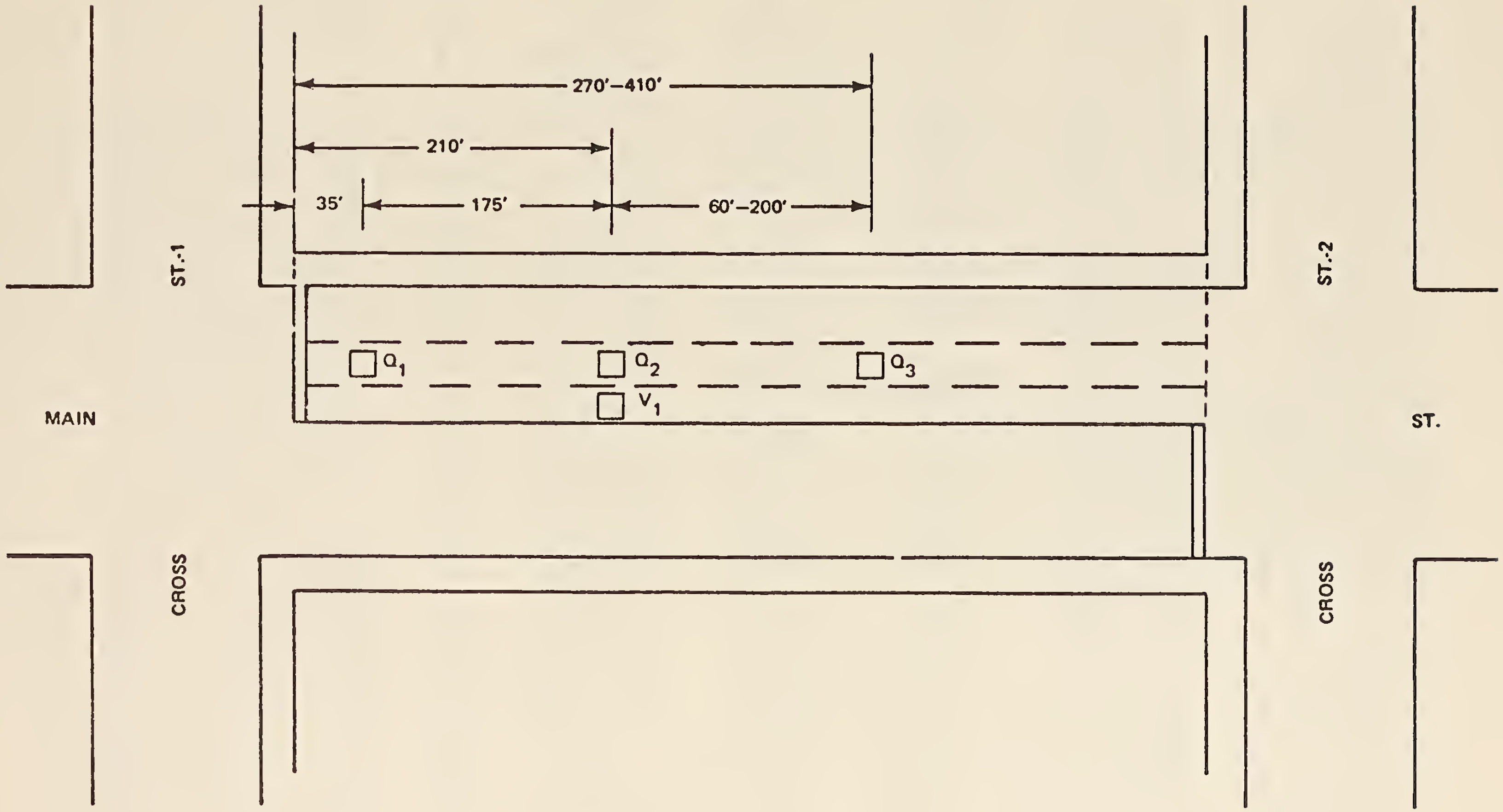

Figure 2. Typical UTCS detector plan. (Source: UTCS/BPS Vol. 1, Design and Installation, Sperry Rand Corp., Nov. 1972).

- Where a lane to be instrumented is adjacent to a lane instrumented for queue measurement, the detector is to be located 210 feet from the stop line, thus siting it next to the second detector in the queue-measured lane. Thus a comparison of speed and occupancy can be readily made.

- Where a link is to be instrumented and no queue-instrumented links are present on that approach, it is desirable to make measurements in an area which is generally representative of free flow or at least average flow conditions, thus implying a mid-block location or a location at some distance from the intersection.

- Queue detectors were not placed in lanes which change direction as a function of time as the problems implicit in the software structure require special consideration.

- Where closely spaced "H" type intersections exist, detectors were placed on the outer legs. This was done to permit a reasonable estimate of queue measurement and to obtain good measurement of overall flow conditions.

- Although parking garages are major sources and sinks, they were not instrumented with detectors in this plan as problems involving detector placement, direction sensing, legal complications, and directional and lane assignment of existing flow required special study.

- The specific positions recommended were modified where physical obstructions, such as manholes, exist.

Using the above criteria, 80 of the original 111 intersections were instrumented. Of the 385 approaches in the test area, 251 (65 percent) were instrumented with at least one detector. There were 48 locations instrumented for critical intersection control with 138 links. In total, 497 loop detectors were used in the detectorization plan. The specific areas of concern for this study were Sections 1 and 3. A graphic portrayal of the existing detectorized links in these Sections is shown on Figures 3 and 4. These detector locations represent the "base case" surveillance system for the research project.

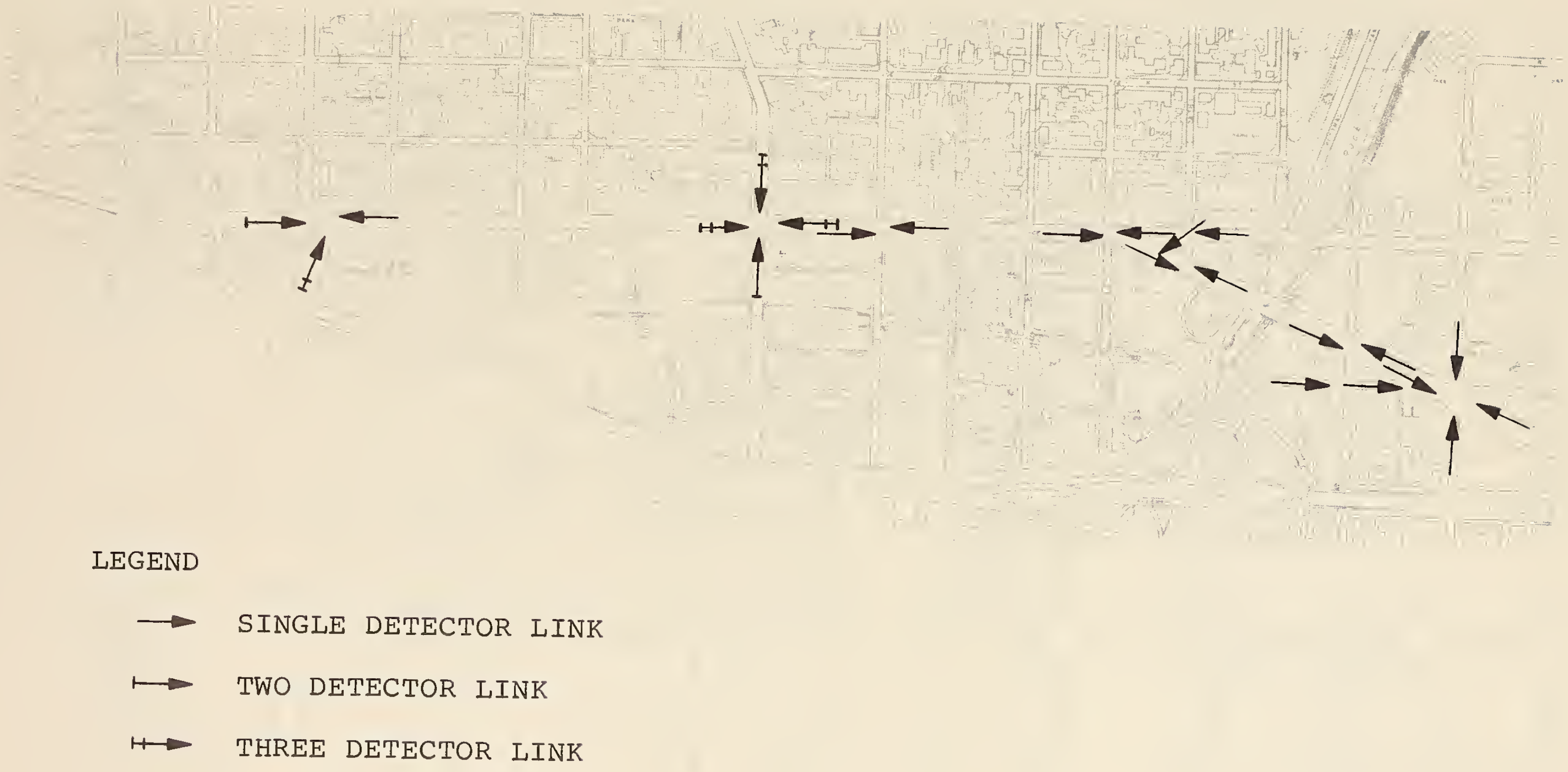

Figure 3. Existing detector locations in Section 1.

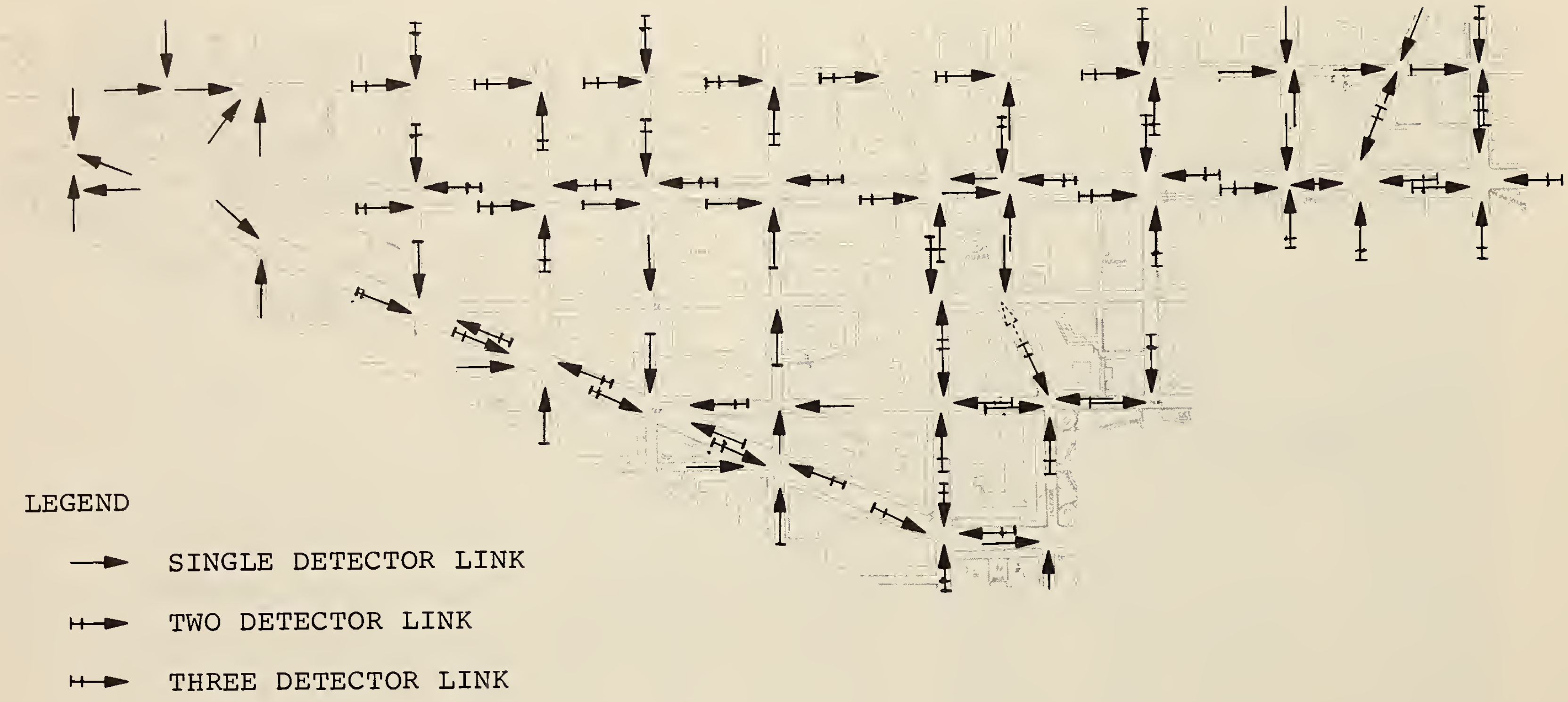

Figure 4. Existing detector locations in Section 3.

COMPUTER PROCESSING

A number of detailed analyses were performed as part of the detector placement study. Some of these analyses consisted of manual manipulation of various data, while others involved computerized procedures. For the most part, the manual data analysis was specifically related to a particular study and is therefore discussed as an adjunct to that study. The computer processing, however, pervades the research and for this reason, is discussed prior to the individual studies.

INPUT DATA

The input data for the computerized studies came from three basic sources. The first source was the raw history or pulse tapes generated by the UTCS system at the control center. The format of the raw data is shown in Table 2, taken from the UTCS Software Manual produced by Sperry Rand Corp.[1]

The raw history tapes consist of one record generated each half-second. Three particular types of data were required for the study. The basic input was the presence indication for each of the detectors in the system. Also used, for control purposes, was the status of the A-phase green return flag and the advance pulse flag. All data consist of bits representing the status of a particular detector or controller, with data for 32 locations in each word.

The second set of data was prepared by JHK from field data collected by observers at several locations in the network. These data were coded according to the format indicated in Table 3. A total of 32 separate data sets covering roughly the period 7:00 AM to 9:00 AM, or about 90 cycles, were coded; these data sets are identified in Table 4. Details of the data collection procedures used are given in Appendix A.

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1. Sperry-Rand Corporation, "Urban Traffic Control and Bus Priority System Software Manual," (PB 220-867/868), Federal Highway Administration, Washington, D. C., February 1973.

Table 2. Raw data format.

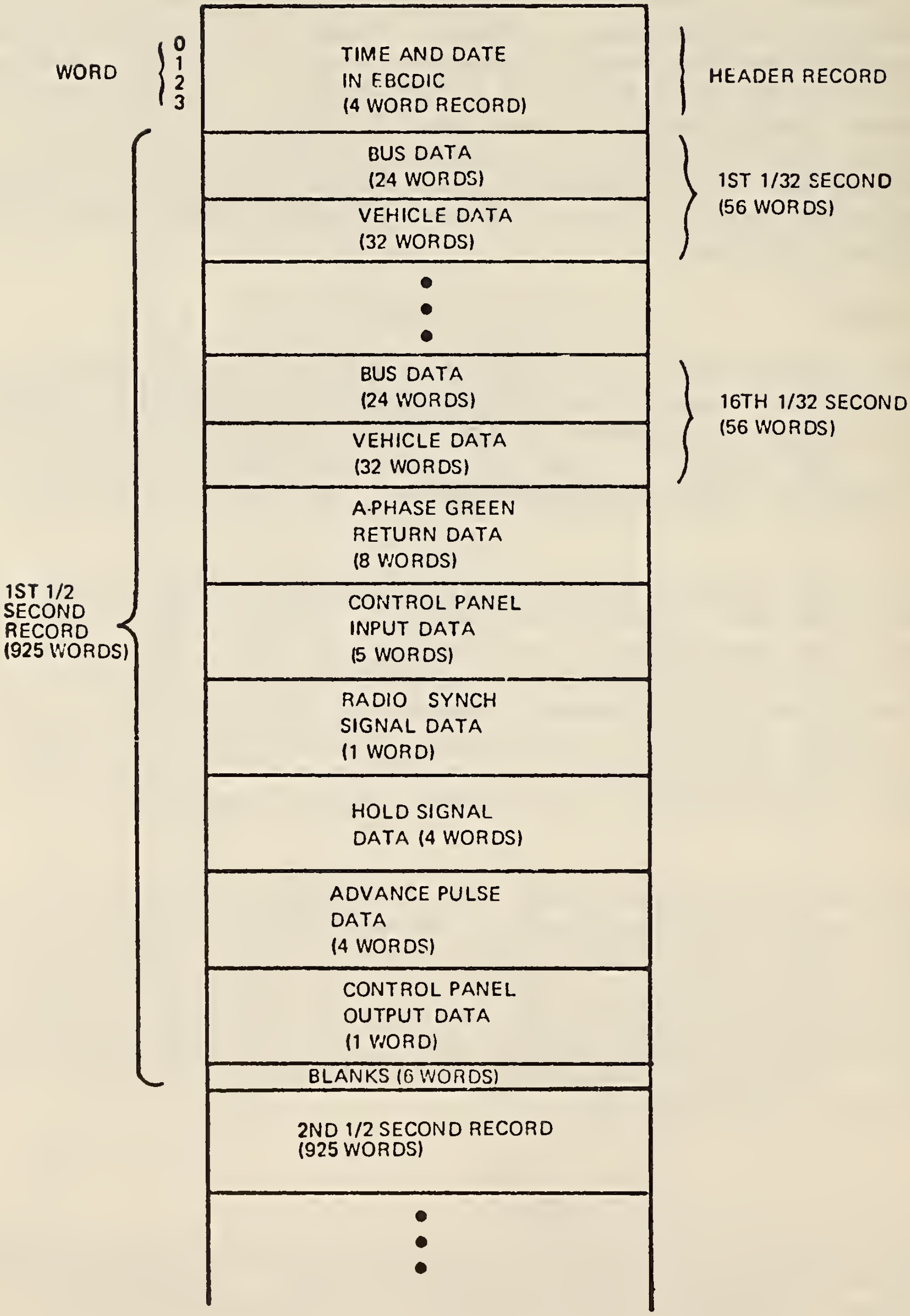

Table 3. Field data format.

Card Column	Identification
1-2	Data set number
4-8	Time check (periodic)
10-12	Cycle number
15-16	Downstream queue - beg. of green
18-19	Downstream queue - 20 sec.
21-22	Downstream queue - yellow
24-25	Upstream queue - beg. of green
27-28	Upstream queue - 20 sec.
30-31	Upstream queue - yellow
33-34	Volume lane 1 - autos
36-37	Volume lane 1 - trucks
39-40	Volume lane 1 - buses
42-43	Volume lane 2 - autos
45-46	Volume lane 2 - trucks
48-49	Volume lane 2 - buses
51-52	Volume lane 3 - autos
54-55	Volume lane 3 - trucks
57-58	Volume lane 3 - buses
60-61	Volume lane 4 - autos
63-64	Volume lane 4 - trucks
66-67	Volume lane 4 - buses

Table 4. Data set identification.

01	L St. between 19th St. & 20th St. EB	4/14/75
02	L St. between 19th St. & 20th St. EB	4/15/75
03	L St. between 19th St. & 20th St. EB	4/16/75
04	L St. between 19th St. & 20th St. EB	4/17/75
05	L St. between 19th St. & 20th St. EB	4/18/75
06	Penna. Ave. between 17th St. & 18th St. WB	4/14/75
07	Penna. Ave. between 17th St. & 18th St. WB	4/15/75
08	Penna. Ave. between 17th St. & 18th St. WB	4/16/75
09	Penna. Ave. between 17th St. & 18th St. EB	4/17/75
10	Penna. Ave. between 17th St. & 18th St. EB	4/18/75
11	Penna. Ave. between 17th St. & 18th St. WB	4/17/75
12	Penna. Ave. between 17th St. & 18th St. WB	4/18/75
13	20th St. between I St. & K St. NB	4/16/75
14	20th St. between I St. & K St. NB	4/17/75
15	20th St. between I St. & K St. NB	4/18/75
16	16th St. between L St. & K St. SB	4/14/75
17	16th St. between L St. & K St. SB	4/16/75
18	21st St. between K St. & Penna. Ave. SB	4/14/75
19	21st St. between K St. & Penna. Ave. SB	4/15/75
20	21st St. between K St. & Penna. Ave. SB	4/16/75
21	H St. at 16th St. EB	4/17/75
22	H St. at 16th St. EB	4/18/75
23	K St. between 18th St. & 17th St. EB	4/14/75
24	K St. between 18th St. & 17th St. WB	4/14/75
25	K St. between 18th St. & 17th St. WB	4/15/75
26	K St. between 17th St. & 18th St. EB	4/15/75
27	K St. between 17th St. & 18th St. EB	4/16/75
28	K St. between 17th St. & 18th St. WB	4/16/75
29	K St. between 17th St. & 18th St. EB	4/17/75
30	K St. frontage between 17th St. & 18th St. EB	4/17/75
31	K St. frontage between 17th St. & 18th St. EB	4/18/75
32	K St. between 17th St. & 18th St. EB	4/18/75

Finally, certain of the analyses required data across all of the detectorized links in the network. The most suitable available data were determined to be those derived from the 15 minute summary reports during the Phase I evaluation. These data had previously been extracted from the summary reports for individual days, broken down by measure of effectiveness (MOE), and grouped for several days during which the same control pattern was in use. For this study, data on MOE 2 (volume) and MOE 3 (occupancy) were used, together with header records giving the date and time, and a link usability vector (LUSE) flagging unusable links.

PROGRAM DEVELOPMENT

A total of seven major types of analyses were performed or assisted through computerized procedures. These analyses included:

1. Statistical analysis of volume and queue/content data from matched observations of detector and field data.
2. Visual displays of the matched volume and queue data.
3. Statistical analysis of volume and speed from the detectors along a link at the three detector locations.
4. Speeds over detectors of vehicles having five and ten second gaps.
5. Statistical variation of field data by cycles and groups of cycles.
6. Patterns in volume and occupancy over all detectors from the 15 minute summary data.
7. Statistical analysis of the relationships between MOE's on various links.

To perform these analyses, a series of special-purpose computer programs were developed. In addition, the statistical post processor developed during Phase I was used extensively.

The overall data processing flow is shown in Figure 5. The programs indicated in the boxes (except POST) were developed as part of this effort and described in detail in

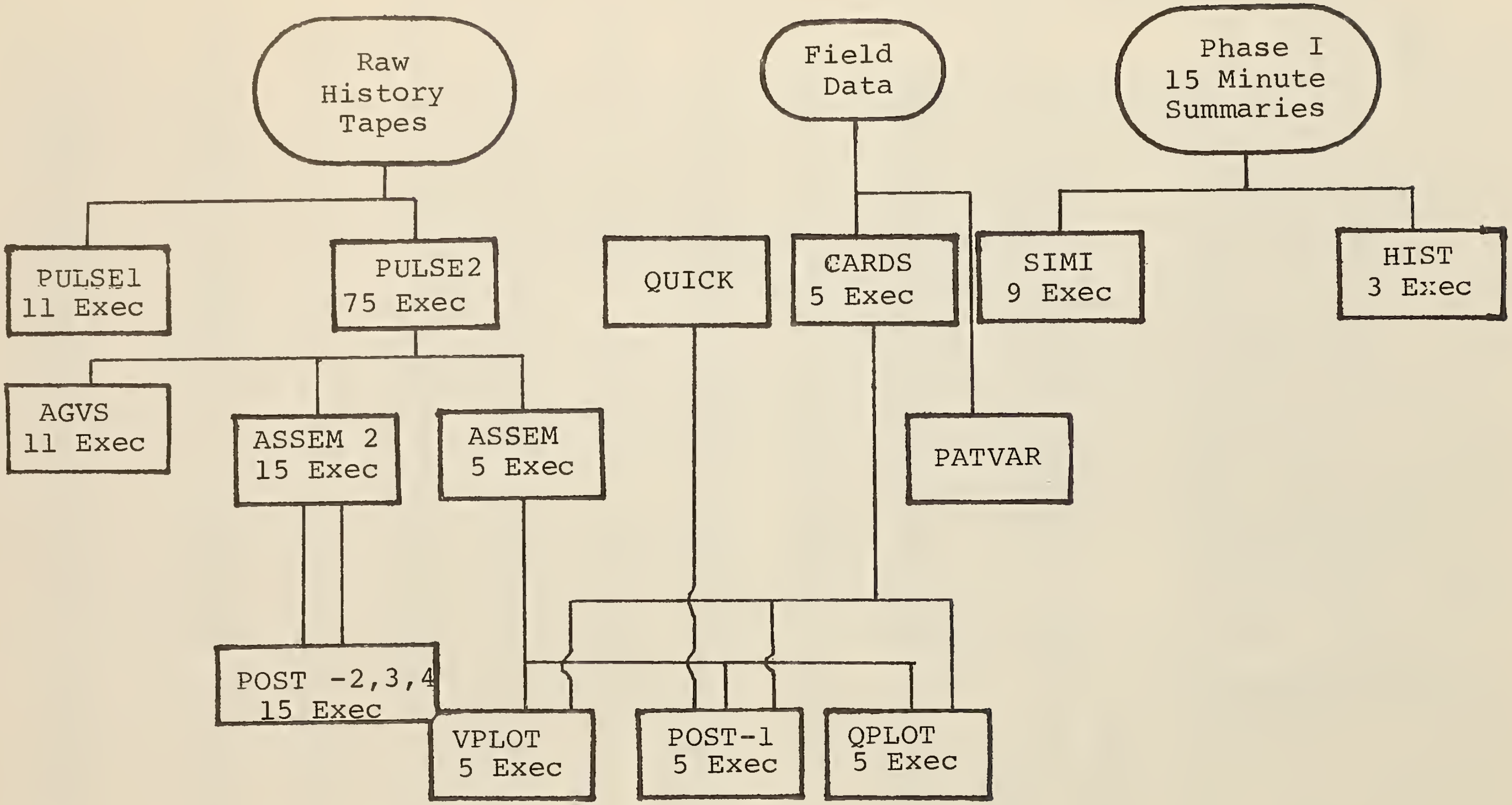

Figure 5. Data processing flow.

a User's Guide included as Appendix B. As can be seen from Figure 5, most of these programs were executed a large number of times, reflecting the considerable volume of data that were processed during the study.

PROGRAM DESCRIPTION

Data were collected for the week of April 14 through April 18, 1975. At this time, coordinated efforts were maintained at the UTCS control center and in the field. Field data were collected at several locations each day, for a total of 32 data sets as previously outlined in Table 3. Three data tapes were collected with the raw history data, each for a period of approximately 30 minutes; these are outlined in Table 5.

The raw history tapes are recorded by the XDS computer in seven track, 556 BPI form. For processing efficiency, the tapes were converted to nine track, 1600 BPI on the FHWA/DOT computer facility in the Nassif Building. Unfortunately, five of the 15 tapes could not be copied, as the system encountered permanent input/output errors. After considerable effort, the problem was diagnosed as extremely poor inter-record gap tolerances on the tapes and they were eventually converted in an arduous manner by a service bureau specializing in such activities.

The first production work in data development was the running of the MOE extraction and summarization program (PULSE 2) for each link desired on each appropriate day. This effort resulted in data sets by cycle containing detector and link summaries of volume, speed, occupancy, and queues, the latter developed by simple count-in/count-out methods. Generally, this effort resulting in 22 cycles worth of data from each tape for each location, although occasionally 21 or 23 cycles were extracted, depending upon the amount of data on the tape and the timing of the first complete A-phase green. One of the 5 tapes with initial I/O errors could still not be fully read and only 11 or 12 cycles of data were extracted from it.

Data from a particular location was merged into a single data set for each day. Controller 60 at 18th and K Streets was found to have failed during April 16 through the 18th, so data for K Street for those days was not usable. In addition, only partial field data were available for Pennsylvania Avenue on April 15 due to inclement weather, so this data set was dropped. Thus, a total of 25 (out of 32) data sets were used for matching and comparing detector and field information.

Table 5. Detector data tapes.

Session	Raw Tape	Copy	Time	Note
Apr. 14A	PM0320*	PM0274,1	7:09:00-7:39-55*	RC≈2986*
Apr. 14B	PM0270	PM0326,2	7:45:00-8:15-40	
Apr. 14C	PM0321*	PM0276,2	8:20:00-8:50:40*	RC≈2959*
Apr. 15A	PM0272	PM0280,1	7:10:00-7:40:50	
Apr. 15B	PM0278	PM0281,1	7:46:00-8:16:40	
Apr. 15C	PM0327*	PM0281,2	8:22:00-8:51:20*	
Apr. 16A	PM0263	PM0311,1	7:03:05-7:35:08	
Apr. 16B	PM0264	PM0311,2	7:38:00-8:08:01	
Apr. 16C	PM0267	PM0319.2	8:13:00-8:43:40	
Apr. 17A	PM0314	PM0318,1	7:05:00-7:35:10	
Apr. 17B	PM0315	PM0319,1	7:40:00-8:10:30	
Apr. 17C	PM0317	PM0318,2	8:16:00-8:46:15	
Apr. 18A	PM0329*	PM0325,1	7:03:00-7:33:00*	I/0 Error RC≈2018
Apr. 18B	PM0330*	PM0326,1	7:37:10-8:08:30*	RC≈2884
Apr. 18C	PM0324	PM0280,2	8:13:00-8:43:00	

* Recopied data, end time not complete, record count on first volume.

PM0276,1 has bad copy of Apr. 14B.

Raw tapes are XDS 7-TRR, 556; copies are IBM SL 9-track, 1600.

The field and detector data were matched by cycle using the recorded times of data collection at the control center and time checks recorded periodically on the field data collection forms. In most cases, the time comparisons were quite easy, with cycles matching within five seconds or less. In a few cases, some estimation of cycles was required, using volume pattern matching. This particularly occurred in cases where an unusual cycle length was observed, making it difficult to interpolate between the time checks as the cycle intervals were then not uniformly 80 seconds. A listing of the final matchings is given in Table 6.

The development of these data sets resulted in the multiple executions of each of the programs noted in Figure 5: Specifically, these programs performed the following functions:

- PULSE2 - Conversion of the raw history tapes to detector and link measures of effectiveness; executed 75 times in total for the 25 data sets for comparisons and the three input tapes for each.
- ASSEM - Selection of pulse data for paired comparisons with field data; executed five times, one for each day.
- CARDS - Conversion of field data into MOE's for paired comparison with detector data; executed five times, one for each day.
- VPLOT - Plot detector vs field volumes from summary data sets; executed five times, one for each day, for a total of 120 graphs.
- QPLOT - Plot detector vs field queue/content values from summary data sets; executed five times, one for each day, for a total of 225 graphs.
- QUICK - Creation of dummy link use (LUSE) and observation use (OBUSE) vectors for post processor control.
- ASSEM2 - Assembly of detector data for longitudinal analysis along individual lanes; executed 15 times, three for each day, for critical lane; lane 1; and lane 3 plus L Street lane 4.

Table 6. Detector/field matches.

Detector Data					Field Data			
			Cycles[1]				Cycles	
Date	Control Box	A	B	C	Card Date Set	A	B	C
14	84	22/22	21/21	22/22	1	29-50	56-76	82-103
	54	22/22	21/22	22/22	6	24-45	51-71	77-98
	88	22/22	21/22	22/22	16	27-48	54-74	81-102
	70	22/22	21/22	22/22	18	30-51	57-77	84-105
	61	22/22	21/22	22/22	23	31-52	58-78	84-105
	60	22/22	21/22	22/22	24	24-45	51-71	78-99
15	84	22/22	22/22	21/22	2	30-51	57-78	84-104
	70	22/22	22/22	21/22	19	29-50	56-77	83-103
	60	22/22	22/22	21/21	25	27-48	54-75	81-101
	61	22/23	22/22	21/21	26	29-50	56-77	83-103
16	84	21/21	21/22	22/22	3	27-47	53-73	79-100
	54	21/22	21/22	22/22	8	21-41	47-67	74-95
	73	21/22	21/21	22/22	13	9-29	36-56	62-83
	88	21/21	21/21	22/22	17	24-44	50-70	76-97
	70	21/21	21/21	22/22	20	15-35	41-61	67-88
17	84	21/21	22/22	21/22	4	27-47	53-74	80-100
	55	21/21	22/22	21/22	9	22-42	48-69	75-95
	54	21/21	22/22	21/22	11	21-41	46-67	73-93
	73	21/22	22/22	21/21	14	27-47	54-75	81-101
	94	21/21	22/22	21/22	21	11-31	37-58	64-84
18	84	11/12	21/21	21/21	5	25-35	51-71	78-98
	55	11/12	21/21	21/22	10	18-28	44-64	70-90
	54	11/12	21/21	21/22	12	16-26	43-63	69-89
	73	11/11	21/22	21/22	15	20-30	45-65	72-92
	94	11/12	21/21	21/21	22	11-21	37-57	64-84

[1] First value is number of cycles for match, second value is total number of cycles for which data were extracted.

- PULSE1 - Summarization of raw history volumes and speeds for vehicles with five and ten second gaps; executed 11 times for illustrative purposes, once for each of the nine data collection locations early in the period and for two locations late in the period.
- AGVS - Supplement PULSE1 with 4-cycle and 11-cycle summaries of speeds and volumes of all vehicles; executed 11 times.
- PATVAR - Computation of statistics from field data on cycle by cycle and 4-cycle aggregate basis; report all field data with 11-cycle averages.
- HIST - Format volume and occupancy from Phase 1 15-minute summary data sets; executed three times for AM, PM, and midday periods.
- SIMI - Computation of statistics and regression parameters for specific pairs of links, using Phase I volume and occupancy data; executed nine times for three sets of comparisons and AM, PM, and midday conditions.
- POST-1 - Execution of statistical post processor for detector vs field comparisons of volume and queue/content.
- POST-2,3,4 - Execution of statistical post processor for longitudinal analysis of volume and speed along specific links from the detector data.

The executions of the post processor noted above were achieved by introducing data sets in a form that permitted the required comparisons. For the POST-1 runs, this effort was straight-forward, although the volume and queue data were mixed in the same data set rather than creating multiple MOE files; this was required since the first is a lane measure and the latter a link measure, rather than merely different types of measurements defined for the same sampling frame. Equally, the effort could have been accomplished using separate executions for volume and queue and either separate data sets or special LUSE vectors.

For the longitudinal analyses in POST-2, 3, and 4 repeated data entries were created in the files so that the various pairs of tests could be achieved. Both data sets were created in the same program (ASSEM2). For all of the POST runs, the subnetwork option was used rather than links, since the former permits labeling to identify the non-homogeneous measures being examined. Finally, dummy link usage and observation usage vectors were created, as noted under program QUICK above.

The post processor itself was not modified and performs the following calculations and statistical tests:

1. The mean and standard deviation for the MOE under consideration over the appropriate sample is calculated for the two cases under consideration. The signed percentage difference of these two means is calculated.

2. A two-tailed t test is performed to test the null hypothesis that the mean values of the MOE for the two cases were not significantly different.

3. A two-tailed Mann-Whitney U test is performed to test the same null hypothesis.

4. A Kolmogorov-Smirnov two-sample test is performed. It is intended to be a one-tailed test of the null hypothesis that either the first case's distribution of MOE values is not significantly "greater" than that of the second or vice versa, whichever tail is appropriate in light of the data. However, if a level of significance in rejecting the null hypothesis in both directions is achieved, the significance test defaults to the two-tailed type. In any case, all test statistics are calculated and made available in the printed output.

t Test

A two-tailed t test was performed to see if the null hypothesis that the mean MOF value for the two cases are equal can be rejected. The t statistic is calculated, assuming that the standard deviation of the two populations involved is not in general equal:

$$t = \frac{|\bar{A} - \bar{B}|}{\sqrt{\dfrac{\sum_{I=1}^{n} [(A(I) - \bar{A})^2 + (B(I) - \bar{B})^2]}{N*(N-1)}}}$$

where N is the common sample size for the two cases (the number of actual elements in vectors A or B). The degrees of freedom used was 2*N-2.

The three significance levels for this test were 95 percent, 98 percent, and 99 percent. Critical significance values for $df > 25$ were estimated using linear interpolation of table values, with a lower bound of the critical values for $df = \infty$.

Mann-Whitney U Test

The nonparametric two-tailed Mann-Whitney U test was performed to determine if the two distributions of MOE values (A and B) are significantly different. The test statistic is based on a sum of ranks from a pooling of A and B values, where tied values are all given a rank equal to the average of all the adjacent integral ranks that these numbers would have assumed if they were all slightly different from each other.

The three significance levels are 95 percent, 98 percent, and 99 percent. For sample sizes exceeding 20, a test approximation based on the normal distribution is used instead of the exact critical table values.

Kolmogorov-Smirnov Test

The third test executed was the two-sample Kolmogorov-Smirnov test. This is a nonparametric statistical test that is well adapted to testing for general differences between distributions. The one-tailed test statistics are based on the maximum and minimum signed vertical differences between the cumulative MOE distributions for the two cases, respectively. The two-tailed statistic is based on the maximum absolute vertical difference between the two distributions.

For sample sizes of 40 or less, the critical significance level values used by the subroutine were the appropriate values taken from test tables. When the sample size exceeded 40, the significance tests had to use an asymptotic approximation formula (based on $\sqrt{2*N}$)

The one-tailed tests was assumed unless both one-tailed tests were significant, in which case the two-tailed, undirectional test was used. For the one-tailed tests, the three significance levels were 95 percent, 97.5 percent, and 99 percent. For the two-tailed tests, the significance levels were 95 percent, 98 percent, and 99 percent.

Test Results

The significance level achieved in rejecting the null hypothesis in a statistical test is graphically presented through the printing of asterisks following the test statistic value(s). If no significance was achieved, no asterisks are printed. If the lowest level of significance is achieved (95 percent), one asterisk is printed. Two asterisks indicate the second significance level (97.5 percent or 98 percent) and three asterisks, the highest significance level (99 percent). If the test could not be made for any reason, an "NT" is printed instead.

Just before the significance level indicator for the Kolmogorov-Smirnov in each row of a table is printed, the test's tail direction indicator appears. A "<" indicates that case 1 (condition A) had a distribution of MOE values that was significantly lower than that of case 2 (condition B). A ">" indicates the reverse. The absence of either indicates:

(a) no significance was achieved if no *'s follow this blank, or

(b) a two-tailed significance test was used if one or more *'s do follow the blank.

Many of the data sets produced were nearly unique in the combinations of links and variables that existed on each. The parameters describing the most relevant data sets are recorded in the program User's Guide in the Appendix B. Particular emphasis is given in the discussions of the following programs:

- PULSE2
- CARDS
- ASSEM2

All data sets were created and stored on disk at the FHWA/DOT computer facility in the Nassif Building. A list of the data set naming conventions is given in Table 7. A summary of locations and mnemonics is given in Table 8.

Table 7. Data set naming conventions.

Program	Data	Output Data Set Name(s)
IEBGENER	Raw History	PMMCO.UTCS.HTAPE.APRddt
PULSE2	Detector MOE's	PMMCO.UTCS.APRdd.CBnn
ASSEM	Detector Matches	PMMCO.UTCS.PULSE.SUM.APRdd
CARDS	Field Matches	PMMCO.UTCS.CARDS.SUM.APRdd
QUICK	Observation Usage	PMMCO.UTCS.PULSE.OBUSE
	Link Usage	PMMCO.UTCS.PULSE.LUSE
ASSEM2	Condition "A"	PMMCO.UTCS.SUMa.APRdd
	Condition "B"	PMMCO.UTCS.SUMb.APRdd
IEBGENER	Field Data	PMMCO.UTCS.CARDS.ALL

Nomenclature:

dd - Date - 14, 15, 16, 17, or 18.

nn - Controller (CB) Number - 84, 54, 88, 70, 61, 60, 73, 55, 94.

a - Data Set Number - 2, 4, 6.

b - Data Set Number - 3, 4, 7.

Table 8. Data collection locations and mnemonics.

Controller Number	Mnenomic	Location
84	L	Eastbound L Street @ 19th Street
54	PW	Westbound Penna. Ave. @ 18th Street
88	16	Southbound 16th Street @ K Street
70	21	Southbound 21st Street @ Penna. Ave.
61	KE	Eastbound K Street @ 17th Street
60	KW	Westbound K Street @ 18th Street
73	20	Northbound 20th Street @ K Street
55	PE	Eastbound Penna. Ave. @ 17th Street
94	H	Eastbound H Street @ 16th Street

REQUIREMENTS OF UTCS CONTROL STRATEGIES

INTRODUCTION

Before detector placement configurations can be designed, a definition for each of the surveillance input variables used in each of the three generations of UTCS traffic control algorithms is required. Two basic approaches were used by the research team to develop these definitions--literature review and direct contact with FHWA staff and the developers of the algorithms. First, documentation for each generation of software was studied. This effort provided the foundation for the definitions. Several questions were generated, however, concerning the precise use of the input variables by the algorithms. In these instances, the developers of the second and third generation algorithms were contacted.

As a result of the review of the literature and of discussions with the second and third generation software contractors, it was determined that most of the variables had two definitions. The first definition can be considered an abstract or idealized definition. The second definition is the definition of the input variable within the context of the UTCS hardware and software system. For example, the first generation traffic-responsive mode uses "existing traffic demand" as the basic input variable. "Existing traffic demand," however, is an abstract value and cannot be measured directly. As an estimate of this idealized value, a criterion composed of volume plus weighted occupancy is utilized. Similar variable definition dichotomies were identified in definition of second and third generation variables. The following paragraphs discuss the specific data requirements for each of the three generations of UTCS software.

FIRST GENERATION, TRAFFIC RESPONSIVE

Using the traffic-responsive algorithm, the computer selects the "best" available pattern for each section as a function of the latest smoothed traffic data, but is constrained to a maximum of one change every 15 minutes. The times for these changes, if they are required, are on each quarter hour. The pattern selection is made by matching the current demand with the stored demand characteristics of the available timing patterns and selecting the pattern which has the closest fit. Because demand is an abstract parameter, the actual comparison criteria used in "volume plus weighted occupancy" which is assumed to yield a realistic indication of existing traffic demand. With this algorithm, three variables are defined: demand, volume, and occupancy.

- Demand - Demand is defined as the number of vehicles that desire to use the link during the next 15-minute period.
- Volume - Volume is defined as a lane-specific variable and measured in vehicles per hour on the link. This parameter is estimated by summing the number of vehicles counted by each detector in the lane-specific link for 15 minutes, then dividing by the number of detectors in the lane, and then converting to an hourly figure.
- Occupancy - Occupancy is defined as a measure based on the percentage of vehicle presence time at a fixed point for a fixed period of time. It should be noted that for a given flow rate of vehicles moving at a given velocity, occupancy will vary as a function of the average vehicle length and, unless factored, the length of detectorized zone. Occupancy as used by the first generation algorithm is the sum of the measured presence time in seconds of each vehicle over the detector minus the detector bias in seconds. This value is then averaged by the number of detectors on a lane-specific link and multiplied by a correction factor to compensate for finite loop lengths. The value of occupancy used in the traffic-responsive algorithm is the average percentage occupancy for a 15-minute period.

FIRST GENERATION, CIC (VOLUME)

The purpose of this algorithm is to aportion the split of the critical phases of an instrumented critical intersection according to the measured vehicle demand on these phases. The green demand is computed from a first-degree polynominal and is equal to a constant times occupancy, plus a second constant times a vehicle count, plus a third constant times the product of occupancy and count. If a controller phase has two critical links assigned to it, the green demand will be set equal to the larger of the two demand times. The algorithm then allocates the available green time between competing phases in proportion to the calculated demand on each phase providing that minimum green times are not violated. The two basic input variables for this algorithm are demand and occupancy.

a. Demand - Demand is a critical lane variable defined as the sum of the vehicles entering the block during a cycle plus the vehicles remaining in the block from the preceeding cycle. The actual values used by the algorithm are the average of the sum of the vehicle counts of each detector, in the instrumented critical lane. These quantities are summed on a cycle-by-cycle basis.

b. Occupancy - Occupancy is also a critical lane variable computed as defined for the traffic responsive mode; however, instead of using an average of the three detectors, occupancy is only calculated for the Q2 detector which is located approximately 210 feet upstream from the stop line. Occupancy is calculated on a cycle-by-cycle basis.

FIRST GENERATION, CIC (QUEUE LENGTH)

A second CIC algorithm has been developed for use in the UTCS network. This algorithm allocates split at critical intersections by proportioning the time in the ratio of the smoothed values of the detected queues in the competing critical lanes. Queue is identified as follows:

Queue - The basic input variable for this program is queue. A queue is defined as the sum of all vehicles, both moving and stopped, within all of the zones of a lane-specific link at the instant the traffic signal controlling link turns green. The queue is calculated by counting in from the upstream detector and counting out from the downstream detector for each zone on the instrumented link. To correct for cumulative errors, the queue is assumed to be zero at the end of each phase green.

SECOND GENERATION, LSTSQS

The basic second generation control package developed by TRW Systems for the UTCS system is called the Traffic Adaptive Network Signal Timing Program (TANSTP). The second generation software consists of six major components as follows:

- A traffic prediction model called PREDICT, to predict traffic volumes and speeds in the network for the ensuing 5-minute period.

- A subnetwork configuration model called RTSND that subdivides the network into groups of signals based on similar cycle length requirements. This subroutine also computes the basic intersection splits by assigning green time proportional to the approach volumes of the critical approach.

- An optimization routine called LSTSQS that may generate optimum offsets for each subnetwork for a 5-minute period.

- A critical intersection control routine called LOCAL computes the optimal split at each major intersection based on current traffic conditions. Both A and B phase offsets are computed to attempt to maintain progression on both arrival and departure links subject to cycle lengths and phase duration constraints.

- An offset transition model called TRNPAR that minimizes the sum total of all intersection offset changes when a new signal pattern is implemented.

- A boundary interface model called INTFC that attempts to minimize vehicle delay on connecting links between two subnetworks.

The critical lane variables used by LSTSQS are volume and speed. In the abstract, LSTSQS utilizes the total link volume for each 5-minute period and the travel speed of free-flowing vehicles on the link. This latter value is converted into an ideal offset difference by dividing the link length by the free flow link speed.

The actual values used by LSTSQS are produced by the subroutine PREDICT. The purpose of this subroutine is to forecast link volumes and speeds for the 5-minute period ahead of the current time. The subroutine is executed every 5 minutes.

An initial goal of the second generation system was to use the identical executive structure, surveillance data, and the command structure of the first generation package. Because the first generation software, however, computes speed based on the average of the critical lane detectors, special software has been developed by TRW to extract speeds and volumes from the upstream detector. The precise definitions of the critical lane variables now being used by

PREDICT are as follows:

- Volume - This parameter is essentially identical to the parameter used in the traffic-responsive mode of the first generation software. It is a critical lane count on a 5-minute basis.

- Speed - Speed is estimated by using a single loop critical lane speed detector that is located 210 feet upstream from the signalized intersection. This detector is used to estimate the free flow speed based on the time mean speed measured by the detector.

SECOND GENERATION, CRITICAL INTERSECTION CONTROL

The second generation critical intersection algorithm, LOCAL, is basically a routine that fine tunes, based on current data, the signal patterns developed by the network optimization program. LSTSQS. This algorithm uses the link travel time (calculated from link lengths and speeds), link volumes, and queue lengths based on current cycle-by-cycle surveillance data. The green time required for each link is then computed as a function of link discharge rates and queue length. The critical lane variables used by this algorithm are link speeds, link volumes, and queue lengths.

- Volume - The volume used by this algorithm is a critical lane count on a cycle-by-cycle basis.

- Speed - The speed used is a time mean speed as defined above computed on a cycle-by-cycle basis.

- Queue - The basic input variable for this program is queue. A queue is defined as the sum of all vehicles, both moving and stopped, within all of the zones of a lane-specific link at the instant the traffic signal controlling link turns green. The queue is calculated by counting in from the upstream detector and counting out from the downstream detector for each zone on the instrumented link. To correct for cumulative errors, the queue is assumed to be zero at the end of each phase green.

THIRD GENERATION, CYRANO

The third generation control strategy uses dynamic traffic-responsive control algorithms to provide optimum signal timings on a cycle-by-cycle basis. The cycle length, split, and offset may vary between intersections and from cycle to cycle, while overall network optimization is maintained. Since the network is in a constant state of transition, the need for an offset transition routine is eliminated. CYRANO uses variable cycle length for a control period of approximately four minutes. The critical lane variables currently required by CYRANO are primary and secondary volume. These parameters are defined as follows:

- Primary Volume - In the abstract, this parameter is the major flow to be serviced from the upstream intersection at the downstream intersection.
- Secondary Volume - Secondary volume is the minor flow to be serviced at the downstream intersection from both the upstream intersection and the link itself. Secondary volume is simply the total volume at the downstream intersection less the primary volume.

Initially, however, primary volume will be estimated by expanding the critical lane volume count by a percentage factor. To be conservative, the link volume is presently generated by multiplying the critical lane volume by the number of lanes. Using the expected measured volume, the software then estimates turning movements using a history of turning movement percentages on a time-of-day basis. The turning movement percentages are available for three traffic conditions; a.m. peak, average, and p.m. peak. The estimated turning movements are then used to distinguish between primary and secondary volume.

If a link is not instrumented, a volume history is used by CYRANO for that link. The factor is calibrated once for each time of day and not adjusted. This operation is transparent to CYRANO, since it just reads volumes as input. The fact that it is actually history is only known by the housekeeping software.

THIRD GENERATION, CIC/QMC

The basic objective of CIC/QMC is to maximize throughput. To accomplish this objective, CIC/QMC attempts to manage congestion and to restrict the spread of the congestion to the path which is under CIC/QMC control. When the congestion begins to ebb, its function is to flush the system as rapidly as possible and return control to CYRANO. The basic critical lane variable for CIC/QMC is link content. This ideal parameter is defined as the number of vehicles in the link at any given instant. The algorithm actually uses the link content value at three specific times during the local cycles of those links that lie along the path of congestion. The first value is when the downstream intersection changes from red to green. The second value is when the downstream intersection changes from green to red. The third value is when the downstream green is approximately half over. The object of this third estimate is to assure that there is no blockage which would cause the link storage capacity to be exceeded. Because of the cycle-free nature of CYRANO, the time interval between evaluations of link content is variable. Initially, the link content will be further constrained by defining it as a zone count of those vehicles between the stop line and the upstream detector. If the data is not available, the program goes into blackout which ignores that link as far as congestion control is concerned. That link, then, remains in the CYRANO mode.

GOALS FOR ACCURACY OF VARIABLES

As an adjunct to the development of the variable definitions, the suggested range of accuracy for each variable given in the statement of work was reviewed. In estimating link-specific volumes, it is known that three components of error combine to limit the accuracy potential of each of the control variables. These components are: 1) a "measurement error" in the data on which the predictor operates; 2) a "prediction error" in estimating the underlying mean, and; 3) a component reflecting the randomness of traffic.

Data error can be expressed as an absolute quantity or as a precentage of the expected value. The percentage method is considered more revealing. A precise statement of the accuracy could be "X percent probability that the error would be within Y percent." Such a statement may be impossible if the probability distribution of the error is unknown. However, a normal distribution is generally a good approximation for the mean value of a large sample.

The following was the range of errors initially considered for each of the control algorithms surveillance requirements that were evaluated.

First Generation

Traffic Responsive

Volume - Volume is a 15-minute count measured on the quarter hour to within an accuracy of plus or minus 10% for 90% of the time.

Occupancy - Occupancy is the sum of the measured pulse lengths (corrected for loop length and detector bias) for a 15-minute period expressed as a percentage. The range of error is plus or minus 10% for 90% of the time.

Critical Intersection Control

Volume - Volume is a critical lane count measured each cycle to an accuracy of plus or minus three vehicles 90% of the time.

Queue - Queue is the sum of all vehicles, both moving and stopped, within all zones of a critical lane counted each cycle at the beginning of the green interval to within an accuracy of plus or minus two vehicles 90% of the time for two detector links and plus or minus four vehicles 90% of the time for three detector links. (This goal proved unrealistic.)

Occupancy - Occupancy is the sum of the measured pulse lengths (corrected for loop length and detector bias) for a one-cycle period for each critical lane approach expressed as a percentage. The range of error is plus or minus 10% for 90% of the time.

Second Generation

LSTSQS

Volume - Volume is a 15-minute count measured on the quarter hour to within an accuracy of plus or minus 10% for 90% of the time.

Speed - Speed is measured by averaging the pulse lengths generated by the number of vehicles actuating the detector during a 15-minute period measured on the quarter hour to within an accuracy of plus of minus 10% for 90% of the time.

Critical Intersection Control

Volume - This volume in identical to the first-generation critical intersection control volume.

Speed - Speed is measured by averaging the pulse lengths generated by the number of vehicles actuating the detector during a period of one signal cycle to within an accuracy of plus or minus 10% for 90% of the time.

Queue - Queue is the sum of all vehicles, both moving and stopped, within all zones of a critical lane counted each cycle at the beginning of the green interval to within an accuracy of plus or minus two vehicles 90% of the time for two detector links and plus or minus four vehicles 90% of the time for three detector links. (This goal proved unrealistic.)

Third Generation

CYRANO

Primary Volume - Primary volume is a critical lane count measured each cycle to an accuracy of plus or minus three vehicles or 10% (whichever is greater) 90% of the time.

Secondary Volume - Secondary volume is the difference between total volume and primary volume measured in the critical lane on a cycle by cycle basis simlar to primary volume.

Speed - Speed is measured by averaging the pulse lengths generated by the number of vehicles actuating the detector during a period of one signal cycle to within an accuracy of plus or minus 10% for 90% of the time.

CIC/QMC

Link Content - Link content is the total number of vehicles, both moving and stopped, within the critical lane link at three instances during the cycle: when the green is first displayed; when the variable green period of 50% completed; and when the red interval is first displayed. For each of these three intervals, link content is measured to an accuracy of plus or minus 20%, 90% of the time when the content is greater than 50% of the capacity of the link and plus or minus two vehicles 90% of the time when the content is equal to or less than 50% of capacity.

LINKS REQUIRING DETECTORIZATION

The basic issue addressed in this research was to determine the optimum number and location of vehicular detectors to provide the traffic surveillance information required to implement each of the three generations of traffic control algorithms used in UTCS. In order to structure the research, the problem was stratified into two components. The first component was to identify which of the 42 signal controlled approaches in Section 1, and 123 signal controlled approaches in Section 3 should be instrumented for each of the three generations of UTCS control algorithms. Once the links requiring detectorization are identified, the second component of the problem becomes the major issue. That is, the number and specific location of the traffic detectors within each link. The second component is addressed in the following chapter while the issues relating to the problem of which links to be detectorized are addressed in this chapter.

The identification of links requiring surveillance was divided into two areas, local control related requirements and system (subsystem) control related requirements. Two types of algorithms exist for local control, actuated phase(s) and critical intersection control (CIC). It is obvious that any actuated phase requires detection and that the detection configuration must be such that there is an extremely low probability of a vehicle approaching the intersection without being detected. However, because there were no semi-actuated intersections within the study area, no work was conducted concerning the appropriate location of local actuation detectors. This type of local control algorithm is mentioned solely for completeness.

There are a number of CIC intersections in the study area and a major effort was made in identifying the appropriate location and number of detectors for the various CIC algorithms. Because all of the CIC algorithms require surveillance on the major conflicting approaches at an intersection, several techniques were developed to identify critical intersections and thus identify an initial set of links that must be detectorized.

Once the CIC intersections and their critical approaches were identified, all other links were analyzed to determine whether they should be detectorized. This analysis placed emphasis on the time element with respect to measuring the input parameters for the system control algorithms. First generation, TRSP, requires input parameters on a 15 minute basis while both second generation, LSTSQS, and third generation, CYRANO, require input parameters within a shorter time frame, approximately five minutes.

LOCAL CONTROL RELATED REQUIREMENTS

As noted, the initial approach to the task of identifying the number and location of detectors for advanced control strategies was to identify those links that required some form of detectorization without regard to the detector location within the link. The review of the UTCS System Software documentation showed that each of the candidate CIC algorithms including the CIC/QMC algorithm used in the third generation software require current real time surveillance data on the approaches to the intersection exhibiting the major demand. The problem of identifying links that require detectorization for local control therefore resolved itself into a problem of identifying which intersections should operate in the CIC mode. Four basic techniques were used in this research to identify criteria for selecting CIC intersections: 1) Existing CIC Criteria 2) Volume/Capacity Measures 3) MOE Data Analysis, and 4) Cycle Failure Measures.

Existing CIC Criteria

Traffic surveys and studies were conducted to identify the type of control required by each specific intersection as part of the initial project and as discussed earlier. The information was extracted from reports that were available and by supplementary surveys and studies. Where no data existed for an intersection, or if extensive changes had occurred at that intersection, a complete re-study of the intersection was required to develop new basic data for the analysis. The data included the intersection physical features such as the roadway type and condition, cross-section information, right-of-way width, number of lanes, and the time of day parking regulations. Additional data collected included descriptions of the operational characteristics of the intersection such as the vehicle flow counts, pedestrian counts, turning movement counts, and classification counts. From these data, determination was made of the peak hour flow, off peak hour flow, and the average daily traffic count.

The collected data was used to select the intersections to be included in the computer controlled system. The survey of traffic patterns and geometric considerations were used to determine the breakdown of the network into the four sections. The traffic patterns were also used to determine the type of control that was implemented in these sections. For example, where the traffic patterns in a given section were not highly predictable on a time-of-day basis, it was considered advantageous to instrument the intersections for traffic responsive operation. These data also provided the foundation from which the decisions were made as to which intersections were to be instrumented for CIC operation.

The fundamental criterion used in selecting CIC intersections was the demand upon competing approaches. Intersections for which CIC operation was considered desirable were those that exhibited a heavy fluctuating demand on the two or three conflicting phases. When this criterion was applied to the UTCS system, two intersections were selected for CIC instrumentation in Section 1, and 28 intersections were selected for CIC instrumentation in Section 3.

Volume/Capacity Measures

A second method to identify critical intersections used the volume-capacity ratio of the major demand on each phase. When these values are summed for all the competing phases at an intersection, an estimate can be made of the level of service at which the intersection is operating. The basic assumption is that critical intersections would tend to operate at lower levels of service during peak traffic conditions than non-critical intersections. Details of the procedures that were used to estimate the capacity of an approach were published in an article in the January 1971 issue of Traffic Engineering, "Intersection Capacity Measurement Through Critical Movement Summations: A Planning Tool", by Henry B. McInerney and Steven G. Petersen. The critical movement technique involves a coarse intersection capacity calculation based on knowing the turning volumes at the intersection under study, the number of lanes on each approach, and any turn restrictions or regulations. This technique was selected because it had a minimum requirement for input data. For the location under consideration, only intersection turning movement data and intersection inventory data were required. The technique has the added advantage that it could be applied to all the intersections in Sections 1 and 3 to obtain a quantitative rating of the demand placed on each intersection.

The technique was exercised on a number of intersections in the study area producing very discouraging results. Basically the technique indicated that these intersections were operating at a very high level of service when in fact it was known that the intersections were operating under force flow conditions at a very poor level of service.

The root cause for the problems experienced with this critical movement summation technique was the overly optimistic estimate of capacity. At least three phenomena that are not accounted for in the capacity estimate combine to significantly reduce the throughput of the intersection; congested downstream (receiving) link, cross-street spillback traffic blocking the intersection, and pedestrians illegally crossing the intersection. Because these phenomena have an unknown and varying impact on the capacity estimate of the intersection throughout the peak hours, it was decided to abandon this method in favor of a more direct measure of the congestion.

It should be noted however that although the technique did not work in either the heavily congested arterial area (Section 1) or the congested grid area (Section 3), it is felt that the technique can be successfully applied in areas where the existing congestion is not as extreme as it is in Washington, D.C. Specifically, it is felt that the technique would yield valid results in those instances where the discharge links from the intersection have sufficient capacity to receive all of the traffic that is directed onto them from either the cross-street or the main street, and in situations where the pedestrian conflicts are not as severe as experienced in the UTCS study area.

MOE Data Analysis

The third technique used to identify potential CIC locations utilized MOE report data that was generated during the evaluation phase of the UTCS research. The actual MOE tapes used were generated between October 24 and November 8, 1974 during the testing of the CIC control alternative.

The 15-minute MOE summary tapes are generated by the System's XDS Sigma 5 computer. In addition to the date and time of the observations, the tape contains information on the detector state for each of the detectorized links in the system, containing the link identification number and the observed value of each of the seven MOE's for the 15-minute period. The tape also contains information concerning the status of each of the system's controllers, control operator actions, plus data on the controller, detector, and communications malfunctions.

For the purposes of the current effort, only two of the system MOE's were used: volume and occupancy. Volume is recorded as a simple count of the number of vehicles passing over a detector within the 15-minute time period. If a link contains more than one detector, the estimate is based on the sum of the observations for each detector divided by the number of detectors. Occupancy reflects the proportion of time that a detector is "on" or covered by a vehicle traversing the link within the 15-minute period. This value is expressed as a percentage. Observations are again averaged, where appropriate, across two or more detectors to produce a single value for each link. The previously described program, HIST, was developed by the research team to format the volume and occupancy from the fifteen minute summary tape, by detector number and date, for each fifteen minute time period, and to produce an average value of volume and occupancy for each detector and time period. The HIST output was manually searched and all detectors that had identical occupancy and volume data for several days were not used since this would indicate that the data came from a historical file rather than from detector actuations.

As an indicator of congestion, an approach was considered congested when the volume rate exceeded 200 vehicles per hour and the occupancy exceeded 20 percent for a 15-minute period in either the a.m. or the p.m. peak hours. The minimum threshold values of 200 vehicles per hour and 20 percent occupancy were arbitrarily selected to avoid implications of congestion when, in fact, low traffic flow conditions actually existed. Next the maximum number of congested 15-minute periods for the two major phases at each intersection was developed. There were 10 15-minute periods available for the a.m. peak condition. Thus, the maximum number of congested-approach periods that a two phase intersection could have would be 38. The maximum number of congested 15-minute periods actually observed was 30 out of the possible 38, at K and 20th Streets. The fewest number of congested periods was 13; this occurred at L and 18th Streets, and at K and 15th Streets at Vermont Avenue. These data were generated for each of the 30 CIC intersections in Sections 1 and 3. Because of detector failures, and links not detectorized because of Metro construction however, complete data were available for only 16 of the 30 intersections. The results of this effort are depicted on Table 9.

The purpose of this task was to attempt to develop a method that would be quantitative in nature and could be used to rank the existing CIC locations in order of congested approaches. The underlying assumption being that intersections exhibiting the most congestion would be the ones that would benefit most from CIC control. Because of the gaps in the data, however, the conclusions were of limited value.

Using the same basic data base of volume and occupancy from the MOE summary tapes, a second technique was developed for the purpose of ranking the existing CIC locations in order of importance. With this second method a set of data was constructed for each intersection, consisting of an

Table 9. CIC congestion and MOE correlation analysis.

	Number of Congested 15 Minute Periods			Competing Approach Correlation Coefficient		
Intersection	AM	PM	Total	AM	Mid	PM
Section 1						
M/Wisc	7	12	19	.78	.27	.09
M/Key Br				.59	.55	.30
Section 2						
Penn/18/H	14	14	28	.82	.08	.59
Penn/17	11	17	28	.84	.27	.68
K/19	13	15	28	.65	.42	.29
* K/18						
* K/Conn/17						
* I/18						
* I/17						
Penn/19/H	12	13	25	.79	.36	.08
* H/17						
Penn/I/20						
Penn/I/21	4	19	23	.70	.39	.37
K/21	11	15	26	.79	.43	.05
K/20	16	14	30	.72	.08	.20
L/21	15	9	24	.86	.05	.44
L/20	12	7	19	.91	.33	.38
L/16	15	14	29	.78	.21	.48
L/19				.81	.08	.13
L/18	3	10	13	.79	.34	.36
* L/Conn						
K/17				.51	.10	.04
K/16	13	15	28	.15	.55	.34
K/15	14	9	23	.37	.14	.54
* I/16						
H/Conn/Jack				.67	.27	.48
H/16				.73	.18	.17
L/14				.17	.20	.51
K/14	13	16	29	.14	.06	.53
K/15/Vt	0	13	13	.57	.26	.21

* Not studied due to impact of Metro construction.

estimate of the demand on opposing phases. The underlying assumption is that if the ratio of demand between the two competing phases remained relatively constant throughout the two peak hours, then little could be gained by instituting a variable split CIC algorithm. If, however, the correlation between the demand on conflicting approaches were extremely low, then it would appear that a great deal of benefit could be achieved by installing a variable split algorithm. The estimate of demand was made by using the average values of volume and occupancy for each of the 15-minute periods by setting demand equal to volume plus twenty times occupancy. This is similar to the estimate of demand used in the algorithms.

A computer program, SIMI, was developed to compute the necessary regression parameters for each pair of detectorized links. As an input to this program, conflicting pairs of links were selected to be tested. For each test, the program used between 62 and 96 pairs of data for each of the three time periods. The program then calculated the equation for the linear curve fit of these data, the correlation coefficient, and the standard error of estimate. The correlation coefficient was used as the figure of merit in the analysis. With this test a high correlation coefficient would indicate that traffic demand on the two competing links tend to fluctuate in a similar manner, and conversely, a low correlation coefficient would indicate that the traffic demand tends to fluctuate independently. Thus, a high correlation coefficient supports the argument that the intersection should not be under CIC control while a low correlation coefficient argues for CIC control.

One of the initial concerns of the researchers when this test was designed was that the test would be extremely insensitive; in that all of these intersections would tend to have very high correlation coefficients. This was based on the concept that as the traffic builds up on one of the major streets, such as K Street, it would also build up in an equal manner on the conflicting street, such as 19th Street. As shown on the previously referenced Table 9, this did not prove to be the case. The average correlation coefficient for the 22 intersections for which data were available was .64 for the a.m. condition .25 for the average condition and .33 for the p.m. condition. Since these correlation coefficients were considerably less than might be expected, doubt is cast on the validity of the test.

Further consideration of the test results yielded at least one plausible explanation for the phenomena. This would occur when one of the two approaches reaches congested conditions before the other. This could occur when either the capacity of the intersection is bound by insufficient green time for that phase or when congestion downstream inhibits vehicles discharging from the intersection while at the same time the other conflicting approach is experiencing relatively little downstream congestion. These conditions would result in a relatively free-flowing movement on one phase and a very constricted movement on the other. The free-flowing movement would react to the demand of the traffic while the constricted movement would produce only a limited flow. Thus, with these conditions the correlation coefficient would be extremely low. It is suspected that the above described phenomena occurs frequently in the UTCS area.

The original purpose of generating these two tests using the MOE data was to attempt to develop a quantifiable methodology of ranking in order of importance the existing CIC locations by taking advantage of the surveillance capabilities of a digital computer traffic control system. Based on the results of these tests, it would appear that neither test offers any advantage in quantifying the need for CIC intersections or shows any promise of development into a more refined technique.

Cycle Failures Measures

Collection of data for cycle and queue failure is described in an earlier chapter and Appendix A. Once the data had been collected, two techniques of analysis were used. The first consisted of establishing a percentage of time during which either a cycle failure or queue failure occurred on either approach. Thus, even if only one approach was consistently violating the criteria, a very high failure rate would be realized for that intersection. The second method consisted of determining when either a cycle or queue failure occurred simultaneously on both conflicting approaches. This measure was designed to minimize the impact of an inappropriate split. In the second method, both approaches would have to experience congestion for a high intersection failure rate to be realized. Data derived using both of these methods were then summarized for 15-minute time intervals, a period encompassing approximately 11 cycles. The summary was expressed in terms of the ratio of failed cycles to the total number of cycles in that time interval (11 for each 15-minute period). The 15-minute time intervals were then summarized into two-hour time periods between 8:00 and 10:00 a.m., and 4:00 and 6:00 p.m.

The analysis revealed that the first method, that is a failure on either approach, was very insensitive to the observed criticality of the intersection since a failure on either approach determined the overall intersection failure ratio. Consequently, many intersections had a high ratio based simply on the traffic condition of one approach. When the second method was used (simultaneous approach failure) an intersection incurred a high failure rate only when there were continuously heavy demands on both approaches. In this case, there appeared to be a much greater difference between the most critical intersections and those which appeared to be somewhat less critical. Those intersections which were heavily used on only one approach were essentially eliminated from critical intersection consideration.

A rating system was then devised in order to rank the intersections from most to least critical. This consisted of summing the ratios for the a.m. and p.m. peak two-hour periods. Thus, the maximum critical intersection value achievable was 2.00 for any one location. The failure ratios derived using the second method are shown in Table 10.

The next problem was the establishment of threshold values for determining when a critical intersection is in fact identified. This is strictly a subjective value, the magnitude of which determines the number of intersections declared critical. For this analysis, an intersection was declared to be critical if the failure ratio was 0.50 or greater. This is analogous to saying that an intersection is critical when a cycle failure or queue failure occurs on both approaches for 25% of the time during the peak traffic periods.

The intersections selected for CIC control using this methodology are shown graphically for Figure 6 along with the intersections currently instrumented for CIC control. In Section 1, the technique identified three locations on M Street at Key Bridge, at 33rd Street and at 34th Street. It did not identify M Street and Wisconsin Avenue as critical. The research team feels, however, because this is the intersection of two major arterials, it should be instumented for CIC.

In section 3, twenty-two intersections were identified as critical including five intersections that are not currently instrumented. It is recommended that these 22 intersections be considered as "Major" intersections for second generation LSTSQS and the remaining intersections in Section 3 be considered as "Minor." In addition, it is recommended that the intersection of K Street, Connecticut Avenue, and 17th Street be considered a "Major" or critical intersections because of the importance of Connecticut Avenue as an arterial   It is felt

Table 10. Critical intersection study - failure ratios.

Intersection	Failure Ratio
Penn./28/M	.40
M/29	.06
M/30	.20
M/31	.02
M/33	.82
M/34	1.35
M/Wisc.	.40
M/36	
M/Key Br.	1.65
Penn./26/L	.29
Penn./25/L	.83
Penn./24/K	.77
K/25	.02
Wash Cir. SW	Not Observed
Wash Cir NE	Not Observed
Penn./18	1.24
Penn./17	.76
Penn./Jackson	.10 a.m. only
K/19	1.27
K/18	.95
K/Conn./17	.33
I/20	*
I/19	*
I/18	*
I/17	*
Penn./19/H	.87
H/18	.27
H/17	1.03
Penn./20	.42
Penn./21	1.55
Penn./22	1.03
K/21	1.17
K/20	.85
L/23	.33
L/22/N.H.	.48
L/21	.37
L/20	.19
L/17	.68
L/16	.53
L/15	.86
L/19	1.03
L/18	.56
L/Conn.	1.85
K/17	.68
K/16	1.20
K/15	.11
I/17/Conn.	*
I/16	*
H/Conn./Jackson	Not Observed
H/16	1.15
L/Vt.	.37
L/14	.07
K/14	.01
K/15/Vt.	1.27

*(Note: Not observed due to Metro construction.)

Figure 6. Summary of CIC locations identified in field study.

that traffic diversion because of Metro construction in the area is the primary reason that this intersection was not identified.

A review of those identified did not indicate any apparent geometric conditions or traffic pattern trends related to the identification process. In fact, it is thought by the research team that if the data had been collected on another day, several different locations may have been identified as critical, and conversely, some of the identified CIC intersections may not be critical. This is not a limitation of the technique used, but rather an observation that different intersections in Section 3 on any particular day are in fact critical.

To further elaborate, in this highly congested CBD core area, the traffic volumes on all streets are heavy during peak periods. The traffic, then, is operating at a very poor level of service and is, in fact, unstable. Any incident such as a minor accident, double parked vehicle, or even a transient event - the passage of an emergency vehicle is sufficient to cause that intersection to become critical and the major constraint to the flow of traffic. Traffic then, backs up on the approaches to that intersection while downstream traffic is relatively free flowing. It is thought that this may occur at any one of the intersections in Section 3 on any particular day. For this reason, in order to evaluate third generation CIC/QMC, every intersection in Section 3 should be considered as critical. While it is unlikely that every intersection would be a same source of congestion, every intersection is within two blocks of potential "source" intersections and would, therefore, become a secondary critical intersection that must be under the control of the algorithm in order to manage the spread of congestion.

SYSTEM CONTROL RELATED REQUIREMENTS

By considering detector placement requirements for the system control algorithms, the emphasis is shifted from individual intersections to all links within the system. The first step in this task was to identify links that do not require detectorization. These are termed unimportant links and are generally characterized by the fact that an overriding constraint inhibits any flexibility that could be implemented by the control algorithm. The second was a detailed analysis of the system MOE parameters to determine whether an upstream link detector can predict results on downstream links with sufficient accuracy to eliminate the need for downstream detectorization. Since the MOE data has a 15-minute resolution, this analysis would only be applicable to first and second generation control algorithms. Because each generation of control algorithm is successively more responsive to real changes in traffic flow, a third technique was developed to quantitatively estimate the potential benefit of satisfying the real-time demands.

The basic approach in the design of CYRANO was to minimize the input data. As a result, only traffic volume is required for each link, stratified into its primary and secondary components. Because of third generation's dependency on this single input parameter, several techniques were developed to analyze this fourth area of primary flow measures.

Unimportant Links

Intersection approaches which do not require instrumentation were defined as unimportant links. These links were determined to be those that experienced a demand during the a.m. peak hour and p.m. peak hour that was less than that which can be accomodated by the minimum cross street green time. The minimum cross street green time was taken to be the equivalent of a seven second "WALK" interval plus a flashing "DON'T WALK" clearance interval of sufficient duration to allow a pedestrian crossing from the curb to the center of the farthest travel lane at a normal walking speed of four ft/sec. For example, the time required for a pedestrian to cross M Street in Section 1 was taken to be 21 seconds. To convert the green time to a service volume, the required green time was divided by an assumed cycle length to obtain a G/C ratio. In order to be conservative, a long cycle length (100 seconds) was selected. Finally, the service volume for that approach was computed using the technique outlined in the Highway Capacity Manual.

None of the intersections in Section 3 exhibited a green demand that could be serviced by the minimum green computed above. Four intersection side streets in Section 1, however, had peak hour demands that were less than that which can be handled by the minimum's green times. These intersections were: M Street at 33rd, 31st, 30th and 29th Streets.

Link MOE Comparisons

To evaluate the necessity for detectorizing adjacent links on any one street, an evaluation of the 15-minute average detector counts was made. The input data for this evaluation was the previously described MOE 15-minute summary tape generated during the evaluation of first generation UTCS/BPS. The comparison was made between the 15-minute detector count (expressed as an hourly flow rate in vehicles per hour) at the upstream link, and the detector count on the downstream link. This constituted one pair of data. Data were available on the tape for three time periods, a.m., midday, and p.m. periods. There were a total of 94 observations during the a.m. period, 62 observations during midday, and 69 observations during the p.m. period.

The computer program, SIMI, produced an average value for the upstream detector count ($\bar{X}$), downstream average ($\bar{Y}$), the variance, SY and SX, the correlation coefficient (R) and its square, the standard error of estimate (SE), the regression coefficients A and B, and a "t" statistic on A and B as to whether A was significantly different from zero and whether B was significantly different from one.

Essentially the paired data test is designed to determine how good a predictor the count at the upstream detector is of the traffic on the downstream link. The parameters utilized in the evaluation are the mean ($\bar{Y}$), the correlation coefficient, the standard error, and the ratio of standard error to the mean (the coefficient of variation).

The links that were selected for the comparisons were those that: 1) had similar detector configurations so that single detector links were only compared to single detector links, etc; 2) had all detectors operable during the time for which the MOE data were generated; 3) had a reasonable continuity of traffic flow on the two links; and 4) had at least three links in a row for which the above three criteria were met. A summary of the paired link data is shown on Table 11.

As stated previously, the accuracy goals for volume measures required by both first and second generation algorithms is plus or minus 10% for 90% of the time. If it is assumed that the errors are normally distributed about a regression estimate, then the projected mean value of the downstream flow can be estimated within plus or minus 1.645 times the standard error of the estimated mean value of the downstream flow. The range of standard errors extended from 22 to 85 with an average of approximately 44. Since these values are hourly flow rates, they correspond to 6,21 and 11 15-minute counts respectively. The 90% confidence limits for the mean 15-minute count would then be 10,35, and 18. To put these values within the context of the actual flow rates experienced, an average error of 18 with a typical traffic flow of 100 vehicles per 15 minutes results in an error of 18% - considerably higher than the stated goal of 10%.

These results suggest the need for a review of the accuracy goals in that they may have been set too stringently. The Proposed Modified Statement of Work suggested a range of volume error for LSTSQS of "from 1 to 3 vehicles per cycle." Since there are approximately forty-five 80 second cycles per hour, a simple extrapolation of this results in an accuracy goal of from 45 to 135 vehicles per hour. If an average flow

Table 11. Summary of paired link data.

Street/Link Number	AM Period				Midday Period				PM Period			
	R	Y	Se	Se/Y	R	Y	Se	Se/Y	R	Y	Se	Se/Y
L Street												
124-129	.94	460	48	.10	.49	300	43	.14	.62	260	50	.19
129-146	.91	520	55	.11	.56	375	40	.11	.77	300	43	.14
146-151	.90	420	55	.13	.15	320	44	.14	.48	365	50	.14
151-153	.85	360	58	.16	.51	350	49	.14	.49	400	50	.13
K Street EB												
109-114	.94	530	29	.05	.77	340	27	.08	.85	260	27	.10
114-73	.88	530	39	.07	.59	320	34	.11	.88	310	26	.08
73-77	.94	525	26	.05	.79	380	23	.06	.91	370	27	.07
77-80	.77	520	56	.11	.52	360	38	.11	.86	415	39	.09
K Street WB												
75-72	.77	230	22	.10	.65	220	38	.12	.83	450	38	.08
72-111	.65	275	25	.11	.61	310	37	.12	.84	525	48	.09
Penn-M Street EB												
13-15	.76	605	45	.07	.46	470	44	.09	.74	470	68	.14
15-18	.95	500	49	.10	.11	330	37	.11	.57	325	65	.20
15-52	.59	425	75	.18	.11	280	56	.20	.03	205	45	.15
52-56	.93	425	50	.12	.33	355	68	.19	.46	365	60	.16
59-65	.94	435	39	.09	.56	460	37	.08	.82	430	50	.12
65-68	.93	470	45	.10	.68	560	27	.05	.77	505	53	.10
Penn-M Street WB												
62-88	.66	290	43	.15	.77	400	20	.05	.63	300	44	.15
58-54	.73	205	27	.13	.00	260	55	.32	.50	305	37	.12
54-50	.72	175	30	.17	.01	170	69	.40	.56	290	37	.13
50-16	.60	235	36	.15	.17	280	33	.12	.80	405	62	.15
AVERAGE	.84	-	43	.11	.44	-	43	.14	.70	-	46	.13

rate of 450 vehicles per hour is assumed, and the suggested range is assumed to correspond to the 90% confidence level, then the goal can be restated as 10% to 30% for 90% of the time. The 18% typical error actually found with the standard error falls in the middle of this range and therefore it may be acceptable to instrument every second block when using relatively insensitive control algorithms.

A more detailed review of Table 11 shows several significant trends. The a.m. period had the highest correlation coefficient and the lowest coefficient of variation. This is to be expected since the system has the lowest marginal friction during the a.m. period. The average standard error varied little; 43 in the a.m. and midday, and 46 during the p.m. This suggests that even though the correlation was poor during the midday period, the error inherent in the predictions may be tolerable.

As a final point, both the correlation coefficient and the coefficient of variation tended to be better when the major traffic flow was in the direction of the link set. For example, Pennsylvania Avenue - M Street had much better characteristics eastbound in the morning than westbound; and conversely, westbound in the evening than eastbound.

To summarize the paired link analysis, the coefficient of variation tended to improve at higher volume levels indicating that in general, there is a higher probability of predicting downstream flows on higher volume streets. The coefficient of variation ranged overall from 5% to 40% but showed a reasonable degree of stability particularly during the peak hours; 5% to 18% during the a.m. and 7% to 20% during the p.m. The correlation coefficient was far more volatile ranging from .00 to .96. Again the peak hours in general showed greater stability. The standard error is perhaps the measure of greatest interest, ranging from a low of 22 to a high of 85 vehicles per hour. In terms of a 15-minute count, the range is from 5 to 22. Of particular importance is the average value during each of the three periods, ranging only from 43 to 46 vehicles per hour. It must be emphasized, however, that the standard error is measured about the mean values and that as the independent variables deviate from the mean, the error of estimate will increase from that shown here.

An additional concern was the degradation of the estimate when the comparisons were extended to two links and then three links downstream. To investigate this, where a series of four or more contiguous links along a street were available, the paired links regression curves were fit to the data. As might be anticipated, there was a significant degradation of all three statistics, correlation coefficient, standard error, and coefficient of variation as shown on Table 12.

The same general trends observed with the alternate link analysis are apparent with the every second and third link analysis. The a.m. consistently produced the highest correlation coefficients and the midday the lowest. The standard errors increased with the most pronounced increase occurring during the a.m. condition, from 44 with alternate links, to 61 with every second link, and 71 with every third link. The coefficient of variation was the most stable statistic with an average value of 13% with every second link and an average value of 16% with every third link. Again it must be emphasized that the standard error estimates are based on mean values, and therefore, both the standard error and the coefficient of variation must be taken as an indication of minimum values.

The basic conclusion that can be drawn from the above analysis is that alternate link detectorization is not an optimum detector scheme for the second generation control algorithm. If for economic reasons, however, the number of sensors must be minimized, a rational method of designing the detector scheme would be to detectorize every second block on streets that have a reasonable continuity of traffic flow.

By inverting the comparisons, and looking for the number of blocks between detectorized links where traffic patterns exhibit major differences, a criteria is identified to locate detectors for first generation traffic responsive. Because major differences are evident when there are three blocks between detectorized links, it is concluded that instrumenting every fourth block would be sufficient for arterials and perhaps, every second block within a grid network.

Offset Benefit

By making the assumption that second and third generation algorithms can develop better offset patterns for a link that is detectorized than for a link that has no surveillance, a technique was developed to evaluate each link in Sections 1 and 3 to estimate the expected benefit that can be achieved if the link were detectorized. The method makes the basic assumption that when a signalized intersection has a uniform arrival rate and there is little or no advantage to selecting any particular offset since the delay and number of vehicles stopped are simply proportional to the percent red. When the traffic approaching a signalized intersection is not uniform, however, and has a pulsing cyclical variation in the arrival

Table 12. Link series comparisons.

Street-/Link Time /Numbers	1 link downstream				2 links downstream				3 links downstream			
	R	Y	Se	Se/Y	R	Y	Se	Se/Y	R	Y	Se	Se/Y
L Street - AM												
124/129-146-151	.94	460	48	.10	.87	520	65	.13	.79	420	75	.18
129/146-151-153	.91	520	55	.11	.82	420	71	.17	.82	360	63	.18
L Street - Mid												
124-129-146-151	.49	300	43	.14	.31	375	47	.13	.21	320	44	.14
129/146-151-153	.56	375	40	.11	.21	320	44	.14	.19	350	55	.16
L Street - PM												
124/129-146-151	.62	260	50	.19	.46	300	60	.20	.14	365	56	.15
129/146-151-153	.77	300	43	.14	.32	365	54	.15	.15	395	57	.14
K Street EB - AM												
109/114-73-77	.94	535	29	.05	.80	525	49	.09		No Data		
114/73-77-80	.88	525	39	.07	.78	525	48	.09	.48	520	79	.15
K Street EB - Mid												
109/114-73-77	.77	335	27	.08	.58	320	34	.11		No Data		
114/73-77-80	.59	320	34	.11	.64	380	29	.08	.43	360	40	.11
K Street EB - PM												
109/114-73-77	.85	260	27	.10	.69	310	41	.13		No Data		
114/73-77-80	.89	310	26	.08	.83	370	36	.10	.70	415	54	.13
Penn. Ave./M St. EB - AM												
13/15-18-52	.96	605	45	.07	.93	500	57	.11	.87	425	80	.19
15/18-52-56	.95	500	49	.10	.89	425	75	.18	.90	425	59	.14
Penn. Ave./M St. EB - Mid												
13/15-18-52	.46	470	45	.10	.11	325	37	.11	.19	280	55	.20
15/18-52-56	.11	325	37	.11	.08	280	56	.20	.17	355	71	.20
Penn. Ave/M St. EB - PM												
13/15-18-52	.74	470	68	.14	.62	325	63	.19	.06	295	45	.15
15/18-52-56	.57	325	66	.20	.01	295	45	.15	.14	365	67	.18
AVERAGE AM	.93	-	44	.08	.85	-	61	.13	.77	-	71	.17
AVERAGE Mid	.50	-	38	.11	.32	-	41	.13	.24	-	53	.16
AVERAGE PM	.74	-	47	.14	.49	-	50	.15	.24	-	56	.15

rate, then significant advantages can be achieved by displaying the green during periods of high flow. A graphics illustration of the Offset Benefit is shown on Figure 7.

The measure itself is the ratio of the time required by the primary volume to the total green time available at the downstream intersection. The primary volume is expressed in vehicles per hour and is the major flow of traffic onto the link from the upstream intersection. In the majority of cases, this would be the straight through traffic from the upstream intersection. This volume is converted into a green time demand expressed in seconds by assuming a conservative arrival headway of three seconds per vehicle and multiplying by the percentage of traffic in the critical lane. The percentage of traffic in the critical lane was measured at approximately 30 links in Sections 1 and 3 that were chosen to be representative of the remaining links in the system. The critical lane ratios are shown on Table 13. The demand value is then divided by the green time available which is simply the percent green for that phase multiplied by 3,600 (seconds per hour).

Since the numerator is an estimate of the platoon length in seconds and the denominator is an estimate of the green window (also in seconds) available to that platoon, the measure is an indication of how sensitive that link is to the offset selected. Stated conversely, when the ratio is low, there is less penalty incurred on that link even though the relative offset is not optimum.

The first step in computing this measure was to plot the turning movements of all intersections in Sections 1 and 3 on a large map of the network. The source of the turning movement data were manual turning movement counts conducted during April through August 1974, by A.M. Voorhees and Associates. The information was originally collected for simulation input to the UTCS-1 Model. In addition, available turning movement counts were acquired from the D.C. Department of Highways. This data was also plotted on a large map of the network and used as a credibility check for the UTCS-1 simulation input data.

The turning movement maps were developed for an a.m. peak hour, a typical midday hour, and a p.m. peak hour. Similarly, the percent of traffic in the critical lane was developed for an a.m. peak hour, a typical midday hour, and a p.m. peak hour. A summary of the hourly approval volumes are shown on Figures 8,9, and 10. The primary flow onto each link was taken directly from the turning movement data. Ideal splits for each intersection were then computed. The

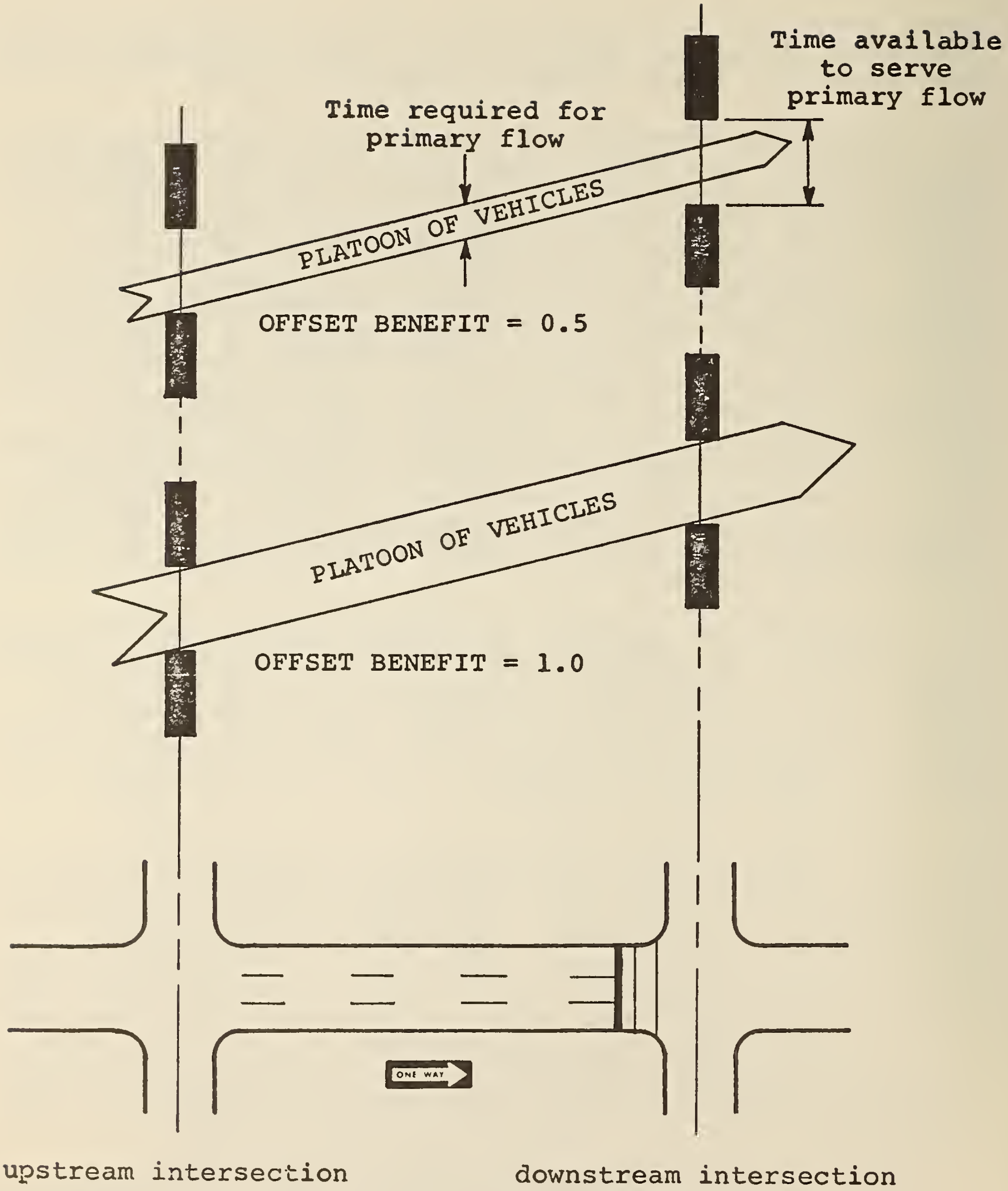

Figure 7. Diagram showing offset benefit.

Table 13. Ratio of critical lane volume to total link volume.

Study Approach	AM	MID	PM
1. 20th @ K St. NB	.39	.58	.44
2. K St. @ 19th EB	.66	.30	.55
3. K St. @ 19th WB	.56	.64	NA
4. 19th @ K St. SB	.50	.70	.49
5. K St. @ 18th EB	.52	.56	.55
6. K St. @ 18th WB	.58	.65	.68
7. K St.* @ 17th WB	.66	.51	.68
8. 16th @ K St. SB	.56	.78	.54
9. K St. @ 15th WB	.69	.52	.63
10. K St. @ 15th WB	.62	.55	.63
11. M St. @ 34th EB	.40	.52	.48
12. M St. @ 34th WB	.78	.50	.34
13. 34th @ M St. SB	NA	NA	NA
14. Penn. @ 28th EB	.40	.54	.51
15. Penn. @ 28th WB	.56	.61	.45
16. 21st @ Penn. SB	.55	.69	.55
17. Penn. @ 19th EB	.38	.56	.47
18. Penn. @ 19th WB	.39	.54	.43
19. Penn. @ 18th WB	.44	.53	.56
20. Penn. @ 17th EB	.47	.46	.56
21. H St. @ 16th WB	.39	.47	.39
22. 17th @ H St. NB	.39	.37	.28
23. 17th @ H St. SB	.70	.91	.62
24. L St. @ 20th EB	.38	.59	.44
25. L St. @ 19th EB	NA	.43	.47
26. L St. @ 15th EB	.37	.43	.42
27. 20th @ L St. NB	.45	.50	.47
28. 19th @ Penn. SB	.54	.69	.42
29. M St. @ 28th WB	NA	NA	NA
30. H St. @ 16th EB	.61	.62	.58

*Westerly intersection.

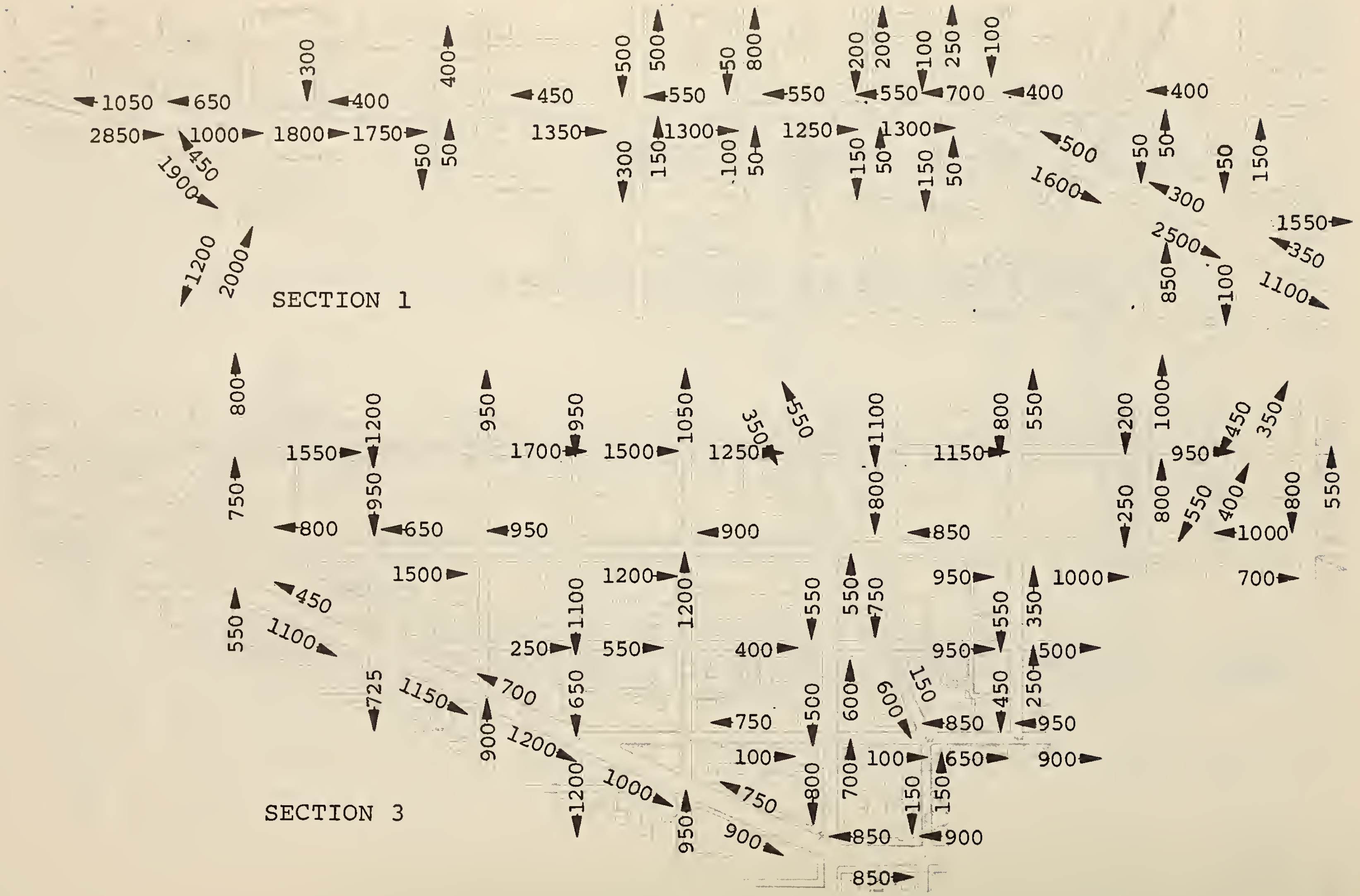

Figure 8. Hourly approach volumes, a.m. period.

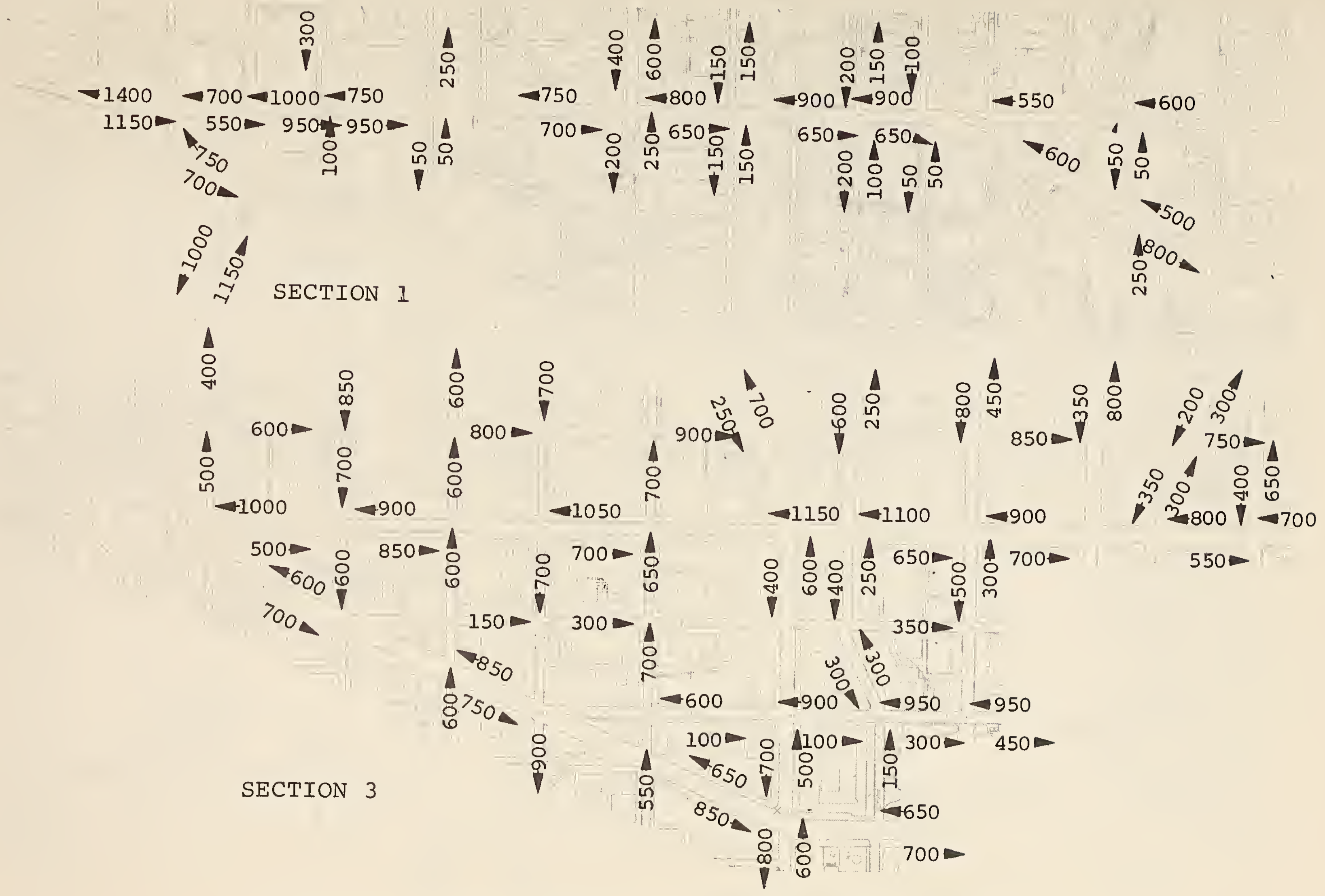

Figure 9. Hourly approach volumes, midday period.

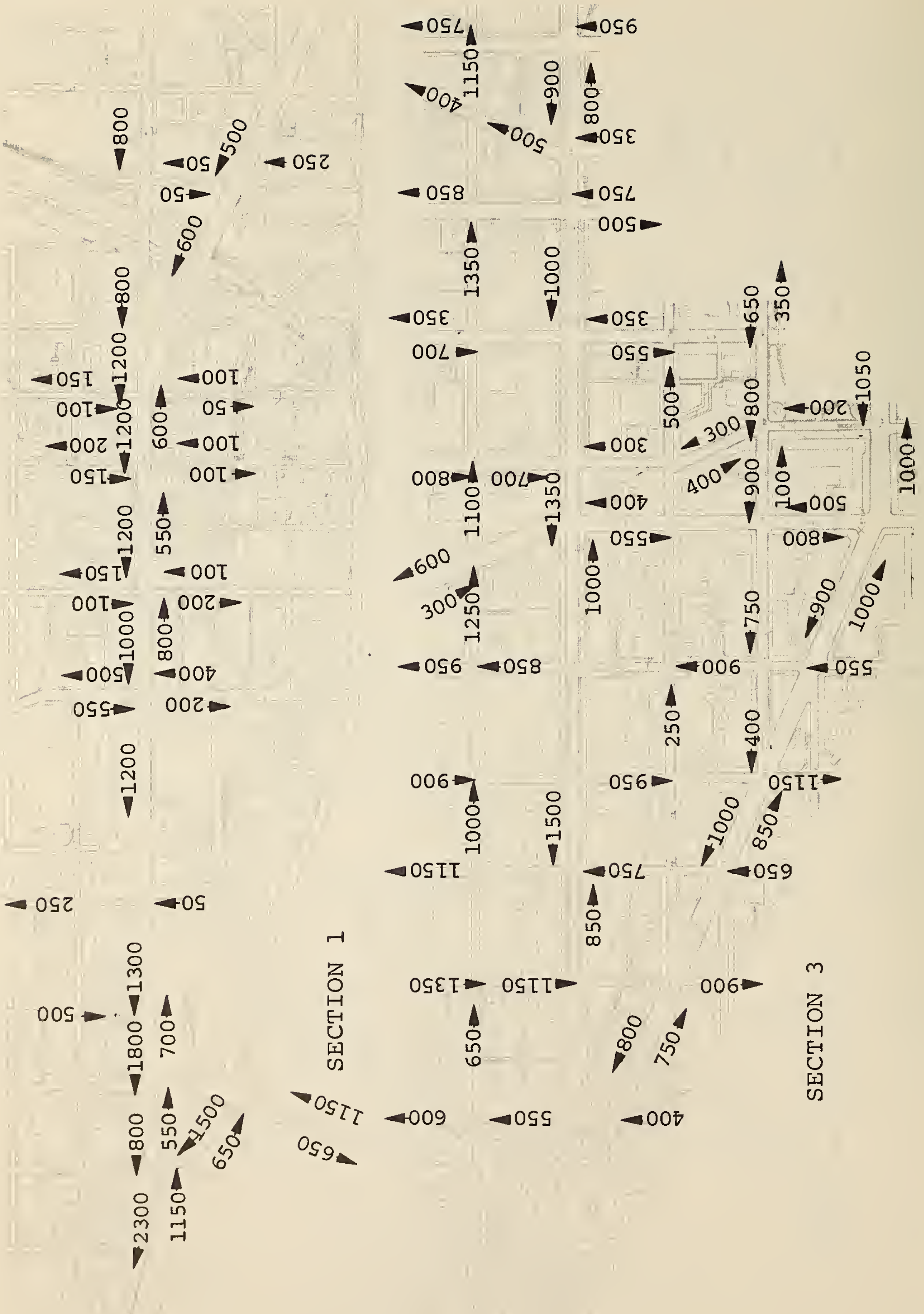

Figure 10. Hourly approach volumes, p.m. period.

splits were made in proportion to the critical demand on each competing approach with the critical demand being defined as the product of the approach traffic times the percent of traffic in the critical lane. The offset benefit was then calculated for each link in Sections 1 and 3 for each of the three time periods using the equation shown below:

$$OB = (3*Vp*R)/(G*3600)$$

where

OB = Offset Benefit
3 = Assumed headway (seconds per vehicle)
Vp = Primary Volume (vehicles per hour)
R = Ratio of critical lane to total link volume
G = Percent green for downstream intersection
3600 = Seconds per hour

The results of typical calculations are shown in tabular form on Table 14.

The mean value and standard deviation of the offset benefits were then computed. The mean was found to be 0.57 and the standard deviation was 0.21. The location of the links that had an offset benefit of greater than one standard deviation (0.78) and the links that had a value greater than the mean but less than one standard deviation were plotted on Figure 11.

The research team feels that there are several significant advantages to using the Offset Benefit Technique:

1 - The required input data are generally available and can be estimated to a reasonable degree of precision.

2 - The technique is direct, involving simple mathematical manipulations.

3 - The technique yields a quantitative estimate of the expected benefit that can be realized from detectorizing a particular link.

4 - Because of the traffic responsive characteristics of second and third generation control algorithms, LSTSQS and CYRANO, it is felt that the technique produces a valid ranking of the priority of link detectorization.

Table 14. Typical link offset benefit.

Intersection	Link	AM	Midday	PM
K Street & 19th Street	K Street EB	1.18	0.28	0.66
	K Street WB	.49	0.86	0.80
Pennsylvania & 22 Street	Pennsylvania NWB	0.24	0.27	0.34
	Pennsylvania SEB	0.51	0.27	0.32
	22nd Street NB	0.51	0.35	0.33
K Street & 20th Street	K Street EB	0.18	0.24	0.56
	K Street WB	0.54	0.73	0.82
	20th Street NB	1.22	0.86	0.90
L Street & 16th Street	L Street EB	0.61	0.30	0.61
	16th Street NB	0.28	0.17	0.34
	16th Street SB	0.61	0.48	0.64
L Street & 19th Street	L Street EB	0.73	0.55	0.54
	19th Street SB	0.59	0.60	0.68
M Street & Key Bridge	M Street EB	1.13	1.98	1.57
	M Street WB	0.73	0.32	0.22
M Street & 34th Street	M Street EB	0.68	0.66	1.08
	M Street WB	0.39	0.51	0.79
M Street & Wisconsin	M Street EB	0.46	0.27	0.62
	M Street WB	0.14	0.36	0.25

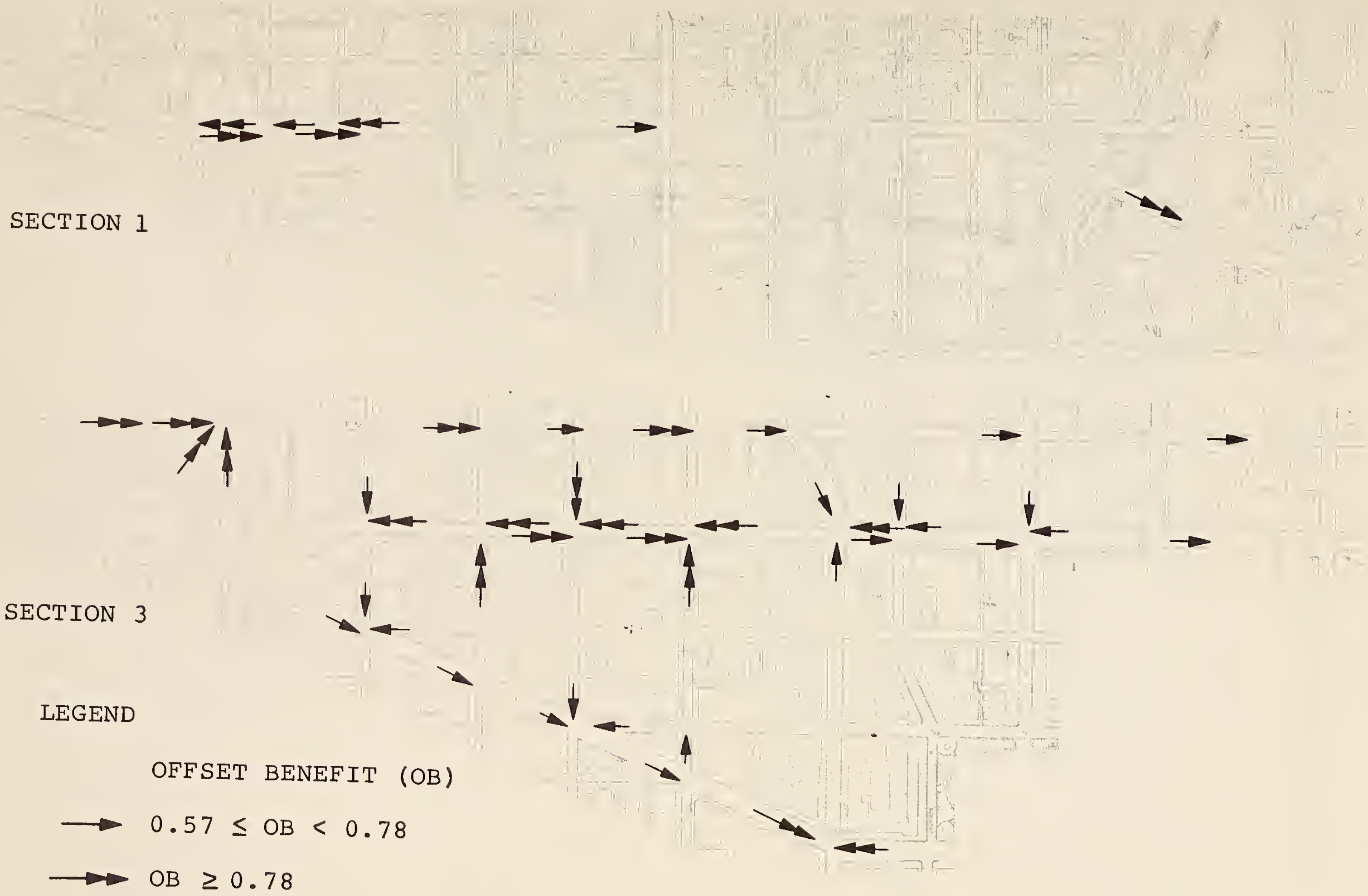

Figure 11. High offset benefit links.

A basic limitation of the technique is that it considers only links that are internal to the network since there can be no offset benefit for links on the boundary of the network.

Primary Volume Measures

Two distinct problems are addressed in the analysis of primary volume measures. The first deals with descriptive statistics concerning the actual variation in the primary to total volume ratio as measured in the field at 30 locations. The second is a direct comparison of the field measures, with an estimate of the primary volume ratio based strictly on turning movement counts, and with the actual values currently being used by CYRANO in the HISGEN file.

The underlying object of both of these efforts is to ascertain the magnitude of the errors that can be expected if an estimate of the primary volume is made by multiplying a cycle by cycle count by a stored ratio appropriate for the particular time period. Since there is no simple direct method of instrumenting for primary volume, manual field collection techniques were used.

Observed Primary Volume

The basic data collected was the primary and secondary volumes, stratified by lane, recorded on a cycle by cycle basis. This was done for 25 cycles by an observer stationed within 100 feet of the upstream intersection. It must be noted that the data collection procedure itself introduces an error beyond the normal observation type errors in that the volumes of interest are the demand volumes at the downstream intersection. This is impossible to observe during heavy volume conditions, however, and it was decided that the upstream values were a reasonable approximation. When the figure of merit is taken to be the ratio of primary flow to total flow rather than the absolute values, then the ratio approximation is exact if the vehicles leaving the link (entering parking garages, alleys, or other sinks) are in proportion to the primary and secondary flows, and negligible traffic enters the link from sources within the link. These conditions are frequently met during the a.m. peak and therefore, more credence is placed on the data collected during the morning than during the other periods. The results of the field studies are tabulated on Table 15. For each of the three periods the average volume per cycle ($\overline{V}$), the primary flow ratio (V_p/V_t), the standard deviation of the primary flow (SD), the 90% confidence limits expressed in vehicles per cycle, and the coefficient of variation are shown.

Table 15. Field observed primary/secondary flow ratios.

	AM PERIOD					MIDDAY PERIOD					PM PERIOD				
	Veh/Cycle	Vp/Vt	Std. Dev.	90% Conf. Inter.	Coeff.of Var. (%)	Veh/Cycle	Vp/Vt	Std. Dev.	90% Conf. Inter.	Coeff.of Var. (%)	Veh/Cycle	Vp/Vt	Std. Dev.	90% Conf. Inter.	Coeff.of Var. (%)
1. 20th @ K St. NB	31	.98	.03	1.5	3	17	.95	.06	1.7	6	16	.95	.06	1.6	6
2. K St. @ 19th EB	21	.96	.05	1.7	5	13	.92	.07	1.5	8	12	.80	.14	2.8	18
3. K St. @ 19th WB	11	.91	.16	2.9	18	17	.84	.09	2.5	11	-	-	-	-	-
4. 19th @ K St. SB	26	.77	.09	3.8	12	16	.75	.11	2.9	15	18	.76	.11	3.3	14
5. K St. @ 18th EB	20	.91	.07	2.3	8	16	.81	.18	4.7	22	13	.95	.06	1.3	6
6. K St. @ 18th WB	23	.85	.07	2.6	8	13	.92	.08	1.7	9	16	.94	.06	1.6	6
7. K St.* @ 17th WB	15	.95	.05	4.5	5	19	.84	.06	1.9	7	15	.92	.06	1.5	6
8. 16th @ K St. SB	16	.92	.06	1.6	6	14	.73	.17	3.9	23	19	.75	.08	2.5	11
9. K St. @ 15th EB	18	.91	.08	2.4	9	16	.86	.08	2.1	9	17	.91	.08	2.2	9
10. K St. @ 15th WB	15	.89	.08	2.0	9	15	.85	.11	2.7	13	13	.80	.09	1.9	11
11. M St. @ 34th EB	43	.77	.05	3.5	6	20	.79	.09	3.0	11	15	.88	.08	2.0	9
12. M St. @ 34th WB	8	.99	.02	0.3	2	15	.97	.04	1.0	4	36	.99	.01	0.6	1
13. 34th @ M St. SB	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
14. Penn. @ 28th EB	27	.94	.04	1.8	4	15	.97	.04	1.0	4	15	.92	.07	1.7	8
15. Penn. @ 28th WB	10	.56	.16	2.6	28	11	.85	.13	2.4	15	18	.72	.14	4.1	19
16. 21st @ Penn. SB	11	.83	.12	2.2	14	12	.80	.12	2.4	15	23	.88	.07	2.6	8
17. Penn. @ 19th EB	14	.93	.05	1.2	5	20	.95	.04	1.3	4	20	.96	.05	1.6	5
18. Penn. @ 19th WB	19	.66	.10	3.1	15	16	.91	.08	2.1	9	19	.88	.09	2.8	10
19. Penn. @ 18th WB	18	.82	.10	3.0	12	17	.85	.08	2.2	9	16	.84	.10	2.6	12
20. Penn. @ 17th EB	25	.73	.06	2.8	8	18	.85	.09	2.7	11	24	.73	.15	5.9	21
21. H St. @ 16th WB	21	.58	.13	4.5	22	16	.58	.13	3.4	22	13	.62	.14	3.0	23
22. 17th @ H St. NB	17	.80	.10	2.8	13	11	.68	.15	2.7	22	8	.87	.15	2.0	17
23. 17th @ H St. SB	9	.87	.10	1.5	11	-	-	-	-	-	12	.83	.29	5.7	35
24. L St. @ 20th EB	37	.78	.05	3.0	6	-	-	-	-	-	15	.66	.10	3.0	15
25. L St. @ 19th EB	36	.67	.06	3.6	9	18	.65	.13	3.8	20	25	.71	.10	4.1	14
26. L St. @ 15th EB	-	-	-	-	-	22	.67	.11	4.0	16	33	.78	.06	3.3	8
27. 20th @ L St. NB	23	.74	.09	3.4	12	16	.64	.11	2.9	17	17	.67	.11	3.1	16
28. 19th @ Penn. SB	14	.93	.07	1.6	8	-	-	-	-	-	23	.97	.03	1.1	3
29. M St. @ 28th WB	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
30. H St. @ 16th EB	8	.48	.21	2.8	50	16	.49	.12	3.2	24	9	.60	.12	1.8	20
AVERAGE					11					13					12

*Westerly intersection.

The standard deviation of the primary flow ratio ranged from a low of .01 to a high of .29. In order to normalize the standard deviation with respect to the primary flow ratio, a coefficient of variation was computed. This statistic had a range of 1% to 50% with a mean value of 12%. When the coefficient of variation was plotted against average link volume, an inverse relationship between the two variables became apparent. The scatter diagram of the data points is shown on Figure 12. Although there was a considerable scatter of the data, high volume links tended to have low correlation coefficients and the reverse for low volume links. The low coefficient of variation of relatively high average volumes is encouraging while the high coefficient of variation at low volumes is not necessarily a problem. For example, one data point with a high coefficient of variation of 18% had an average volume of 10 vehicles per cycle and a standard deviation of .16. These values result in 90% confidence intervals of 2.6 vehicles per cycle - within the 1-3 vehicles per cycle criteria suggested in the Proposed Modified Statement of Work. Critical values tend to occur at midpoints in the range of both variables, for example, another data point, with a coefficient of variation of 21%, had an average volume of 24 and a standard deviation of .15. The 90% confidence intervals for this example was 5.9 vehicles per cycle - considerably beyond the suggested error limits.

A careful study of the 90% confidence limits shows a range of from 0.3 to 5.7 vehicles per cycle. It must be remembered that these values are total approach volumes. If a form of critical lane detection is to be used, then the 90% confidence limits would be expected to be less than that shown here, however, the standard deviation would tend to increase because of the smaller samples. This point is discussed in more detail in the following chapter.

Historical Primary Volume

Three techniques were employed to estimate the primary volume ratios for the thirty typical approaches within the study area: the field measures described above, the values currently being used by CYRANO in the HISGEN file, and primary volume ratio estimates based on turning movement data that was developed for the Offset Benefit calculations. Because HISGEN actually stores secondary flow rates, the primary flow ratios were calculated for comparison purposes by subtracting the secondary flow ratios from 1.00. The primary flow ratios were developed from the turning movement data by dividing the largest link input flow by the total flow on the link. The three values are shown on Table 16.

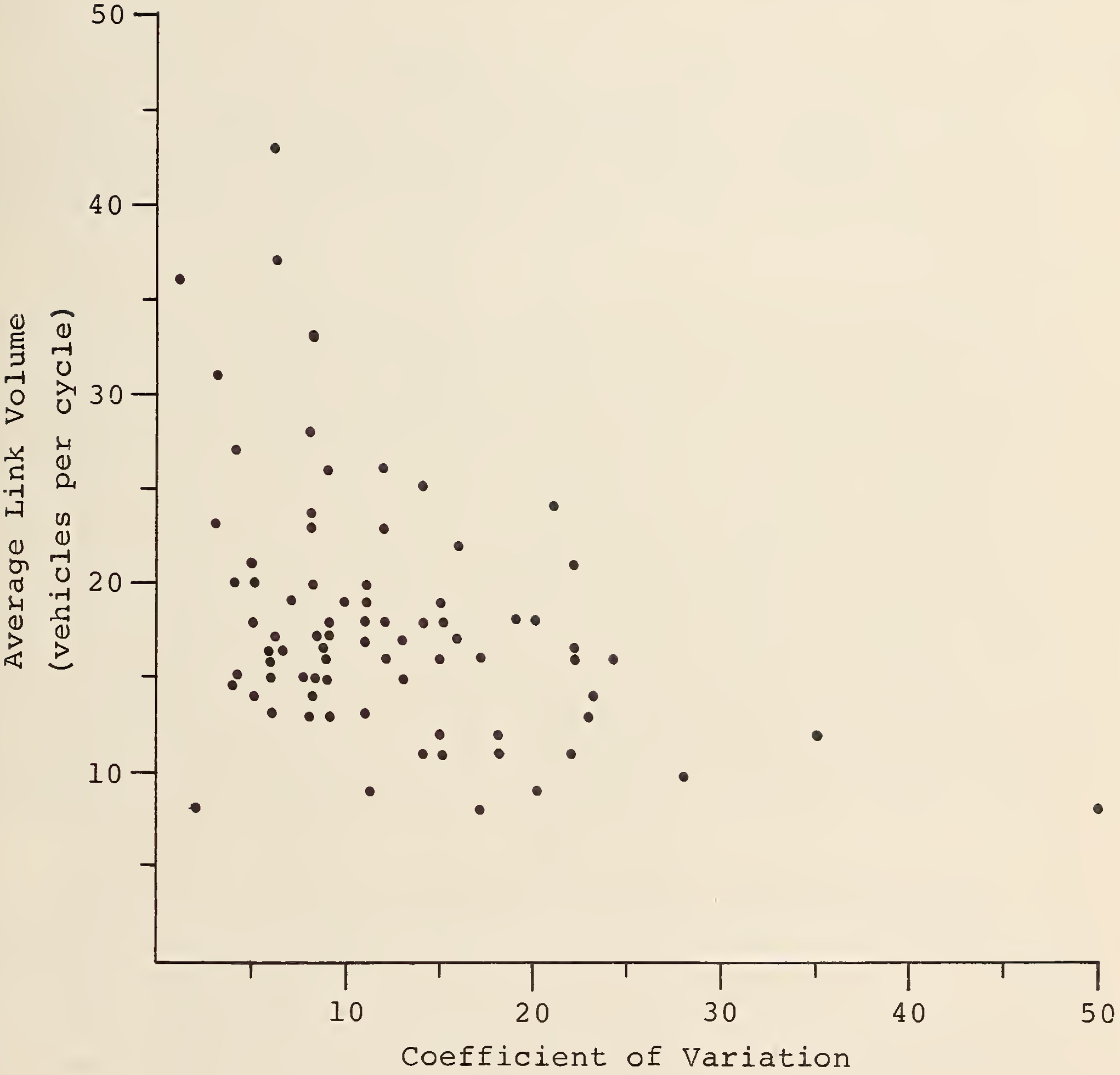

Figure 12. Diagram of link volume to coefficient of variation for primary/secondary flows.

Table 16. Historical data based primary flow ratios.

APPROACH	AM PERIOD			MIDDAY PERIOD			PM PERIOD		
	Field Data	HISGEN	Turning Movements	Field Data	HISGEN	Turning Movements	Field Data	HISGEN	Turning Movements
1. 20th @ K St. NB	.98	1.00	1.00	.95	1.00	1.00	.95	1.00	1.00
2. K St. @ 19th EB	.96	.94	.94	.92	.80	.81	.80	.82	.83
3. K St. @ 19th WB	.91	.81	.84	.84	.92	.90	NA	.86	.86
4. 19th @ K St. SB	.77	.82	.80	.75	.80	.77	.76	.73	.73
5. K St. @ 18th EB	.91	.95	.95	.81	.92	.93	.95	.93	.94
6. K St. @ 18th WB	.85	.88	.87	.92	.82	.83	.94	.90	.90
7. K St.* @ 19th WB	.95	.82	.83	.84	.80	.82	.92	.80	.81
8. 16th @ K St. SB	.92	.93	.92	.73	.84	.85	.75	.81	.80
9. K St. @ 15th EB	.91	.88	.88	.86	.76	.76	.91	.86	.86
10. K St. @ 15th WB	.89	.80	.82	.85	.73	.74	.80	.74	.73
11. M St. @ 34th EB	.77	.80	.80	.79	.75	.74	.88	.85	.85
12. M St. @ 34th WB	.99	.97	.97	.97	.97	.97	.99	.99	.99
13. 34th @ M St. SB	NA	1.00	NA	NA	1.00	NA	NA	.99	NA
14. Penn. @ 28th EB	.94	.95	.95	.97	.91	.91	.92	.88	.88
15. Penn. @ 28th WB	.56	.63	.62	.85	.79	.79	.72	.83	.81
16. 21st @ Penn. SB	.83	.83	.85	.80	.88	.86	.88	.87	NA
17. Penn. @ 19th EB	.93	.94	.87	.95	.90	.91	.96	.95	.95
18. Penn. @ 19th WB	.66	.84	.90	.91	.85	.92	.88	.88	.96
19. Penn. @ 18th WB	.82	.74	.81	.85	.77	.70	.84	.87	.87
20. Penn. @ 17th EB	.73	.93	.94	.85	.88	.88	.73	.90	.90
21. H St. @ 16th WB	.58	1.00	NA	.53	1.00	NA	.62	1.00	NA
22. 17th @ H St. NB	.80	.82	.83	.68	.87	.85	.87	.93	.84
23. 17th @ H St. SB	.87	.88	.90	NA	.81	.78	.93	.81	.82
24. L St. @ 20th EB	.78	.77	.78	NA	.65	.65	.66	.60	.60
25. L St. @ 19th EB	.67	.71	.71	.65	.77	.76	.71	.75	.74
26. L St. @ 15th EB	NA	.69	.70	.67	.66	.70	.78	.79	.80
27. 20th @ L St. NB	.74	.69	.71	.64	.58	.55	.67	.66	.66
28. 19th @ Penn. SB	.93	.97	.91	NA	.94	.95	.97	.96	.97
29. M St. @ 28th WB	NA	1.00	.99	NA	1.00	.99	NA	1.00	.99
30. H St. @ 16th EB	.48	.85	.85	.49	.64	.68	.60	.73	.62

*Westerly intersection.

An inspection of this table reveals a close agreement between the values used by HISGEN and the values generated from the turning movement data. This was expected since the original source of both values was the field data collected in April through August 1974, for use as input to the UTCS-1 simulation model.

The differences between the primary volume field data and either HISGEN or the turning movement data are far more pronounced. Approximately 26% of the comparison had differences of greater than 0.10. No trend relating the geometric configuration on operating characteristics to the magnitude of the error could be discerned.

From the analysis of the measured and estimated primary volume ratios, two conclusions may be drawn. First, the real-time volatility of the primary volume ratios on a cycle by cycle basis is such that errors introduced solely by the randomness of traffic, excluding errors of surveillance, are enough to make an estimate of primary volume volume (based on multiplying a cycle volume count by a primary volume ratio) inadequate for use in the third generation control algorithms. Secondly, even if these error rates were acceptable, an estimate of the primary volume ratio based on turning movement counts does not yield results within 0.10 of those actually observed in 26 percent of the cases studied. To relate this to a cycle error in count, using a ratio of 0.80 rather than an assumed true value of 0.90, results in an error of 2 vehicles per cycle undercount bias with a flow of 20 vehicles per cycle which then must be combined with the measurement and surveillance errors.

Because the use of historical data to estimate primary flow appears to be insufficient for the algorithm input requirements, two alternative techniques are suggested that enable an estimate to be made of primary volume in real-time. The first technique uses a volume pattern matching algorithm that identifies the volume pattern across a detector on a link as a function of the cycle length. Studies by Webster have shown that a definite bi-modal pattern exists at flows less than the saturation rate. This algorithm would identify the primary volume at the cordon as the larger of the two modes. The advantage of this type algorithm is that it can identify changes in the primary volume and respond in real time. For example, if the upstream intersection has two phases and during part of the day one phase approach constitutes the primary volume, while during other times of the day another phase approach constitutes the primary traffic volume, this algorithm would be able to identify the true primary volume in real time and provide an accurate measure of the primary volume.

The second technique that may be used to identify primary volume is the signal phase related counting technique. With this algorithm, the phase contributing the primary volume from the upstream intersection is assumed to be known. The green time for the phase serving the primary volume is similarly assumed to be known. By assuming average acceleration characteristics and average speeds for the link, the displacement in time of the "green window" can be transferred from the upstream signal to the location of the detector on the downstream link. With this algorithm, then, the number of vehicles crossing the detector during the time of the primary phase "green window" is classified as the primary volume.

CONCLUSIONS

- First and Second Generation CIC algorithms (CIC and LOCAL) require instrumentation on all major approach links to the CIC controlled intersection. Approaches not requiring detectorization are those that move concurrently with a major approach and never exhibit a demand greater than the major approach demand.

- Third Generation (CIC/QMC) requires instrumentation on all links between signalized intersections internal to the subnetwork for which the algorithm is designated.

- First Generation traffic responsive (TRSP) requires instrumentation on the entrance links and approximately every fourth block thereafter on arterials and every third block within grid networks. Links that carry low volumes (peak hour volumes of less than 100 vph/lane should not be instrumented because of their volatility and unimportance of basic platoon flows.

- Second Generation (LSTSQS) requires instrumentation on all links between "major" intersections which are defined as those intersections operating within the strategic optimization routine.

- Third Generation (CYRANO) requires instrumentation on all links between signalized intersections.

Because vehicle count is the single common input element of all the local and system control algorithms used in UTCS, this parameter has been stressed in this chapter. Vehicle count (volume) has another characteristic that is particularly relevent. It is the only algorithm input that can be collected without elaborate and costly electronic mechanisms such as radar meters or occupancy counters. As a result of this characteristic, studies using volume measures in various forms are the ones that have the greatest potential for use in the design of other surveillance systems. The development of the methodology used in the analysis and the transferability of these techniques to the design of other systems is considered an important aspect of the UTCS research. A list of the recommended studies and their relationship to a design of a surveillance system is shown below:

- In locations where extreme congestion is not experienced on intersection receiving links, volume capacity measures should be used to identify critical intersections.
- In locations where extreme congestion exists, a cycle failure study should be conducted to identify critical intersections.
- Offset Benefit study should be conducted for systems using second or third generation control with the results being used to identify "major" or critical intersections and as a method of assigning priorities for system detectors.
- Primary secondary volume studies should be conducted for all links internal to a network of signalized intersections for systems using third generation control.

DETECTOR PLACEMENT WITHIN A LINK

INTRODUCTION

The development of the rationale of where detectors should be placed within a link uses as a primary source the data collected by the temporary loop detectors. These data were extensively developed in the longitudinal comparisons wherein the detector values are first compared to the observed values, then with values generated at three locations within the link. These comparisons include a detailed analysis on a cycle by cycle basis of both volume and queue estimates. The data also provided a source for evaluation of several different techniques to identify "free flow speed," and an analysis of the operating characteristics of multilane loop detectors.

A second item addressed in this section is the viability of using a critical lane detectorization scheme. Several methods are developed to identify the critical lane and a technique is developed that provides an estimate of the penalty associated with not detectorizing a lane that may be critical during some cycles but not always. Finally, the effects of sinks and sources are investigated.

LONGITUDINAL COMPARISONS

This study included detailed analysis of the vehicle pulses generated by temporary loop detectors installed at various locations within nine links in the study area.

The input data for the computerized studies came from two sources. The first source was the raw history or pulse tapes generated by the UTCS system at the control center.

The raw history tapes consist of one record generated each half-second. Three particular types of data were required for the study. The basic input was the presence indication for each of the detectors in the system. Also used, for control purposes, was the status of the A-phase green return flag and the advance pulse flag. These data enabled the reconstruction of the vehicle pulses by cycle and by phase.

The second data set was prepared from field data collected by observers at several locations in the network. These data included a count by lane by cycle on each of the study links. In addition, observations were made as to the lane-specific link content at the beginning of green for the link-phase, 20 seconds after the beginning of green, and at the end of green.

An example of a typical link instrumentation scheme is shown on Figure 13. This figure indicates the location of the temporary loop detectors at the Q_1, Q_2, and Q_3 positions which were 35 feet, 210 feet, and 270-410 feet upstream from the stop line. The figure also shows the downstream and upstream zone count regions and the count line. Details of the temporary loop installation and the manual counting procedures are given in Appendix A.

Detector data at the Q_2 locations were matched with observations yielding 56 data points on nine different links. Because observations were conducted throughout the week on L Street between 20th Street and 19th Street and on westbound Pennsylvania Avenue between 18th Street and 17th Street, 19 and 12 data points were available respectively on these two links. The remaining seven links: Pennsylvania Avenue eastbound between 18th and 17th Streets, K Street both eastbound and westbound between 18th and 17th Streets, H Street eastbound between 17th and 16th Streets, 16th Street southbound between K and L Streets, 20th Street between K Street and Pennsylvania Avenue, and 21st Street between K Street and Pennsylvania Avenue. Each of these links yielded from two to five data points.

The data were initially analyzed using the previously described program POST. This program computes the mean and standard deviation for the detector data and observed data and then applies three statistical tests. As noted earlier, three tests were employed rather than one to permit different types of comparisons to be drawn, and also to allow for varying assumptions concerning the nature of data. The three tests employed are:

- Two-Sample Student's "t" test;
- Mann-Whitney "U" test, and
- Kolmogorov-Smirnov two-sample test.

Details of the statistical tests are given in the COMPUTER PROCESSING chapter and in Appendix B. An example of the computer output is shown on Figure 14.

Once the comparisons between detector and observed values were conducted, studies were made for both volume and speed using the detector data only comparing the Q1 position with the Q2 position, the Q1 position with the Q3 position, and the

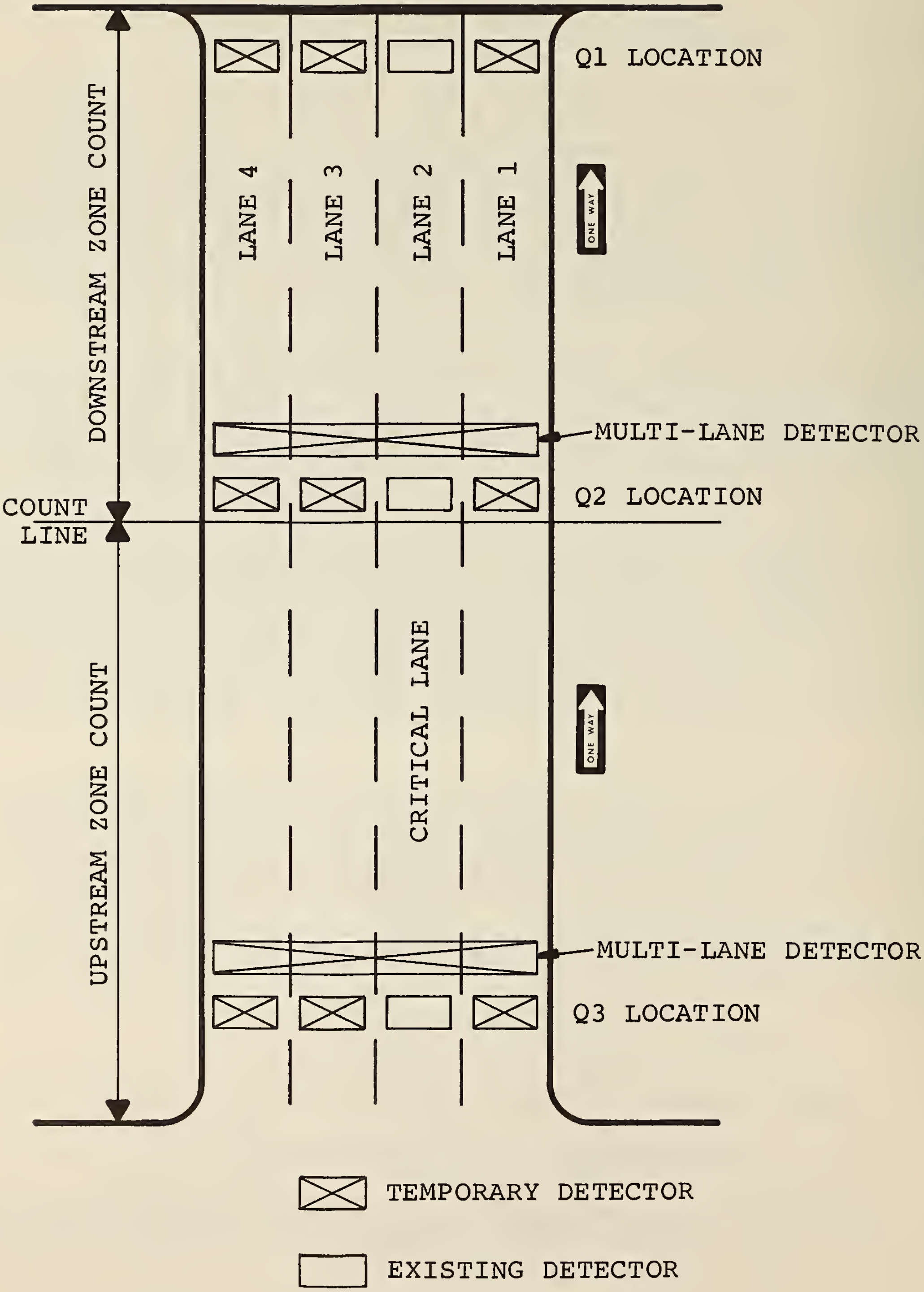

Figure 13. Typical instrumented link.

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              STATISTICAL COMPARISON OF A MEASURE OF EFFECTIVENESS FOR CONDITIONS "A" AND "B"

          CONDITION A : PULSE DATA - APRIL 15
          CONDITION B : FIELD DATA - APRIL 15
          COMPARISONS MADE BY LINK AGGREGATE

          STATISTICAL SAMPLE SIZE USED :  65

          BASIS OF COMPARISONS :  VOLUME AND QUEUE     BY AGGREG.


  LINK /     CONDITION A          CONDITION B         DIFFERENCE      T TEST        U TEST      KOLMOGOROV-SMIRNOV
  OBSERV.    MEAN     STDV        MEAN     STDV        OF MEANS         T             U/Z        T+  T-  T2  SIG.
  -------  -------  -------     -------  -------     ----------   ----------    ----------    ------------  ----
   I 1        8.3      3.8         8.2      3.8         0.9 %     0.115          -0.123          5   4   5
   I 2       11.3      3.6        10.6      3.8         6.5 %     1.098          -0.936          0   6   6
   I 3       10.4      3.5         9.8      3.4         5.9 %     0.982          -1.267          2   8   8
   I 4        6.5      3.6         6.1      3.5         6.1 %     0.618          -0.629          1   5   5
   I T       36.5     10.5        34.7     10.8         5.0 %     0.942          -1.236          1   9   9
   I C       11.3      2.5        10.6      3.8         6.2 %     1.136          -1.008          3  10  10
  OG 1        2.1      2.0         3.1      2.2       -39.5 %     2.813 ***      -2.771 ***     14   0  14  <*
  OG 2        1.6      1.5         0.0      0.0       200.0 %     8.671 ***      -6.810 ***      0  45  45  >***
  OG T        3.7      2.1         3.1      2.2        17.5 %     1.568          -1.739          1  12  12
  OM 1        2.4      2.3         2.1      1.9        15.1 %     0.906          -0.519          2  11  11
  OM 2        2.4      1.9         0.0      0.0       200.0 %    10.384 ***      -7.869 ***      0  52  52  >***
  OM T        4.8      2.9         2.1      1.9        80.0 %     6.350 ***      -5.541 ***      0  29  29  >***
  OA 1        2.2      2.7         1.1      1.5        65.7 %     2.969 ***      -2.016 *        0  13  13
  OA 2        2.0      1.8         0.0      0.0       200.0 %     8.966 ***      -6.507 ***      0  43  43  >***
  OA T        4.2      2.5         1.1      1.5       116.3 %     7.834 ***      -7.043 ***      0  39  39  >***
  21 1        4.3      2.4         2.6      1.9        50.1 %     4.465 ***      -4.046 ***      0  20  20  >***
  21 2        0.0      0.0         6.7      3.2      -200.0 %    17.047 ***      -9.685 ***     64   0  64  <***
  21 3        2.5      1.8         3.4      2.5       -28.1 %     2.192 *        -1.774         16   4  16  <**
  21 T        6.8      3.4        12.6      5.3       -59.8 %     7.429 ***      -6.100 ***     37   0  37  <***
  21 C        4.7      2.1         6.7      3.2       -35.5 %     4.298 ***      -3.967 ***     23   0  23  <***
  OG 1        0.0      0.0         4.5      2.0      -200.0 %    18.517 ***      -9.534 ***     63   0  63  <***
  OG 2        6.2      3.4         0.7      1.0       150.0 %    10.094 ***      -7.341 ***      0  45  45  >***
  OG T        5.2      3.4         5.3      2.5        -1.8 %     0.175          -0.105          8   7   8
  OM 1        0.0      0.0         0.4      0.7      -200.0 %     4.451 ***      -3.027 ***     20   0  20  <***
  OM 2        5.5      3.7         0.2      0.4       184.0 %    12.498 ***      -8.519 ***      0  54  54  >***
  OM T        5.5      3.7         0.6      0.9       160.6 %    11.260 ***      -7.913 ***      0  48  48  >***
  OA 1        0.0      0.0         0.2      0.4      -200.0 %     3.511 ***      -1.665         11   0  11
  OA 2        8.0      3.8         0.1      0.4       193.2 %    16.600 ***      -9.799 ***      0  63  63  >***
  OA T        8.0      3.8         0.3      0.6       185.2 %    16.139 ***      -9.725 ***      0  60  60  >***
  KW 1        5.1      2.4         4.5      2.5         4.0 %     0.464          -0.519          1   4   4
  KW 2        5.5      2.4         5.1      2.1         7.8 %     1.046          -0.831          0   7   7
  KW T       10.6      3.8        10.0      4.0         6.0 %     0.899          -0.882          1   6   6
  KW C        5.4      1.8         5.1      2.1         6.7 %     1.086          -1.159          3  10  10
  OG 1        3.5      2.6         4.3      1.8       -20.8 %     2.046 *        -2.567 ***     22   2  22  <***
  OG 2        0.0      0.2         0.1      0.3       -85.7 %     0.916          -0.447          3   0   3
  OG T        3.5      2.6         4.4      2.0       -21.7 %     2.144 *        -2.661 ***     22   1  22  <***
  OM 1        0.2      0.5         0.1      0.3        60.0 %     1.147          -0.487          0   3   3
  OM 2        0.6      1.2         0.2      0.5       135.5 %     2.753 ***      -1.934          0  12  12
     T        0.8      1.2         0.3      0.6        93.3 %     4.193 ***      -2.533 **       0  15  15  >*
  OA 1        0.5      0.7         1.1      0.8       -73.4 %     4.504 ***      -4.109 ***     25   0  25  <***
  OA 2        1.4      1.5         1.8      1.2       -26.6 %     1.478          -2.873 ***     24   5  24  <***
  OA T        1.8      1.5         2.5      1.4       -43.9 %     3.601 ***      -4.060 ***     26   3  26  <***
  KF 1        7.0      2.2         6.3      2.2        10.4 %     1.767          -1.928          1  10  10
  KF 2       12.6      2.5        11.6      3.3         8.5 %     1.894          -1.916          1  13  13
  KF T       19.6      4.1        17.8      4.2         9.2 %     2.354 **       -2.379 **       2  17  17  >**
  KF C       10.8      2.4        11.6      3.3        -6.9 %     1.538          -1.248         14   3  14  <*
  OG 1        2.1      2.2         4.2      2.6       -66.3 %     4.969 ***      -4.898 ***     28   0  28  <***
  OG 2        0.8      0.5         3.6      2.4      -127.7 %     8.810 ***      -7.841 ***     44   0  44  <***
  OG T        2.9      2.8         7.8      4.3       -91.2 %     7.944 ***      -7.236 ***     38   0  38  <***
  OM 1        3.5      2.6         3.1      1.4        12.5 %     1.146          -0.331          9  18  18  >***
```

Figure 14. Typical POST analysis output.

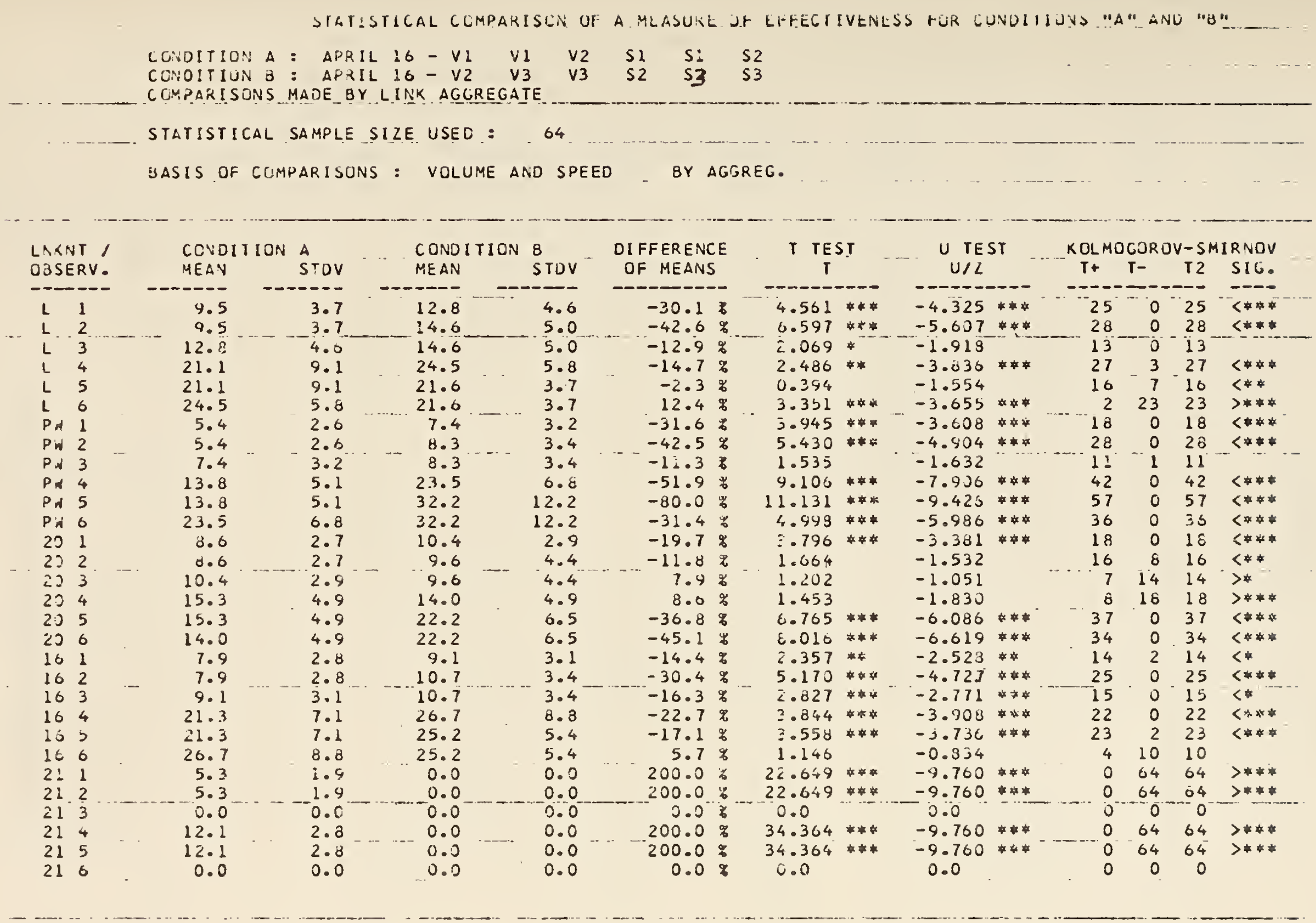
Q2 position with the Q3 position. These tests were made using a variation of the program POST that was previously described. An example of the output is shown in Figure 15.

Because both of the above techniques compared mean values generated over a two hour period, a cycle by cycle analysis was also performed on the data using the programs VPLOTS and QPLOTS. These programs produce scatter diagrams plotting the detector values versus the observed values on a cycle by cycle basis. An example of one of the scatter diagrams is shown on Figure 16.

Comparisons of Observed and Detector Counts

A careful study of the POST output shows that in general, there is close agreement between the observed and detector counts. A summary of these values is shown on Table 17. The samples, however, exhibit a fairly high standard deviation. Typical values would be a count of 10 with a standard deviation of 3. If the traffic had been constant throughout the data collection period, this would have been an item of considerable concern. Since the first data were collected at approximately 7:00 a.m., however, and the last data collected at 9:00 a.m., there was a significant change in the traffic flow in the study area. The early morning traffic was lighter with frequent counts of three or less vehicles per lane per cycle, while at the end of the data collection period, the traffic was generally congested and most lanes were carrying more than 15 vehicles per cycle. When using a mean value to describe this range, a fairly high standard deviation is expected. It is encouraging to note that the standard deviation of the detector data compares favorably with the standard deviation of the observed values.

The review of this data also suggested that the detectors were consistently overcounting when compared to the observed values. This was expected since it is possible for one vehicle to pass over the zone of detection of two detectors when changing lanes. To quantify the magnitude of the detector bias, a scatter diagram was prepared plotting the detector counts versus the observed counts. This plot is shown as Figure 17. When a least squares regression line was fit to the data of the form Y = A + Bx, the coefficients were found to be A = 0.22, B = 1.02 indicating a linear fit with the A coefficient (the Y intercept) closed to zero and the B coefficient (the slope) close to one.



```
                  STATISTICAL COMPARISON OF A MEASURE OF EFFECTIVENESS FOR CONDITIONS "A" AND "B"

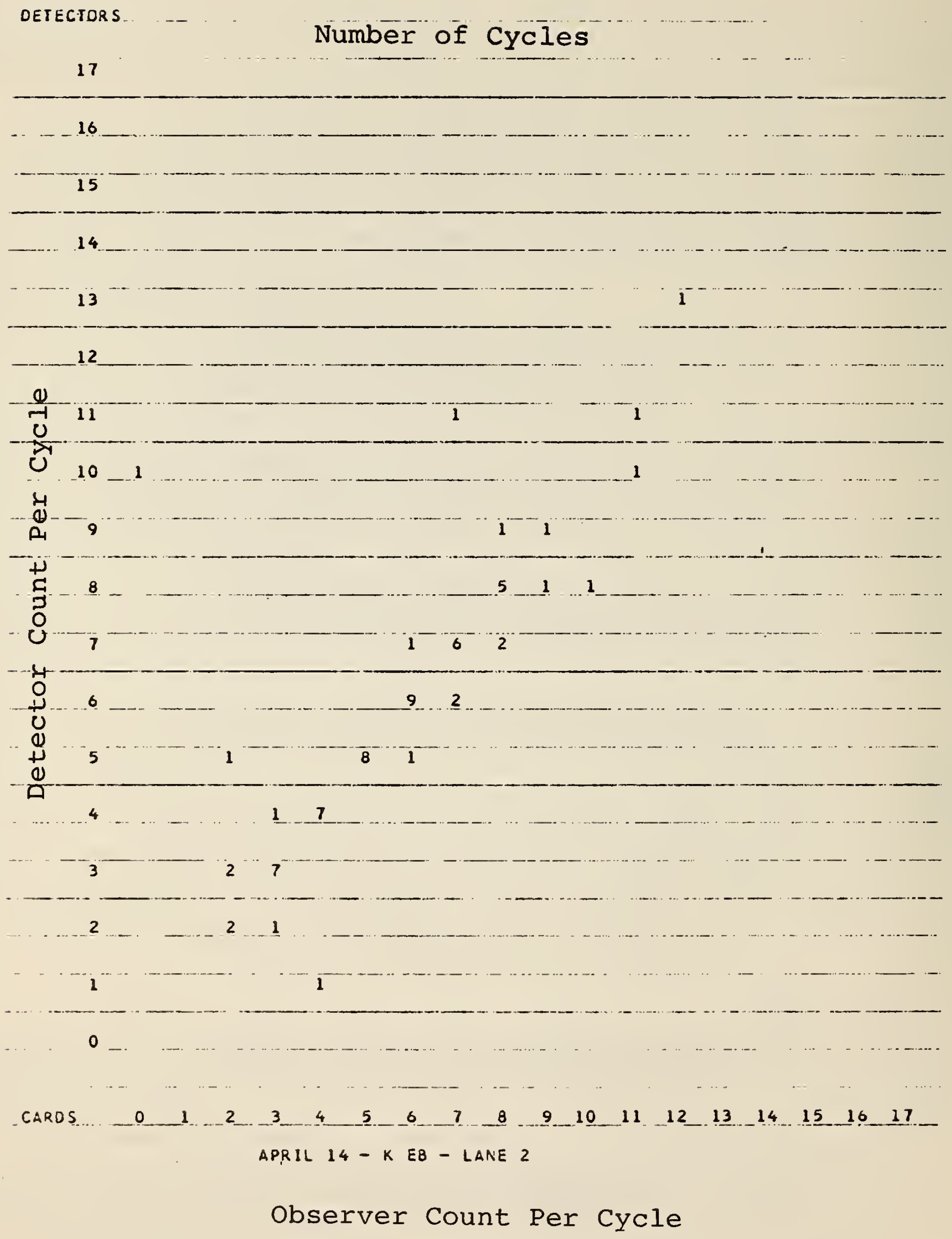
          CONDITION A :  APRIL 16 - V1    V1    V2    S1    S1    S2
          CONDITION B :  APRIL 16 - V2    V3    V3    S2    S3    S3
          COMPARISONS MADE BY LINK AGGREGATE

          STATISTICAL SAMPLE SIZE USED :    64

          BASIS OF COMPARISONS :  VOLUME AND SPEED    BY AGGREG.

 LNKNT /      CONDITION A         CONDITION B       DIFFERENCE     T TEST        U TEST       KOLMOGOROV-SMIRNOV
 OBSERV.     MEAN      STDV      MEAN      STDV      OF MEANS        T            U/Z          T+   T-   T2   SIG.
 -------   -------   -------   -------   -------   ----------   ----------   ----------    ------------   ----
  L  1        9.5       3.7      12.8       4.6      -30.1 %     4.561 ***   -4.325 ***     25    0   25   <***
  L  2        9.5       3.7      14.6       5.0      -42.6 %     6.597 ***   -5.607 ***     28    0   28   <***
  L  3       12.8       4.6      14.6       5.0      -12.9 %     2.069 *     -1.918         13    0   13
  L  4       21.1       9.1      24.5       5.8      -14.7 %     2.486 **    -3.836 ***     27    3   27   <***
  L  5       21.1       9.1      21.6       3.7       -2.3 %     0.394       -1.554         16    7   16   <**
  L  6       24.5       5.8      21.6       3.7       12.4 %     3.351 ***   -3.655 ***      2   23   23   >***
  PW 1        5.4       2.6       7.4       3.2      -31.6 %     3.945 ***   -3.608 ***     18    0   18   <***
  PW 2        5.4       2.6       8.3       3.4      -42.5 %     5.430 ***   -4.904 ***     28    0   28   <***
  PW 3        7.4       3.2       8.3       3.4      -11.3 %     1.535       -1.632         11    1   11
  PW 4       13.8       5.1      23.5       6.8      -51.9 %     9.106 ***   -7.906 ***     42    0   42   <***
  PW 5       13.8       5.1      32.2      12.2      -80.0 %    11.131 ***   -9.425 ***     57    0   57   <***
  PW 6       23.5       6.8      32.2      12.2      -31.4 %     4.998 ***   -5.986 ***     36    0   36   <***
  20 1        8.6       2.7      10.4       2.9      -19.7 %     3.796 ***   -3.381 ***     18    0   18   <***
  20 2        8.6       2.7       9.6       4.4      -11.8 %     1.664       -1.532         16    8   16   <**
  20 3       10.4       2.9       9.6       4.4        7.9 %     1.202       -1.051          7   14   14   >*
  20 4       15.3       4.9      14.0       4.9        8.6 %     1.453       -1.830          8   18   18   >***
  20 5       15.3       4.9      22.2       6.5      -36.8 %     6.765 ***   -6.086 ***     37    0   37   <***
  20 6       14.0       4.9      22.2       6.5      -45.1 %     8.016 ***   -6.619 ***     34    0   34   <***
  16 1        7.9       2.8       9.1       3.1      -14.4 %     2.357 **    -2.523 **      14    2   14   <*
  16 2        7.9       2.8      10.7       3.4      -30.4 %     5.170 ***   -4.727 ***     25    0   25   <***
  16 3        9.1       3.1      10.7       3.4      -16.3 %     2.827 ***   -2.771 ***     15    0   15   <*
  16 4       21.3       7.1      26.7       8.8      -22.7 %     3.844 ***   -3.908 ***     22    0   22   <***
  16 5       21.3       7.1      25.2       5.4      -17.1 %     3.558 ***   -3.736 ***     23    2   23   <***
  16 6       26.7       8.8      25.2       5.4        5.7 %     1.146       -0.834          4   10   10
  21 1        5.3       1.9       0.0       0.0      200.0 %    22.649 ***   -9.760 ***      0   64   64   >***
  21 2        5.3       1.9       0.0       0.0      200.0 %    22.649 ***   -9.760 ***      0   64   64   >***
  21 3        0.0       0.0       0.0       0.0        0.0 %     0.0          0.0            0    0    0
  21 4       12.1       2.8       0.0       0.0      200.0 %    34.364 ***   -9.760 ***      0   64   64   >***
  21 5       12.1       2.8       0.0       0.0      200.0 %    34.364 ***   -9.760 ***      0   64   64   >***
  21 6        0.0       0.0       0.0       0.0        0.0 %     0.0          0.0            0    0    0
```

Figure 15. Typical link detector comparison output.



DETECTORS	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
17																		
16																		
15																		
14																		
13													1					
12																		
11								1				1						
10	1											1						
9									1	1								
8									5	1	1							
7							1	6	2									
6							9	2										
5			1			8	1											
4				1	7													
3			2	7														
2			2	1														
1					1													
0																		

Figure 16. Typical scatter diagram plot.

Table 17. Comparison of mean observed and detector volumes. (vehicles per cycle)

Link	Lane 1			Lane 2			Lane 3		
	Volume Q_1	Volume Q_2	Volume Observed	Volume Q_1	Volume Q_2	Volume Observed	Volume Q_1	Volume Q_2	Volume Observed
L Street April 14	7.8	9.3	7.9	10.5	12.2	11.8	10.4	10.4	9.2
15	7.2	8.3	8.2	9.4	11.3	10.6	N.A.	10.4	9.8
16	7.0	6.6	6.3	9.5	12.8	12.7	9.0	10.2	9.5
17	8.2	8.7	8.1	5.9	N.A.	6.8	5.2	5.9	5.6
18	8.1	9.1	8.7	8.2	9.9	9.8	6.0	7.1	7.2
Penn. Ave. WB April 14	4.5	5.7	6.4	N.A.	N.A.	N.A.	4.1	4.5	5.2
16	4.4	N.A.	3.7	5.4	7.6	6.2	3.8	4.4	4.6
17	5.0	5.6	4.8	6.1	8.0	7.2	N.A.	4.5	3.8
18	N.A.	5.9	8.4	N.A.	N.A.	N.A.		4.9	5.2
Penn Ave. EB April 17	5.4	5.7	4.1	9.2	11.8	11.0	4.5	N.A.	4.8
18	5.7	6.0	4.9	9.7	11.8	10.2	4.9	N.A.	5.2
16th Street April 14	2.1	2.1	1.8	7.6	N.A.	8.2	5.1	5.4	4.7
16	2.4	1.0	0.7	7.9	9.1	8.9	5.8	6.2	6.1
21st Street April 14	3.0	4.1	2.6	5.6	N.A.	6.3	3.1	2.7	3.4
15	2.9	4.3	2.6	5.4	N.A.	6.7	3.6	2.5	3.4
16	2.9	N.A.	2.4	5.3	N.A.	6.0	3.6	2.8	3.3
K Street WB April 14	4.8	4.5	4.4	5.6	5.8	5.6			
15	N.A.	5.1	4.9	5.4	5.5	5.1			
K Street EB April 14	N.A.	6.9	6.9	6.7	N.A.	11.0			
15	N.A.	7.0	6.3	7.1	12.6	11.6			

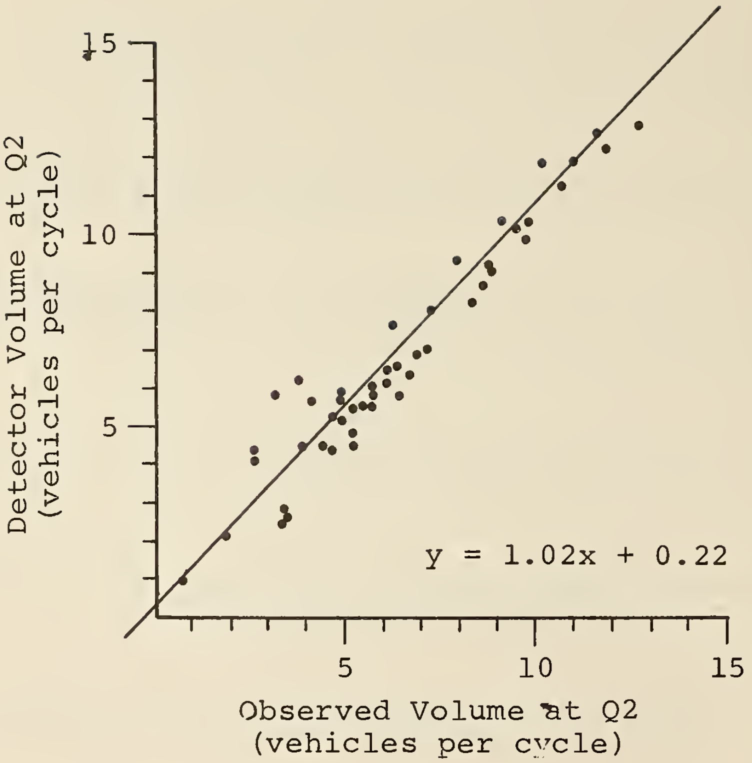


Figure 17. Detector to observed data comparisons (all sets).

Because there is considerable interest in using a critical lane detection scheme, data from lane 2 were then plotted and a least squares regression line fit as shown on Figure 18. This method produced better results with an A value of 0.71 and a B value of 0.99. The correlation coefficient for this fit was 0.98. From this analysis it is concluded that the detector has an average bias of approximately seven tenths of a vehicle per cycle and produces an accurate measure of the observed traffic at the Q2 location.

Comparisons of Detector Counts by Location

As previously described, three longitudinal locations within the link were considered, the Q1, Q2, and Q3 locations. There were two items of concern; how lane counts varied between locations, and how measures of speed (and its inverse occupancy) varied.

Comparisons of the detector count data shows that in general the Q1 location exhibited a lower count than the Q2 location, and the Q3 location produced widely fluctuating count measures. Since both the observed counts and the Q1 location counts produced values less than the Q2 location detectors, a plot of the Q1 location was made against the observed values to see if there was a correlation. This plot is shown on Figure 19. The least squares regression coefficient had an A coefficient of 1.25 and a B coefficient of 0.71 indicating a poor agreement.

This is confirmed by the statistical comparisons made by the POST program that compares the detector count at Q1 with the detector count at Q2. For the 18 comparisons of data in the critical lane (lane 2), 16 reject the hypothesis at the 5% level of significance or greater that the two means are equal using the t test. Of these 16, the Kolmogorov-Smirnov test showed that 14 mean comparisons indicated that Q1 was less than Q2 at the 5% or greater level of significance.

As an additional evaluation, a "t" test was performed on the difference of means of the observed values at Q2 with the detector values at Q1. Of the 59 comparisons made, 34 comparisons reject the null hypothesis that the means are equal at the 5% level of significance. It is concluded therefore, that on a lane-specific basis, the Q1 counts differ from the observed counts at Q2.

A major reason for this poor agreement is thought tc be caused by lane changing within 200 feet of the intersection. Particularly at relatively low volumes, vehicles were

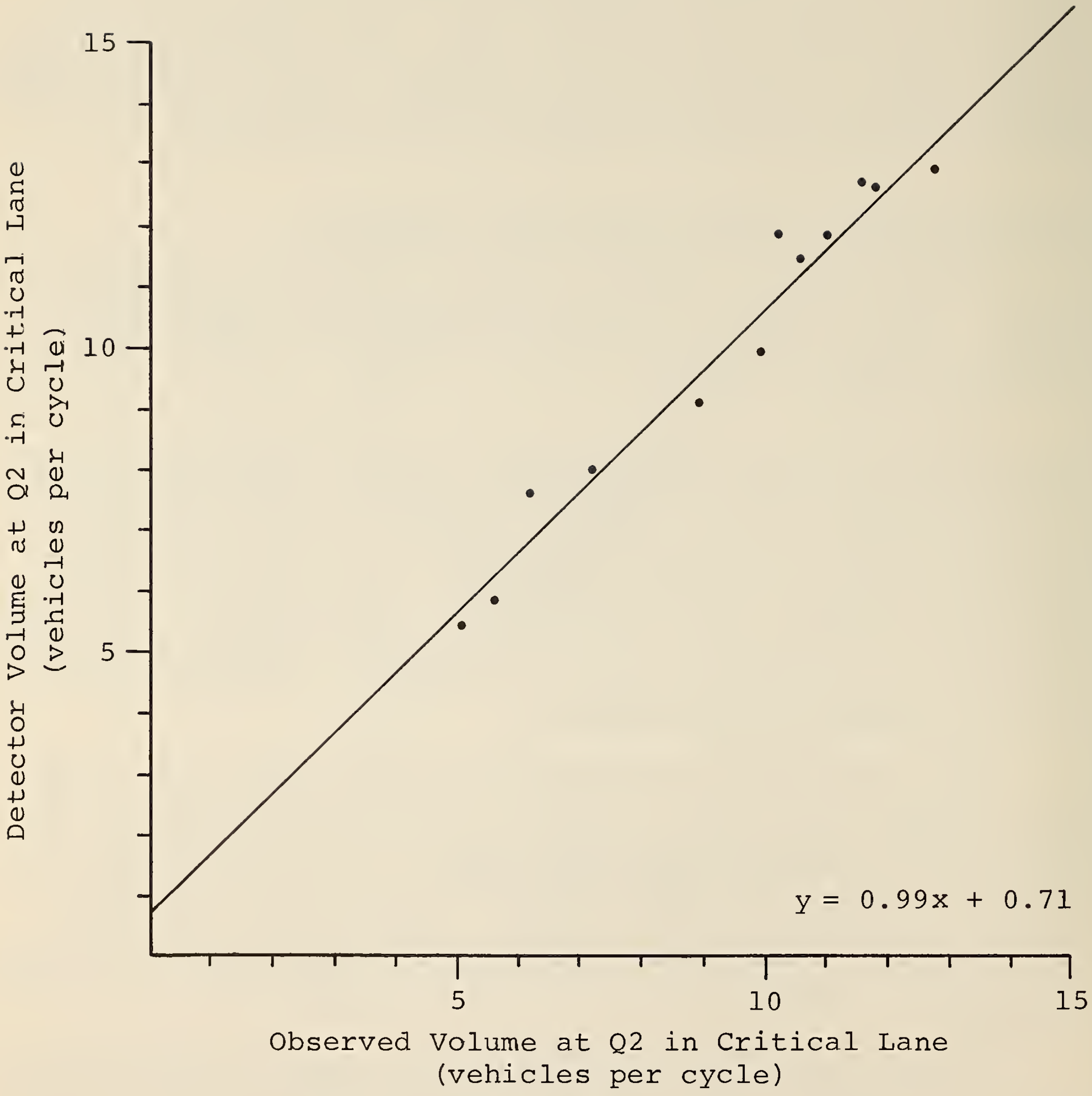


Figure 18. Critical lane detector to observed data comparison (all sets).

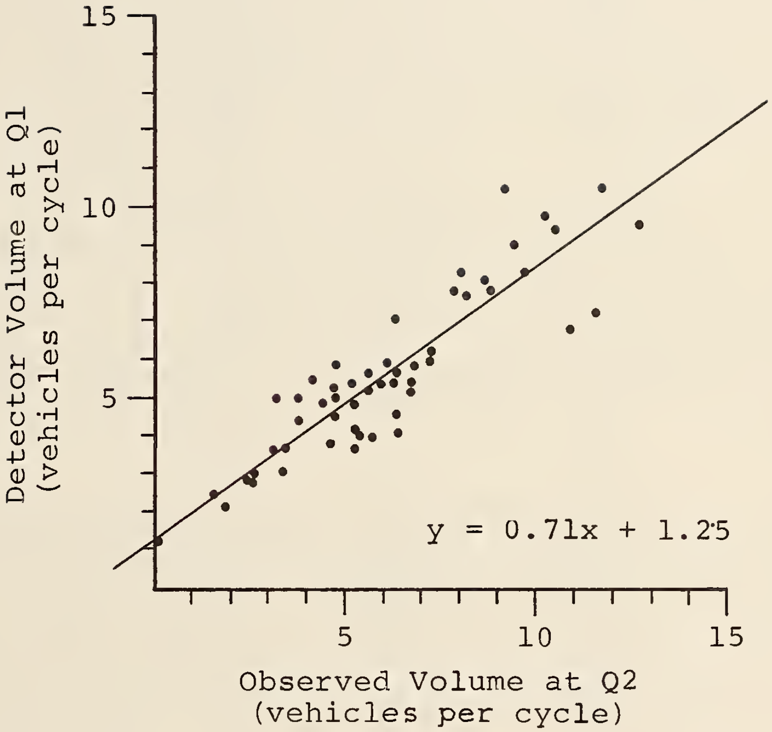


Figure 19. Volume comparisons at Q1 location.

observed to favor the middle lanes to avoid mid-block marginal friction then fan out into all lanes at the intersection. In support of this hypothesis, higher counts were frequently found in the curb lanes at the Q1 location particularly where turning movements were prevalent.

When the statistical comparisons generated by POST for the count at Q2 with the count at Q3 are evaluated, 16 comparisons are available for the critical lane. At the 5% level of significance or higher, these tests show that 11 comparisons are significantly different when subjected to the null hypothesis that the two mean values are equal. There is no undercounting or overcounting trend with these comparisons, however, the Kolmogorov-Smirnov test showed that where the differences were apparent, four were overcounting and four were undercounting at the 5% level of significance.

Because of the differences found between both the Q1 and Q3 detector counts, when compared to the Q2 detector counts, and because the Q2 detector counts have a good correlation with the observed counts, it is concluded that the UTCS practice of summing the counts at the three locations and using a mean value should be discontinued. Instead, the volume measures should be made using only the counts from the Q2 detector.

Of more serious concern in the examination of the data produced by the detectors in the three different positions is the estimate of speed (occupancy). A summary of the average speed data is shown on Table 18. General conclusions reached from a study of this table are that the Q1 location consistently produces a lower value of speed than that experienced at the Q2 location, and that the speed measures at the Q3 location produce erratic results. The speed measures at the Q1 location are low because of the propensity for stopped vehicles to be located over the detector thus producing low speed readings.

The behavior of the Q3 detector is more difficult to analyze. Where the detector was far enough downstream to be beyond the zone of acceleration of vehicles entering the link, the speed measures tended to be similar to that measured at Q2. This is thought to be the case, for example, on L Street although the slightly lower average speeds would seem to indicate that the average traffic flow had not yet reached its free flow speed. The erratic speeds experienced at the other locations are thought to be caused by vehicles turning onto the link and clipping the zone of detection thus generating a very short pulse (high speed), and at the opposite extreme, vehicles accelerating but still traveling at slow speeds while

Table 18. Average speeds - total traffic.[1]

Street	Date	Detector		
		Q1	Q2	Q3
"L" St.	4/14	19.8	23.1	21.9
	4/15	20.4	24.7	22.1
	4/16	21.1	24.5	21.6
	4/17	13.5	N.A.	14.5
	4/18	16.9	18.8	18.3
Pa. Ave. WB	4/14	N.A.	N.A.	28.0
	4/16	13.8	23.5	32.2
	4/17	15.0	21.2	14.8
	4/18	13.8	22.7	16.1
16th St.	4/14	22.4	N.A.	25.0
	4/16	21.3	26.7	25.2
21st St.	4/14	12.8	N.A.	22.9
	4/15	12.0	N.A.	22.0
	4/16	12.1	N.A.	N.A.
	4/17	14.2	14.0	N.A.
	4/18	13.3	12.1	N.A.
"K" St. EB	4/14	12.5	N.A.	23.9
	4/15	13.7	19.4	22.0
"K" St. WB	4/14	16.1	25.8	10.5
	4/15	14.8	24.9	11.9
20th St.	4/16	15.3	14.0	22.2
Pa. Ave. EB	4/17	16.6	20.7	13.5
	4/18	16.8	20.6	16.9
"H" St.	4/17	20.0	20.6	4.9
	4/18	17.7	21.3	10.1

[1] All data taken from critical lanes. Data for all vehicles for approximately 1-1/2 hour period during AM peak.

crossing the detector. These hypotheses tend to be supported by the high standard deviations frequently found with the Q3 speed data.

From the analysis of the speed data, it is concluded that speed measurements should be made at the Q2 location. The current UTCS practice of averaging the values generated from the Q1, Q2, and Q3 locations should be discontinued. Because this is the identical conclusion reached from a study of the volume measures, the general conclusion is drawn that one detector should be used for both volume and speed (occupancy) measures and that this detector should be located 210 feet upstream from the stop line.

Two additional points must be made in support of the above conclusion. First with respect to the counting accuracy of the Q1 detector, the research team feels that the lower counts experienced at Q1 are a result of lower lane flow rates and that the detectors are accurately reflecting this. The rejection of the Q1 detector for volume is based on the fact that a detector close to the stop line is not a measure of demand. The second concerns the specific location of the Q2 detector. Because the research focused on the UTCS system, conclusions are drawn that are specific to that system. The research team suggests, however, that the location of the single detector can be located within an area of the link defined as downstream from the zone of acceleration and lane changing found in the link entrance area where the Q3 detector is located; and upstream from the area of frequent queue buildup. Specifically this area is defined as more than 200 feet upstream from the stopline and 200 feet downstream from the link entrance. Additional research is suggested to investigate the optimum location of the detector within this region.

Cycle by Cycle Comparisons

To examine the data on a micro basis, computer output scatter diagrams were used. VPLOT produced the volume diagram and QPLOT produced the queue diagram. The volume data on a cycle specific basis confirms the observations made using the mean values. As can be seen from Table 19, the percent of cycles during which the average detector count agreed with the average observed count improved from 66% to 81% to 90% as the acceptable limits were expanded from one to two or three vehicles respectively. In each of these three cases, a pronounced overcounting bias is evident from 26% to 15% to 7% respectively as the limits are increased. The poorest results were experienced with the

Table 19. Cycle by cycle critical lane count accuracy.

Location	Number of Cycles	Within +/- One Vehicle			Within +/- Two Vehicles			Within +/- Three Vehicles		
		% Over	% Within	% Under	% Over	% Within	% Under	% Over	% Within	% Under
L Street	247	16	73	11	7	89	4	3	95	2
Penn. Ave. WB	117	43	40	17	24	65	11	15	77	8
16th Street	64	39	58	3	27	71	2	9	89	2
K Street EB	65	12	83	5	5	90	5	3	94	3
K Street WB	130	24	66	10	15	80	5	8	90	2
20th Street	64	9	86	5	2	98	0	0	100	0
Penn. Ave. EB	118	37	55	8	24	74	2	14	86	0
AVERAGE		26	66	8	15	81	4	7	90	3

two links on Pennsylvania Avenue which is explained by the large number of lane changes experienced in the vicinity of the Q2 detector on both links.

Two suggestions are made that would improve the accuracy of the detector count. First a simple correction of seven tenths of a vehicle per lane per cycle would improve the estimate. Secondly, the research team feels that a reduction in the width of the loop detector in the field would reduce the probability of a vehicle being sensed by two adjacent detectors simultaneously. Although the improvement in measurement would not justify the extensive cost involved in reconfiguring the loops in UTCS, other surveillance systems could benefit from using a narrower loop.

The cycle by cycle queue analysis was conducted in a manner similar to the volume analysis. The algorithm used to develop the detector queue is actually a count-in count-out lane-specific link content algorithm and is described in the COMPUTER PROCESSING chapter. The observed values are also link content and lane-specific. The downstream zone was from the Q2 detector to the stop line. An upstream link content was also calculated using detector data and observations from the field. Data points were calculated and observed at the beginning of green, 20 seconds after the begining of green, and at the end of green. Figure 20 shows a typical scatter diagram for the observed link content versus the detector generated link content at the beginning of green.

Because the review of the scatter diagrams showed that the downstream queue at the beginning of green appeared to give the best results, the majority of the analysis concentrated on this set of data. A summary of average downstream queue data is shown on Table 20.

As with the volume data, the estimated queue when compared to the observed queue improved from 51% to 74% to 86% as the acceptable limits were expanded from one to two to three vehicles respectively. And also similar to the volume data, a pronounced bias is evident from 32% to 18% to 9% underestimation respectively as the limits are increased. The magnitude of the problem is appreciated when it is realized that the maximum length of a queue between the stop line and the Q2 detector is eight or nine vehicles.

When the technique was expanded to consider the queue at 20 seconds after the start of green and at the queue at the end of green. The scatter diagrams were simply random. Similarly, when the queues were plotted for all three time periods at the upstream location, the diagrams were also random.

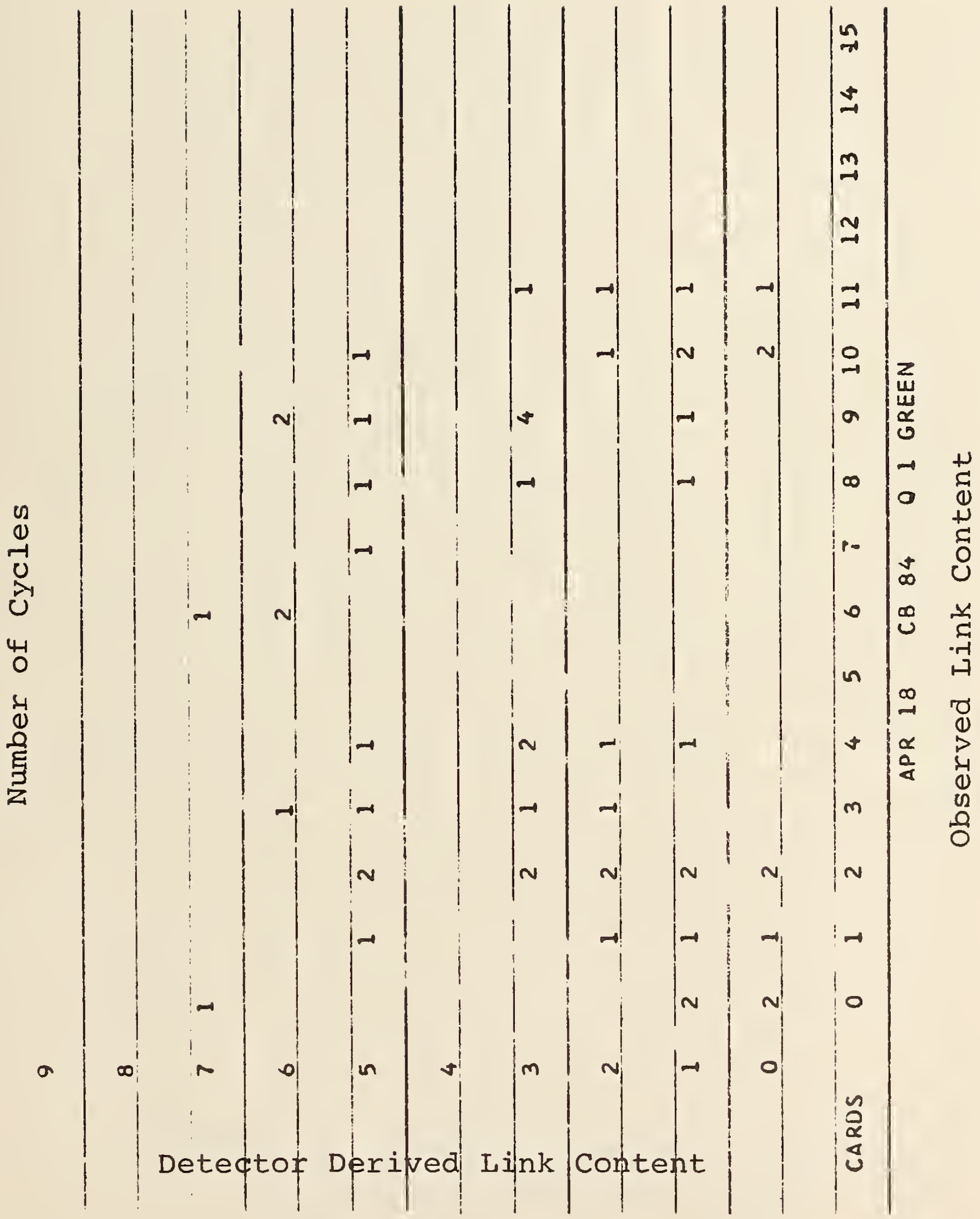


Figure 20. Observed versus detector generated link content.

Number of Cycles

Detector Derived Link Content

9																
8									1							
7						2				1						
6			1	1												
5	1				2											
4																
3		2	1	1				1	1	1						
2	1	2	2	1		1	1	2								
1	2		1	1			1			1						
0		1	4	2	3	1	2	5	2	1						
CARDS	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

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Observed Link Content

Figure 20. Observed versus detector generated link content (continued).

Number of Cycles

Detector Derived Link Content

CARDS	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
9																
8																
7		1			1		1				1					
6	1															
5	1					1		1			1	1				
4																
3	3				3					1						
2	2	3		1		1				1		1				
1	6	2	1				1									
0	5	1				2	1	2	2	3	1					

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Observed Link Content

Figure 20. Observed versus detector generated link content (continued).

Table 20. Cycle by cycle queue accuracy.

Location	Number of Cycles	Within +/- One Vehicle			Within +/- Two Vehicles			Within +/- Three Vehicles		
		% Over	% Within	% Under	% Over	% Within	% Under	% Over	% Within	% Under
L Street	247	13	55	32	7	72	21	4	80	16
K Street WB	130	4	53	43	0	80	20	0	95	5
K Street EB	65	6	68	26	3	85	12	3	91	6
20th Street	181	23	43	34	8	68	24	6	76	18
16th Street	64	14	56	30	6	81	13	5	86	9
21st Street	64	0	47	53	0	72	28	0	94	6
Penn. Ave. EB	64	41	50	9	22	73	5	13	84	3
Penn. Ave. WB	117	37	44	19	23	62	15	12	79	9
H Street	117	15	46	39	9	70	21	3	87	10
AVERAGE		17	51	32	8	74	18	5	86	9

Because the research team found that count-in, count-out estimates of queue or link content did not yield results sufficiently accurate to be used as an estimate of link content necessary for third generation CIC/QMC, an alternative algorithm is suggested for development and evaluation. This algorithm uses detectors that are long enough to cover a gap between vehicles when a stoppage exists yet short enough so that moving vehicles will not be declared a stoppage. A previous study[1] indicates that a 25 foot long detector would identify 99% of all stoppages. In order to reduce errors resulting from declaring a stoppage when in fact the vehicles are still moving but closely spaced, a time element is introduced so that the detector must be "on" for a certain time period, such as six seconds, before a stoppage is declared.

The basic assumption with this technique is that the identification of a stoppage in a particular area of the link is equivalent to identifying a queue within that same area of the link. To illustrate this algorithm, assume that the average distance headway between vehicles stopped in a queue is 25 feet and that the 99th percentile gap between stopped vehicles is also 25 feet. With one detector located 75 feet downstream from the upstream intersection and a second detector located 75 feet downstream from the first detector, a queue within 150 feet of the intersection can be estimated to within plus or minus one vehicle as follows:

a. When a stoppage is detected on the downstream detector and there is no stoppage on the upstream detector, the technique estimates that there are two vehicles in the queue when in fact there are between one and three.

b. When there is a stoppage detected on both detectors, the technique estimates that there are five vehicles in the queue when in fact the queue ranges from four to six vehicles.

Although this example uses two detectors and a coverage zone beginning at the upstream intersection, the technique can cover any segment of the link if a detector is added to identify the upstream limit. By increasing the distance between detectors to 125 feet, a queue within 250 feet of an intersection can be estimated to an accuracy of plus or minus two vehicles.

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[1] Benioff, B., and Moghaddas, A.; Peat, Marwick, Mitchell & Co., STOPPED VEHICLE SPACING ON FREEWAYS, Traffic Engineering, February 1970.

Speed Analysis

The basic parameter that can be measured with an instrumented surveillance system is a spot speed. Two alternative schemes are available using inductive loop detectors. With the first scheme, one detector is used and speed is computed by dividing the sum of the average effective vehicle length plus the effective detector length by the time of the pulse.

The second scheme uses two detectors to produce an estimate of speed that is independent of the vehicle length and can be structured to identify the average speed of an accelerating or decelerating vehicle. In this way, the separation between the sensors is divided by the difference in time from when the first detector goes on until the actuation of the second detector. As with the calculation of occupancy, a significant error may be introduced into the calculation of average speed by the utilization of a finite sampling rate. The error due to sampling rate, however, is directly controllable. That is, speed errors are decreased by increasing the sensor sampling rate. The studies conducted in the research effort used only the first technique, the sum of the individual pulse lengths, since this is the method currently programmed in the UTCS software.

Speed, as used in the various algorithms, is in a sense an idealized "free flow speed." A primary situation is the necessity of actually measuring such a quantity on-line, as opposed to using either an hypothesized value or some simple historical measures with some relevant stratification.

Since "free flow speed" is required, several alternative techniques were evaluated to provide a measure. Initially, an effort was made to measure the speed of the first car in a platoon after the light changes with the assumption that its speed is not seriously impeded by downstream congestion.

The second technique utilized the assumption that vehicles preceeded by a time gap of either five or ten seconds were essentially "free flow" vehicles. A third technique was examined using the average speed data initially of all entries and finally using only entries that indicated a speed within a range of 10 mph to 55 mph.

The basic source of data for all three evaluations was the raw history or pulse tapes generated by the UTCS system recording data measured by the temporary loop detectors. The computer programs PULSE1 and AGVS were utilized in interpreting the raw pulse data.

First Vehicle Speed

In the first technique, it was hypothesized that the measurement of speed for the first vehicle in a platoon, assuming it has accelerated fully and has not been impeded downstream, will give an estimate of "free flow speed." To accomplish this, at each detector the pulse length of each vehicle was recorded in 1/32 of a second intervals. These pulse lengths were converted into spot speeds by assuming an average vehicle length. Speeds were then averaged for each cycle, each 4 cycles, 11 cycles and finally, the total 22-cycle period (approximately 1/2 hour).

After the initial effort to measure speed using the first vehicle in a platoon, the technique was discarded for a number of reasons, such as variability of vehicle length, vehicle height, relatively low sampling rate of 32 pulses per second, and this small sample size inherently introduces a large error in the scheme. With an 80 second cycle, a sample size of only 11 was available for 15 minute averages.

Time Gap Speed

In the second technique, vehicle speeds were segregated into those vehicles which had a five-second and ten-second headway preceeding them. As with the total number of vehicles, vehicle speeds were summed and averaged for the five and ten-second gaps each cycle, each 4 cycles, each 11 cycles and for the total period. Finally, a listing of each entry pulse length was made for 5 and 10 second gap vehicles. A summary of the average speeds with 5 and 10 second gaps is shown on Table 21.

A study of the various pulse lengths indicates that there are a number of entries which are so low or so high that they completely distort the overall average speed. Low entries of perhaps 2/32 of a second are caused by vehicles changing lanes as they cross a detector. This entry which equates to 218 mph severely distorts the overall average and is not smoothed out except over a very long time period. Conversely, very long pulse lengths, which equate to speeds of as low as 0.01 mph, were noted over almost all detectors. These entries were particularly prevalent at the downstream detectors (Q1), where vehicles are stopped at the signal. The average speeds at these Q1 detectors were quite low, even after the very long pulse lengths were eliminated. Consequently, a pulse length range of from 8 to 40 was selected as a valid range

Table 21. Average speeds - 5 and 10 second gaps.

Detector-Location	Total All Vehicles	5 second gap		10 second gap	
		All Vehicles	Valid Entries	All Vehicles	Valid Entries
186 Q2) Pa. Ave.	24.43	24.88	25.34	26.44	26.69
195 Q3) WB	35.45	32.43	26.17	30.21	26.14
294 Q3) 21st St.	23.15	23.36	19.49	24.26	18.71
307 Q2) SB	25.13	22.63	23.31	22.48	23.41
332 Q2 Lane 3) L St.	22.35	22.42	21.91	23.48	22.30
362 Q2 Lane 2) EB	27.85	23.68	24.51	24.70	25.20
363 Q3 Lane 2)	24.38	21.38	20.40	21.63	20.35
345 Q3) 16th St.	23.92	24.45	23.18	25.37	22.91
391 Q2) SB Lt.Lane	22.60	23.57	23.86	23.59	23.59
236 Q2) K St.	21.48	20.52	23.39	20.84	23.46
237 Q3) EB Lt.Lane	23.24	23.04	24.78	23.45	24.49
304 Q2) 20th St.	25.66	26.25	25.95	26.19	26.58
291 Q3) NB	27.72	26.24	26.49	26.00	26.00

Note: Valid Entries considered as pulse lengths from 8 to 40 (10.9 to 54.5 mph).

within which to develop an average speed. This range corresponds to a speed range of approximately 10 mph to 54 mph.

Data from two typical detectors at midblock (Q2) and upstream (Q3) are shown in histogram form on Figure 21. These give a fairly representative display of the distribution of the pulse lengths at these locations. Generally, it was found that greater than 80% of the entries fell in a valid speed range (10-54 mph) for those midblock and upstream detectors. At the downstream (Q1) locations, the number of "valid" entries was generally between 25 and 40%.

Additional analysis of the histograms indicates that the range to be considered as valid entries could be narrowed further to perhaps pulse lengths from 8 to 27 units. As seen, there is a rather scattered number of entries beyond this range. A pulse length of 27 corresponds to a speed of approximately 16 mph.

The conclusions which can be drawn from this analysis of pulse lengths are as follows:

- Downstream (Q1) detectors cannot be used to develop an accurate "free flow speed". Typically, as many as 75% of the entries are beyond a range of "valid" entries. The remaining entries yield an average speed significantly lower than the Q2 and Q3 detectors.

- The majority of the Q2 detectors yielded stable data, with at least 80% of the entries within the valid range.

- The Q3 detectors yielded very stable results in most cases. The places where problems developed (a large number of invalid entries) were those detectors which were close to the upstream intersection. This would imply that vehicles completing a turn maneuver and not yet at desired speed were biasing the totals, and that many of these same vehicles were changing lanes at the detectors. At the Q2 and Q3 detectors, speed measurements must be taken in the critical lane. The curb lane does not yield valid results. On L Street, Lane 3 also yielded consistent results. On Pennsylvania Avenue, the left lane also yielded good data.

Once the invalid entries were removed from the overall averages, the average speeds were fairly consistent and within what would be considered a reasonable range. A summary of these values for several of the Q2 and Q3

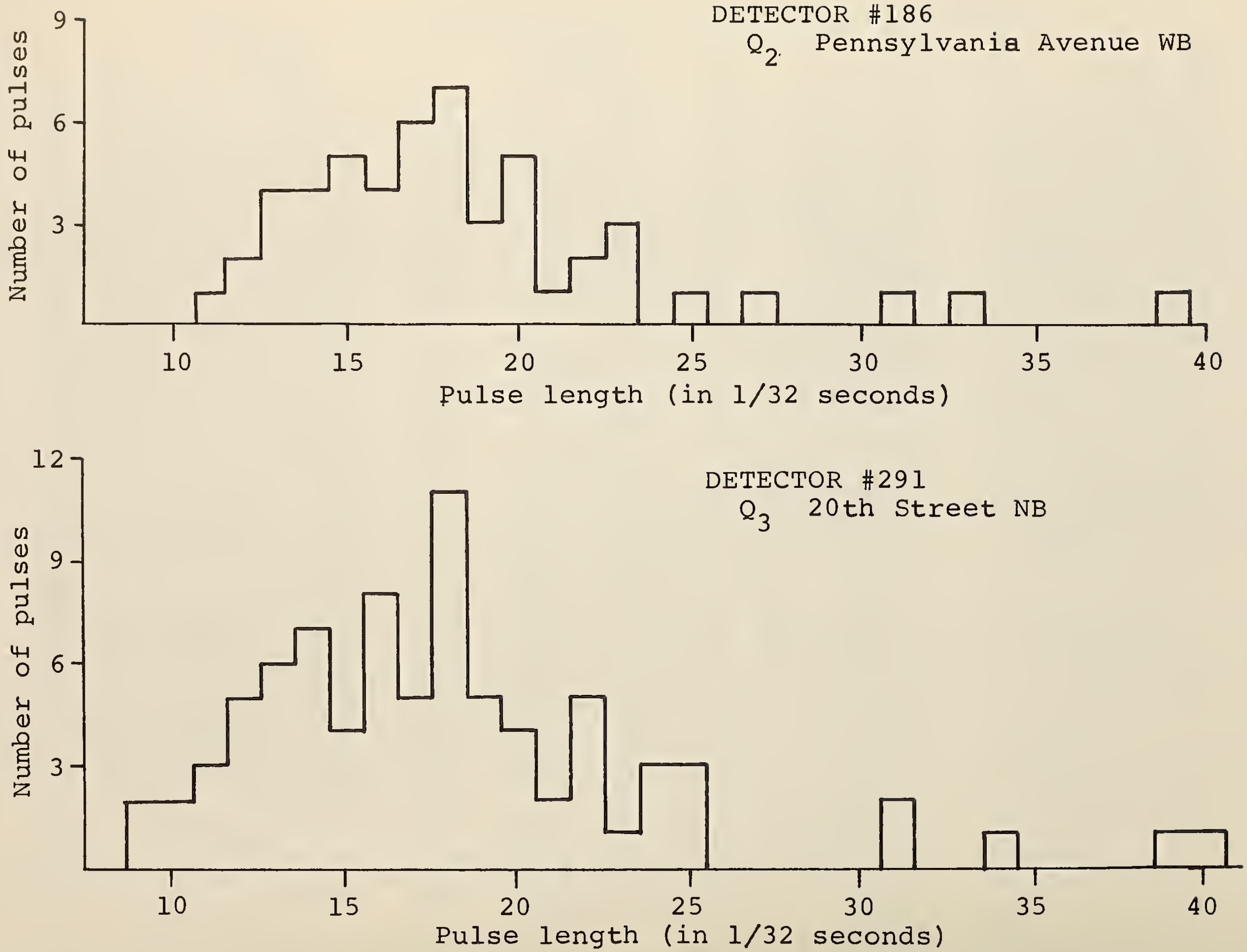


Figure 21. Typical distribution of pulse lengths, (Q_2 and Q_3 detectors).

detectors is shown in Table 21, shown earlier, which also compares the data for vehicles with a 10-second gap, as well as all vehicles in the traffic stream.

It can therefore be concluded that in order to obtain any reasonable estimate of free flow speed, invalid pulse lengths should be eliminated when computing the average. Moreover, there is no advantage in using a 10-second gap requirement to estimate the average speed. The 5-second gap appears to be satisfactory and has the advantage of being totaled over a larger population of values.

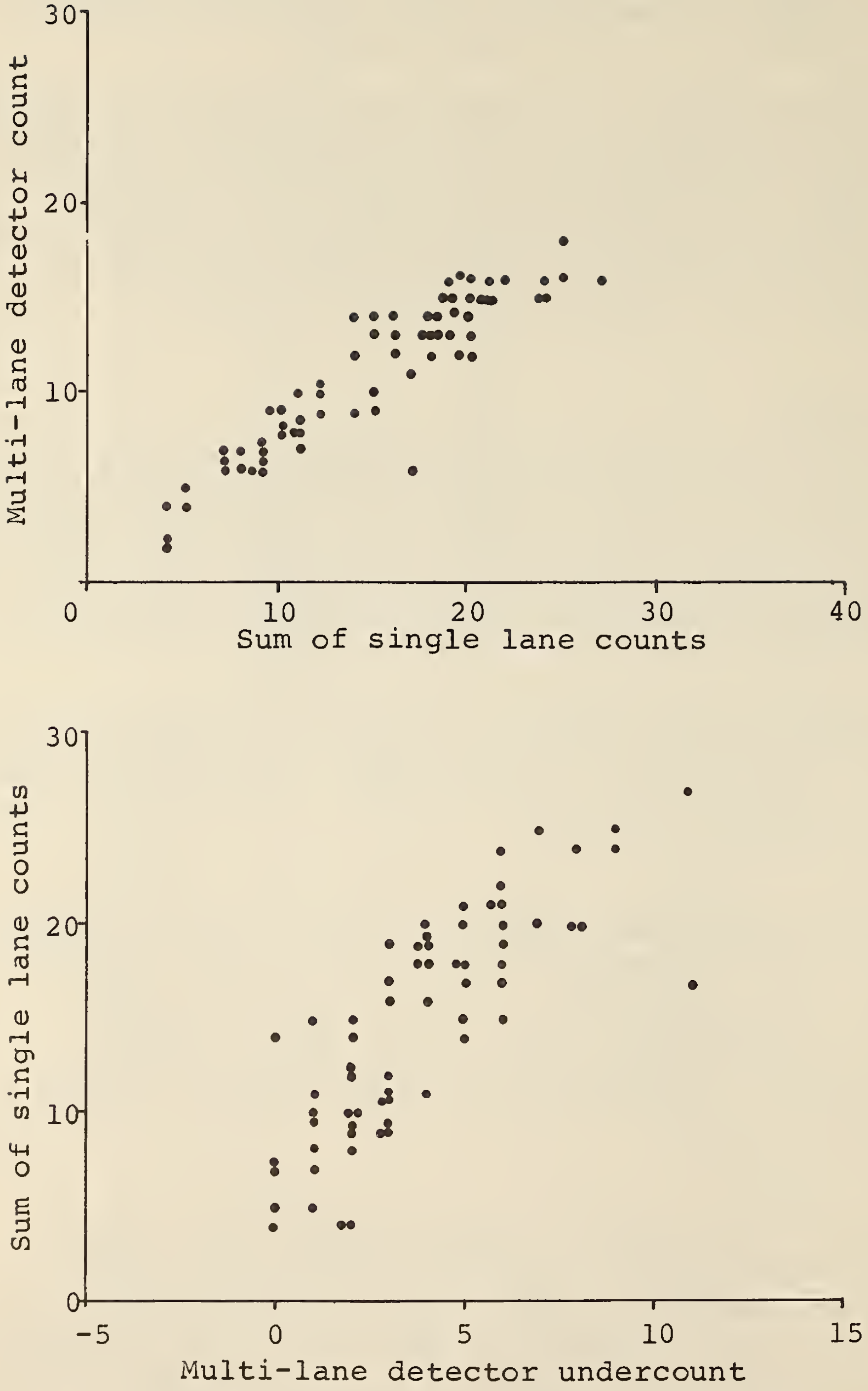
Multi-Lane Loops

This phase of the detector placement study consisted of comparing volume counts as measured by one detector over all lanes on a link versus a count obtained by summing the individual detectors on each lane of the link. This comparison was made to evaluate the efficiency of using a single detector per approach but locating the detector to sample all lanes rather than simply one lane.

Data were collected for a number of links on several days. The three links selected for detailed analysis were: 16th Street SB at K Street, 21st Street SB at Pennsylvania Avenue, and K Street EB at 17th Street. Data were collected during the a.m. peak hour.

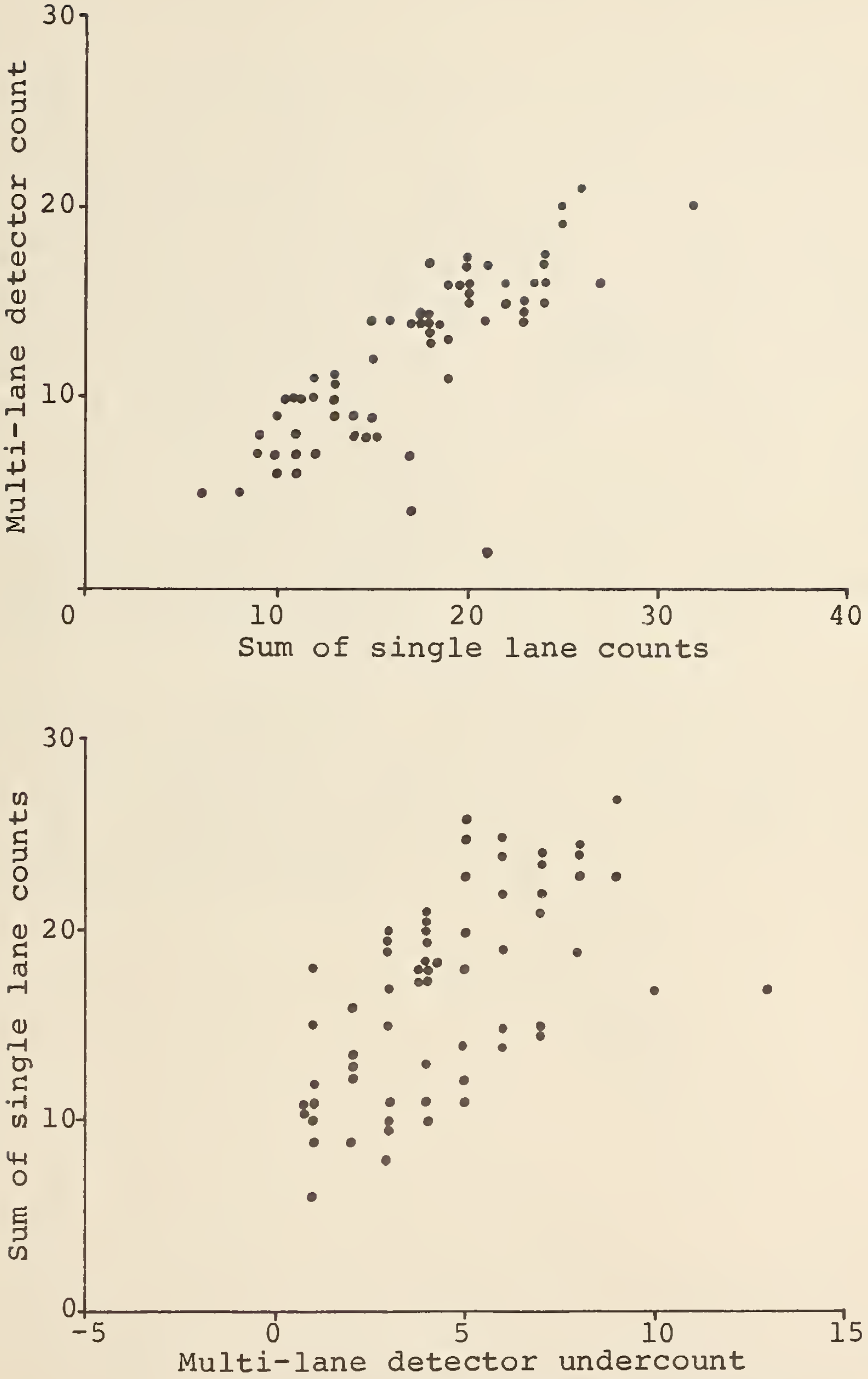
Both the single lane loops and multilane loops were the previously described temporary loops. The multilane loops were placed adjacent and downstream of the individual loop cordons. The count data used was that which was recorded on the raw pulse tape. Each tape recorded approximately 22 consecutive cycles of data. The first tape encompassed a time period of approximately 7:10 to 7:40, the second tape from 7:45 to 8:15, and the third from 8:20 to 8:50 a.m.

The analysis of the data can be best described in graphic form. The graphs in Figure 22 plot the count of the multilane detector by cycle versus the sum of the individual lane detector counts. As can be seen there is considerable scatter between the count over the multilane lane detector. This relationship is biased in the direction of the sum of the individual detectors. This indicates that the sum of the counts at the individual detectors is consistently higher than that for the multiple lane detector. This relationship seems to simulate the real world condition, as expected, the multilane loop undercounted especially when volumes were



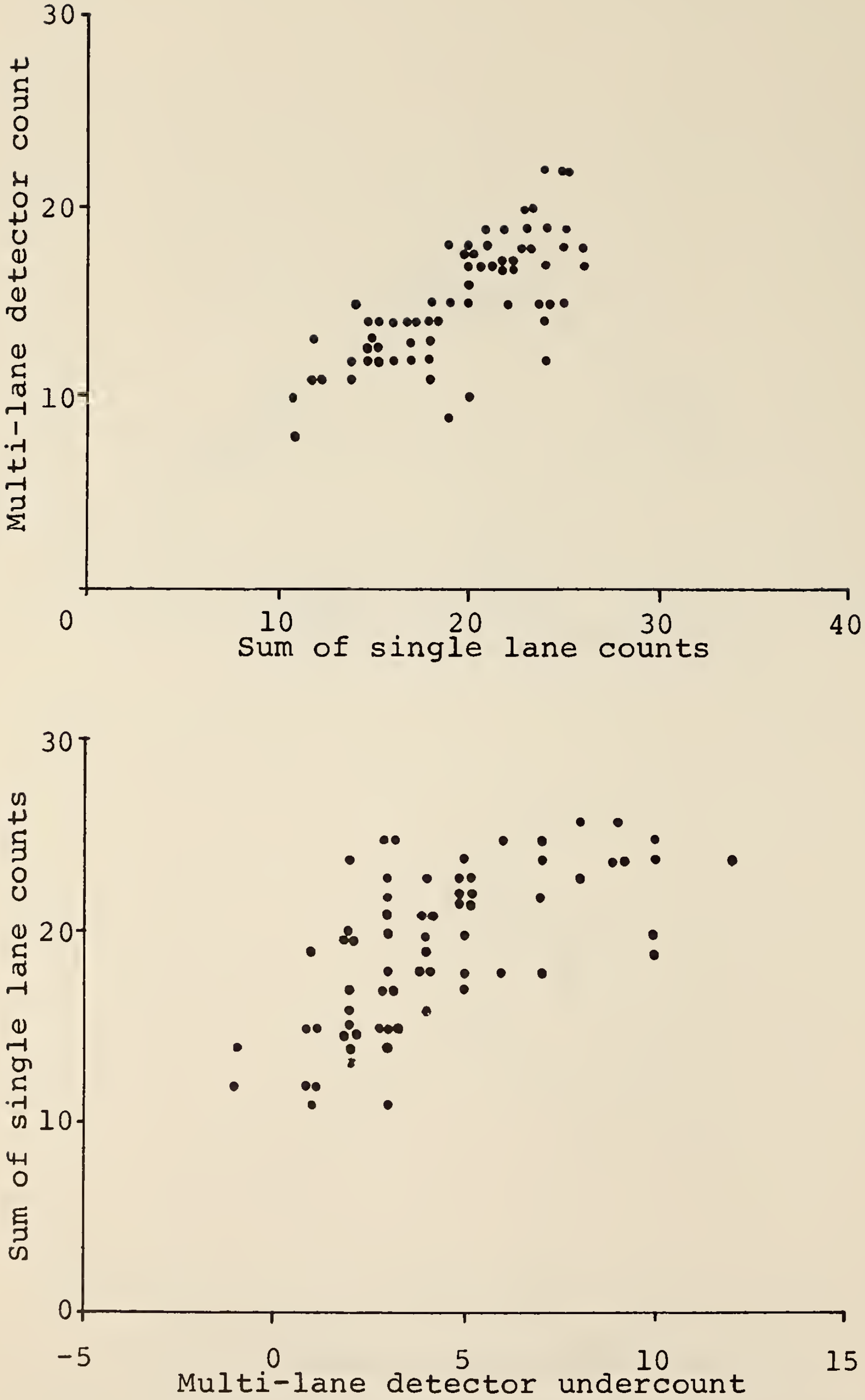
21st Street southbound at Pennsylvania Avenue

Figure 22. Multi-lane loop to single lane loops comparison.



16th Street southbound at K Street

Figure 22. Multi-lane loop to single lane loops comparison, (continued).



K Street eastbound at 17th Street

Figure 22. Multi-lane loop to single lane loops comparison, (continued).

high. The question of the magnitude and consistency of this bias and its relationship to traffic volume then arises.

This question was addressed by plotting the sum of the individual counts against the undercount by the multilane loop expressed as a delta error in number of counts per cycle. These are shown in the lower plots on Figure 21. The relationship shows the magnitude of the error. It can be expected that, in general, a higher volume count is accompanied by a greater error. It would also be expected, however, that the magnitude of the error would be much less than actually experienced, particularly at lower volumes. A cursory review of the curve plots shows that the large variance of the error does not permit an opportunity to predict the error (or true count) based on the count of the multi-lane detector. If the error were either stable for all traffic volumes, or bore some predictable relationship to traffic volumes, a factoring process could be employed to estimate the actual street count from the multilane detector count. The above analysis indicates that this is not practical.

It can be concluded, therefore, from the above analysis that the use of multilane loop detectors is not adequate to achieve an accurate volume count. The magnitude of error in the multilane counts bears little relationship to the volume of vehicles on the street. It appears to be as much a random occurrence as one which would have a predictable relationship.

CRITICAL LANE ANALYSIS

A critical lane detectorization scheme is advantageous when one of two conditions exist: The critical lane always exerts the major demand on the approach, or the "critical lane" exerts a demand that is not significantly less then another lane which, for short time periods, may be experiencing the major demand. The initial problem is to develop an unambiguous definition of "critical lane", then to measure the traffic parameters on each lane of the link over time to ascertain whether the critical lane shifts from one lane to another on an approach. If shifts in critical lanes are apparent, the regularity of such shifts and whether they can be related to time of day must be determined.

Critical Lane Identification Criteria

To serve as a basis for the critical lane analysis, several traffic parameters were defined to identify the critical lane. Each definition attempted to quantify the demand placed on an intersection by each of the traffic lanes. These criteria were subsequently evaluated as to their worth in the identification of critical lane from the perspectives of both accuracy and ease of use.

The criteria initially chosen for analysis and selection of the critical lane includes the following:

- The lane with the maximum volume
- The lane serving the longest platoons (this criteria is very similar to the first)
- The lane with the greatest platoon discharge time
- The lane with the highest product of total lane volume and average time headway.

Time of Day Comparisons

The first question to be addressed is that of determining whether the critical lane shifts by time of day. The analysis was performed on the basis of the second and third definitions of critical lane, that is, the lane with the maximum number of vehicles in the platoon and the lane with the greatest platoon discharge time. The studies taken are noted earlier and in Appendix A.

The results of the data for the two links for both days are shown in Table 22. In addition to the average discharge time per cycle and average platoon length per cycle, the

Table 22. Results of queue discharge study.

L Street eastbound at 19th Street, 5/21/75

Time	Lane 2			Lane 3		
	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Platoon Length (vehicles)	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Platoon Length (vehicles)
7:45 - 8:15	3.85	7.6	2.7	2.38	4.7	2.0
8:15 - 8:45	2.81	22.0	7.9	3.31	19.3	5.8
8:45 - 9:15	2.86	19.6	6.9	3.08	23.3	7.6
9:15 - 9:45	2.78	14.1	5.1	2.93	18.9	6.5
9:45 - 10:15	3.06	23.2	7.6	2.80	24.3	8.7
10:45 - 11:15	3.21	31.7	9.2	3.26	28.8	8.7
11:15 - 11:45	3.22	26.6	6.9	2.84	27.9	9.4
11:45 - 12:15	4.01	28.7	6.7	2.99	27.8	9.4
12:15 - 12:45	3.16	22.4	6.9	2.96	25.8	8.8
2:00 - 2:30	3.30	28.1	7.3	2.92	29.9	10.2
2:30 - 3:00	*	30.6	6.3	*	26.1	9.6
3:30 - 4:00	3.39	17.5	5.2	2.92	21.3	7.3
4:00 - 4:30	3.04	11.4	3.8	2.91	17.0	5.9
4:30 - 5:00	3.34	25.7	6.9	3.33	31.6	7.4

*Discharge rate not meaningful due to downstream congestion.

Table 22. Results of queue discharge study (continued).

L Street eastbound at 19th Street, 5/22/75

Time	Lane 2			Lane 3		
	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (vehicles)	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (vehicles)
7:45 - 8:15	2.77	9.3	3.4	3.31	13.1	4.0
8:15 - 8:45	2.36	13.3	5.6	2.79	12.3	4.4
8:45 - 9:15	3.17	21.1	6.7	3.21	20.0	6.2
9:15 - 9:45	2.88	15.9	5.5	3.32	25.8	7.8
9:45 - 10:15	3.32	17.5	5.3	2.84	19.8	7.0
10:45 - 11:15	3.01	24.0	8.0	2.87	26.3	9.2
11:15 - 11:45	3.51	28.4	8.1	3.25	28.4	8.7
11:45 - 12:15	3.30	23.1	6.7	2.98	26.2	8.2
12:15 - 12:45	3.43	16.0	4.7	3.08	24.4	7.9
2:00 - 2:30	3.11	30.0	9.6	2.94	25.2	8.6
2:30 - 3:00	3.11	23.7	7.6	2.97	21.6	7.3
3:30 - 4:00	3.00	19.3	6.4	2.99	19.7	6.6
4:00 - 4:30	3.47	11.7	3.4	2.96	15.4	5.2
4:30 - 5:00	*	26.6	5.0	*	27.8	6.3

*Discharge rate not meaningful due to downstream congestion.

Table 22. Results of queue discharge study (continued).

Pennsylvania Avenue westbound at 18th Street, 5/22/75

Time	Lane 1			Lane 2			Lane 3		
	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (vehicles)	Average Time Headway (sec.veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (vehicles)	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (vehicles)
7:45 - 8:15	2.75	17.2	6.3	2.67	14.6	5.5	2.69	8.4	3.1
8:15 - 8:45	3.03	14.3	4.7	2.59	15.7	6.1	2.66	8.7	3.3
8:45 - 9:15	3.99	16.0	4.0	2.69	17.6	6.6	2.61	11.3	4.3
9:15 - 9:45	3.23	13.8	4.3	2.63	15.8	6.0	2.52	9.2	3.7
9:45 - 10:15	2.45	7.0	2.9	3.03	19.4	6.4	2.94	5.2	1.8
10:45 - 11:15	3.36	11.0	3.3	2.68	12.8	4.8	2.44	8.8	3.6
11:15 - 11:45	3.34	11.6	3.5	3.13	14.7	4.7	2.73	7.7	2.8
11:45 - 12:15	3.76	14.1	3.8	2.78	18.9	6.8	2.54	9.6	3.8
12:15 - 12:45	4.30	13.6	3.2	3.01	20.2	6.7	2.55	9.3	3.7
2:30 - 3:00	3.13	7.2	2.3	2.54	17.3	6.8	2.73	9.2	3.4
3:30 - 4:00	2.97	6.6	2.2	2.59	14.3	5.5	2.41	5.8	2.4
4:00 - 4:30	2.69	11.1	4.1	3.52	17.8	5.1	3.63	6.9	1.9
4:30 - 5:00	3.26	16.2	5.0	2.54	16.8	6.6	2.81	11.1	4.0

Table 22. Results of queue discharge study (continued).

Pennsylvania Avenue westbound at 18th Street, 5/21/75

Time	Lane 1			Lane 2			Lane 3		
	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (vehicles)	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (vehicles)	Average Time Headway (sec./veh.)	Discharge Time/Cycle (sec.)	Average Platoon Length (veh.)
7:45 - 8:15	2.80	13.8	4.9	2.49	14.7	5.9	3.50	12.6	3.6
8:15 - 8:45	3.63	15.1	4.2	2.75	15.9	5.8	3.73	12.2	3.3
8:45 - 9:15	4.12	16.8	3.8	3.40	21.5	6.3	3.19	11.0	3.5
9:15 - 9:45	3.56	11.4	3.2	3.14	20.9	6.7	2.81	12.3	4.4
9:45 - 10:15	3.40	9.4	2.7	3.23	20.4	6.7	3.05	10.7	3.5
10:45 - 11:15	4.20	9.7	2.3	2.87	17.7	6.2	2.41	8.5	3.5
11:15 - 11:45	4.13	13.1	3.2	3.10	20.0	6.0	2.73	7.0	2.5
11:45 - 12:15	4.33	11.9	2.8	3.18	21.8	6.9	2.36	8.3	3.5
12:15 - 12:45	5.25	17.1	3.3	2.81	17.6	6.3	2.52	11.5	4.6
2:30 - 3:00	3.51	9.0	2.6	3.03	14.4	4.8	2.52	9.2	3.7
3:30 - 4:00	3.42	8.9	2.6	2.65	12.2	4.6	2.78	7.5	2.7
4:00 - 4:30	3.36	7.9	2.4	3.10	12.4	4.0	2.70	6.0	2.2
4:30 - 5:00	3.20	14.6	4.6	2.96	16.6	5.6	2.95	8.7	3.0

average time headway is shown. This number was derived by dividing the total time required for the platoon to discharge (i.e. cross the stop bar) by the number of vehicles in the platoon. This was done on an averaged basis for each 20-cycle data collection period. It was originally planned to use headway as derived in this manner as the fourth critical lane identification criteria (headway multiplied by the total lane count). As shown in the tables, however, there is not only considerable variation in headway from one time period to the next, but also from one day to the next within the same time period. Because the headway parameter is so volatile, the criteria was rejected as a technique to identify critical lanes. Contributing factors to this volatility include the influence of pedestrians, the percentage of trucks and buses, and the effect of human factors in perception/reaction time.

Table 23 summarizes the critical lane selection on L Street for each time period on the two days using both criteria. In general, the two criteria are complementary. On only two occasions do they indicate different lanes as being critical. It is apparent from the table that shifts in critical lane do occur over time for both days. However, the consistency of such shifts from day to day is somewhat in doubt. For instance, when the critical lane is determined by the discharge time criteria, the indicated critical lane is different for the two days on five occasions. In general, it does appear that lane 3 is critical more often than lane 2, but from Table 22 it can be seen that the margin of difference is not great.

The data on Pennsylvania Avenue reveals that lane 2 is the critical lane for all the time periods for which data was collected. The next most critical lane fluctuates between lanes 1 and 3, but this is not important since the lane of maximum demand is the one of primary interest.

Several conclusions are drawn from this detailed analysis of two links. First, as mentioned previously, the use of the product of lane volume times headway is not a viable technique to measure critical lane demands. Secondly, either the discharge time or the platoon length criteria yielded consistant results for the lanes considered on both L Street and Pennsylvania Avenue. Thirdly, consideration must be given to the implication of using either discharge time or platoon length, as a technique to identify critical lanes. Using the time measure, such factors as pedestrian interference with turning movements are directly accounted for. The vehicle count criteria, on the other hand, is a measure which one would desire to optimize in obtaining the maximum utilization of the street. Thus, if one lane had a higher

Table 23. Change in critical lane by time of day.

L Street eastbound at 19th Street

Time	Date: 5/21/75		Date: 5/22/75	
	Critical Lane		Critical Lane	
	Discharge Time	Platoon Length	Discharge Time	Platoon Length
7:45 - 8:15	2	2	3	3
8:15 - 8:45	2	2	2	2
8:45 - 9:15	3	3	2	2
9:15 - 9:45	3	3	3	3
9:45 - 10:15	3	3	3	3
10:45 - 11:15	2	2	3	3
11:15 - 11:45	3	3	3	3
11:34 - 12:15	2	3	3	3
12:15 - 12:45	3	3	3	3
2:00 - 2:30	3	3	2	2
2:30 - 3:00	2	3	2	2
3:30 - 4:00	3	3	3	3
4:00 - 4:30	3	3	3	3
4:30 - 5:00	3	3	3	3

average discharge time due to pedestrian interference, etc., but had a lower average volume, the goals of the signal system might best be served be designing only for the maximum volume. In most cases, the critical lane is the same for either the platoon length or discharge time criteria so that instances in which there is a conflict between the two will be few. There may be some unusual instances where the critical lane should be chosen on the basis of the discharge time criteria. Such a case might be where long queues develop in a turning lane faced with exceptionally large pedestrian flows.

Critical Lane Volume Measures

The next phase of the analysis consisted of examining the first critical lane criterion, total lane volume. Using the previously described data collected for the primary volume studies, average values of total vehicles per lane per cycle were tabulated. This data was for the 30 approaches selected by the research team to be typical of the links within the study area. Data were available for a.m., mid-day and p.m. periods for 25 cycles, or slightly over 30 minutes, in each period. Before proceeding with the analysis results, the measurement of primary and secondary volumes and their application to the demand at the downstream intersection is discussed. In most cases, these volumes were taken upstream of the Q2 location and do not reflect the actual lane demand at the intersection. If significant lane changing occurs between the point where the volumes were measures and the downstream intersection, the volumes are not a true measure of demand. It is felt, however, that the net lane change was small enough to warrant the use of the sum of the primary and secondary volumes by lane for the critical lane identification criteria. The lane volumes per cycle on each of the 30 approaches are shown in Table 24.

Based on experience using the first two critical lane criteria, it appears that both give an adequate indication of the critical lane. The total lane volume criteria is somewhat more practical than the average platoon length since total volume data are typcially more useful for other purposes and entail a more straight forward data collection procedure. Therefore, all subsequent analysis in this study was performed using only criteria number one, total lane volume.

Of most interest in the placement of critical lane detectors is the penalty paid by not having the detector in the critical lane. Table 25 summarizes the penalty be means of the numberical difference between the presently detectorized critical lane and adjacent lane demands. This table

Table 24. Lane volume per cycle by lane and time period.

Approach	A.M.				MID-DAY				P.M.			
	Lane 1	Lane 2	Lane 3	Lane 4	Lane 1	Lane 2	Lane 3	Lane 4	Lane 1	Lane 2	Lane 3	Lane 4
1. 20th @ K St. NB	0.6	10.5	11.5	8.3	1.0	10.1	6.2	0.0	1.6	7.2	7.0	0.5
2. K St. @ 19th EB	7.2	13.9			9.2	4.0			5.5	6.8		
3. K St. @ 19th WB	5.2	6.6			6.1	10.9			NA	NA		
4. 19th @ K St. SB	5.9	13.1	7.0		4.1	11.6	0.7		6.4	11.2	4.9	
5. K St. @ 18th EB	9.3	10.2			7.0	8.8			6.0	7.5		
6. K St. @ 18th WB	4.8	6.2			4.5	8.7			5.2	11.1		
7. K St* @ 19th WB	6.4	12.6			6.5	6.7			4.8	10.6		
8. 16th @ K St. SB	1.5	8.8	5.3		0.0	3.0	10.1		0.0	10.2	8.4	
9. K St. @ 15th EB	5.6	12.6			7.5	8.2			6.4	11.0		
10. K St. @ 15th WB	5.6	9.2			6.6	8.1			4.7	8.0		
11. M St. @ 34th EB	8.6	17.5	17.2		5.2	10.4	4.4		10.7	10.1		
12. M St. @ 34th WB	1.8	6.4			0.7	7.4	6.5		5.4	12.3	10.6	
13. 34th @ M St. SB	NA	NA	NA		NA	NA	NA		NA	NA	NA	
14. Penn. @ 28th EB	8.4	10.9	7.7		0.0	8.4	7.0		0.3	7.8	6.8	
15. Penn. @ 28th WB	4.5	5.9			4.0	6.6			10.2	8.3		
16. 21st @ Penn. SB	2.4	6.4	2.8		3.0	8.24	0.5		5.0	12.9	5.2	
17. Penn. @ 19th EB	6.2	10.4	3.7		4.3	11.3	4.4		4.3	9.4	5.9	
18. Penn. @ 19th WB	5.4	7.4	5.7		1.6	8.8	5.7		4.6	8.5	6.4	
19. Penn. @ 18th WB	4.0	7.9	4.8		1.9	9.4	6.2		3.3	8.9	3.4	
20. Penn. @ 17th EB	8.1	13.2	6.3		3.6	8.5	6.4		3.8	13.6	6.7	
21. H St. @ 16th WB	7.2	8.3	5.5		4.3	7.5	4.0		5.2	5.3	2.8	
22. 17th @ H St. NB	0.9	6.6	10.2		0.0	4.0	6.8		0.0	2.4	5.9	
23. 17th @ H St. SB	0.2	6.7	2.6		0.0	1.5	15.2		0.0	7.6	4.6	
24. L St. @ 20th EB	4.1	14.3	11.4	7.0	0.0	6.8	9.4	0.0	0.0	7.8	9.1	0.0
25. L St. @ 19th EB	8.0	13.4	12.1	6.1	0.0	7.8	9.1	1.2	2.4	10.5	9.6	2.3
26. L St. @ 15th EB	NA	NA	NA	NA	0.0	9.4	12.3		2.9	15.5	14.1	
27. 20th @ L St. NB	1.5	10.3	9.2	1.9	0.0	8.0	7.7		0.2	8.0	8.2	0.3
28. 19th @ Penn. SB	5.4	7.7	1.0		NA	NA	NA		3.0	10.0	10.3	
29. M St. @ 28th WB	NA	NA	NA		NA	NA	NA		NA	NA	NA	
30. H St. @ 16th EB	2.9	4.7			2.5	3.3	3.3		3.8	5.3		

Table 25. Differences in critical lane versus next most critical lane.

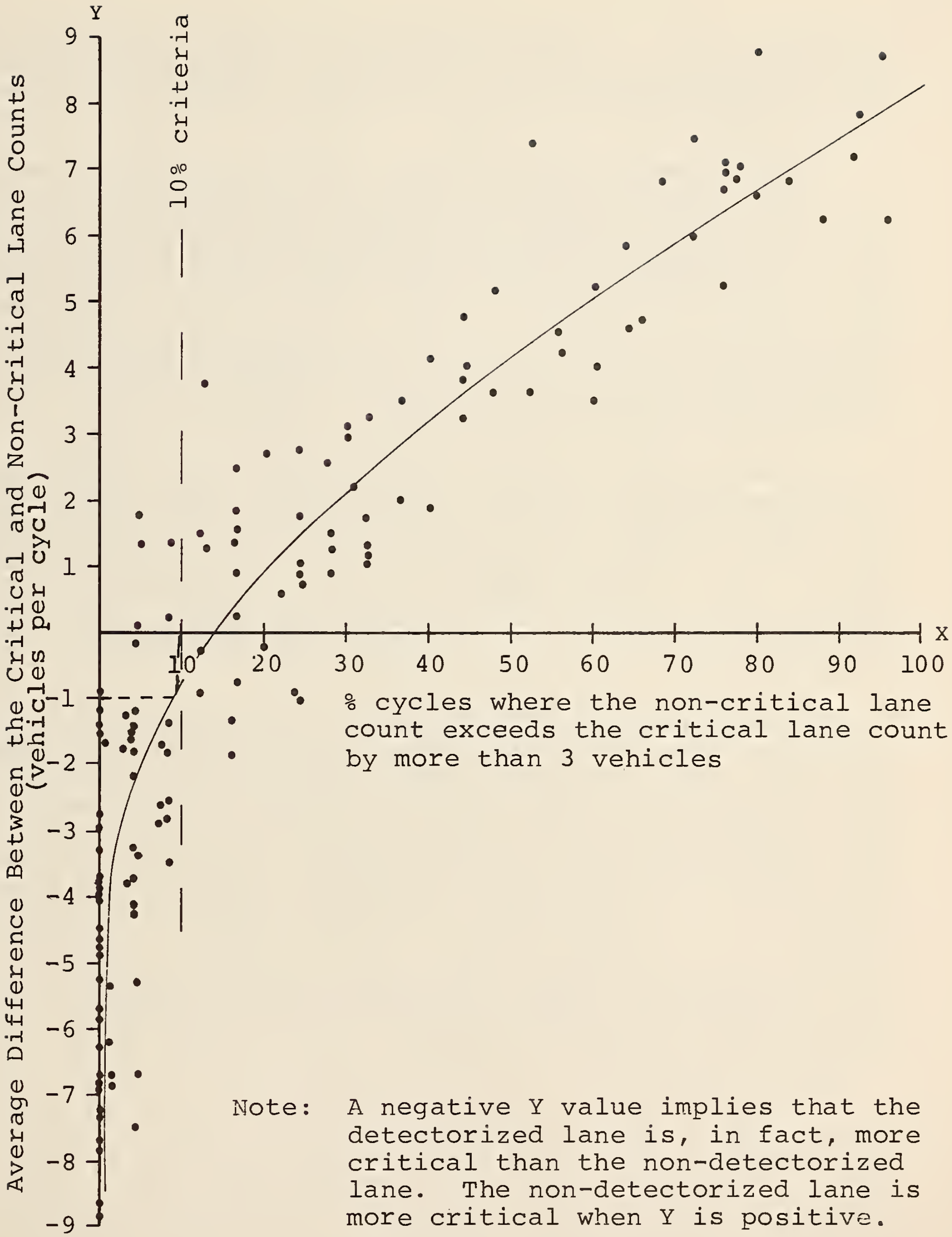
Approach	Existing Critical Lane	A.M.		MIDDAY		P.M.	
		Alternate Lane	Average Difference Per Cycle	Alternate Lane	Average Difference Per Cycle	Alternate Lane	Average Difference Per Cycle
1. 20th @ K St. NB	2	3	+1.0	3	-3.9	3	-0.2
2. K St. @ 19th EB	2	1	-6.7	1	+5.2	1	-1.3
3. K St. @ 19th WB	2	1	-1.4	1	-4.8	NA	NA
4. 19th @ K St. SB	2	3	-6.1	1	-7.5	1	-4.8
5. K St. @ 18th EB	2	1	-0.9	1	-1.8	1	-1.5
6. K St. @ 18th WB	2	1	-1.4	1	-4.2	1	-5.9
7. K St. @ 17th WB	2	1	-6.2	1	-0.2	1	-5.8
8. 16th @ K St. SB	2	3	-3.5	3	+7.1	3	-1.8
9. K St. @ 15th EB	2	1	-7.0	1	-0.7	1	-4.6
10. K St. @ 15th WB	2	1	-3.6	1	-1.5	1	-3.3
11. M St. @ 34th EB	2	3	-0.3	1	-5.2	1	+0.6
12. M St. @ 34th WB	2	1	-4.6	3	-0.9	3	-1.7
13. 34th @ M St. SB	2	NA	NA	NA	NA	NA	NA
14. Penn. @ 28th EB	2	1	-2.5	3	-1.4	3	-1.0
15. Penn. @ 28th WB	2	1	-1.4	1	-2.5	1	+1.9
16. 21st @ Penn. SB	2	3	-3.6	1	-5.2	3	-7.7
17. Penn. @ 19th EB	2	1	-4.2	3	-6.9	3	-3.5
18. Penn. @ 19th WB	2	3	-1.7	3	-3.1	3	-2.1
19. Penn. @ 18th WB	2	3	-3.1	3	-3.2	3	-5.6
20. Penn. @ 17th EB	2	1	-5.1	3	-2.1	3	-6.9
21. H St. @ 16th WB	2	1	-1.1	1	-3.2	1	-0.1
22. 17th @ H St. NB	2	3	+3.6	3	+2.8	3	+3.5
23. 17th @ H St. SB	2	3	-4.1	3	+13.7	3	-3.0
24. L St. @ 20th EB	2	3	-2.9	3	+2.6	3	+1.3
25. L St. @ 19th EB	2	3	-1.3	3	+1.3	3	-0.9
26. L St. @ 15th EB	2	NA	NA	3	+3.9	3	-1.4
27. 20th @ L St. NB	2	3	-1.1	3	-0.3	3	+0.2
28. 19th @ Penn. SB	2	1	-2.3	NA	NA	3	+0.3
29. M St. @ 28th WB	2	NA	NA	NA	NA	NA	NA
30. H St. @ 16th EB	2	1	-1.8	3	-6.5	1	-1.5

reveals that the existing detectorized lane is the critical lane in a very high percentage of cases. For the total lane volume criteria, most of the exceptions occur during the mid-day period when traffic conditions are somewhat different from the peak periods.

Although the presently detectorized lane is, in most cases, the critical lane, it remains to be shown whether the non-detectorized lane exerts enough demand to exceed the desired limit of error. An unacceptable error would exist when the count in the non-detectorized lane is greater than the count in the detectorized lane by a margin of more than three vehicles for 10% of the time. This goal is derived from the accuracy goals stated for volume where a critical lane count is to be measures to an accuracy of plus or minus three vehicles 90% of the time. The error was examined by comparing the primary volume counts on a cycle-by-cycle basis for the 25-cycle samples. A direct answer as to the percentage of time that the criteria was exceeded could then be obtained.

In order to relate the cycle-by-cycle differences to the 25-cycle averages a curve was plotted with the average difference in lane count as the ordinate and the percentage of time that the criteria was exceeded as the abscissa. A negative difference existed when the count in the detectorized lane was greater than the count in the non-detectorized lane. Since most of the data points were on the negative side of the curve, additional data points were plotted assuming that the non-detectorized lane was actually detectorized and that the detectorized lane was not. This yielded an abundance of points for the positive side of the scale insuring a more accurate representation. A curve was then hand-fit to the data points. This curve is depicted on Figure 23. The curve was hypothesized to be parabolic in shape for y values greater than -4.0.

The primary point of interest is at the intersection of the curve and the 10% criteria line. These lines intersect at approximately -1.0 on the vertical scale. Any value lower than this point on the ordinate indicates that, when the detectorized lane is compared to the non-detectorized lane, that the criteria fails. Furthermore, when the average differences between the two lanes is greater than +1.0, it is likely that the criteria passes for the non-detectorized lane. In terms of detector placement, a value of less than -1.0 for one time period means that the detectorized lane is most critical during that time period and no additional detectors are required. A +1.0 value indicates that the detector should be removed from its present location to the



Note: A negative Y value implies that the detectorized lane is, in fact, more critical than the non-detectorized lane. The non-detectorized lane is more critical when Y is positive.

Figure 23. Relationship between average and per cycle differences in lane volume counts on a link.

alternate lane. A value between +1.0 and -1.0 means that the criteria is probably exceeded no matter which lane is detectorized and that detectorization of both lanes should be considered.

When considering the lane volume differences for the a.m., p.m. and mid-day samples, the criteria should pass for all three time periods if additional detectorization is avoided. However, if two time periods pass by a significant and the third period is in the range of zero, the penalty paid for detectorizing only one lane will not be severe, particularly if the third value is during the mid-day period. Value judgements must be made for each individual case, incorporating one's priorities and the knowledge of link-specific traffic characteristics.

A major use for this curve is seen in the design of new surveillance systems. In instances where there is doubt as to which lane should be detectorized, a simultaneous lane volume count is made for a half-hour to one-hour time period. One lane is then tentatively designated as critical and the average per cycle differences between the counts are computed by subtracting the count from the assumed critical lane from that of the non-critical lane.

If the difference if negative, the assumed critical lane was correct; if positive, the alternate lane is more critical. The percent of cycles where the non-critical lane count by more than three vehicles can then be read directly from the chart. Decisions as to whether one or both lanes should be detectorized can be based on the error criteria desired for each individual system. Care must be exercised on intrepreting this chart, however, since it was based on an 80 second cycle. The percent of time that the criteria was exceeded would likely be less for shorter cycles due to the lower volume per cycle. However, the curve serves as a good basis by which to determine detector needs on each lane of a link.

To place this analysis in perspective, it is important to note that the original selection of critical lanes was made based on the engineering judgement of the system designers. Few, if any, analytical studies were made. These original decisions were found to be wrong by this analysis in only a few instances and then it was typically only for one of the three time periods. The point to be made is that the more complex techniques suggested in this section need to be applied in a relatively few cases; in the vast majority of cases, identification of the critical lane can be made by the system designers in conjunction with local traffic operations personnel.

The critical lane analysis identifies four conditions with which the designer must cope: 1) Approaches where one lane is always critical; 2) Approaches where the critical lane shifts between two lanes, but the difference in volume is not great or the shift occurs infrequently; 3) Approaches where shifts in the critical lane are significant; and, 4) Approaches exhibited specific critical lanes during peak hours, but for a myriad of reasons, erratic shifts in the critical lane occurred during non-peak periods.

Fortunately, the majority of the approaches are of the first type described above. Approaches of the second and third type are not uncommon on approaches (links) with four lanes. Whether both lanes should be detectorized or not is a value judgement that must be made by the system designer. With the fourth type, it is suggested that the peak-period critical lane be instrumented and that the sensor simply be disabled during non-peak traffic conditions with the control algorithm reverting to a backup mode.

Critical Lane Primary Volumes

Another issue related to the measurement of critical lane deals with the problems associated with using critical lane primary volumes rather than total approach primary volumes. A "t" test was performed on the mean values of primary flow ratios for the total volumes and critical lane volumes with the null hypothesis that the two population had the same mean. Both the "t" statistic and the degrees of freedom were calculated using standard methods and assuming that the variances are unequal.

The results of the calculations are shown on Table 26. As seen on the table, there are few instances where the results are significantly different. It is concluded, therefore, that instrumenting only the critical lane will provide a sufficiently accurate estimate of primary volume.

Table 26. Comparison of critical lane and total volume primary flow ratios.

AM Peak Hour

LOCATION & DIRECTION	TOTAL VOLUME		CRITICAL LANE		T test		Significance
	Primary Ratio	Standard Deviation	Primary Ratio	Standard Deviation	Degress of Freedom	t Statistic	
1. 20th @ K St. NB	.98	.03	.95	.06	38.2	2.236	++
2. K St. @ 19th EB	.96	.05	.98	.04	49.6	-1.562	
3. K St. @ 19th WB	.91	.16	.91	.12	48.2	0.000	
4. 19th @ K St. SB	.77	.09	.81	.09	52.0	-1.571	
5. K St. @ 18th EB	.91	.07	.92	.08	25.3	-0.470	
6. K St. @ 18th WB	.85	.07	.88	.19	32.9	-0.741	
7. K St.* @ 17th WB	.95	.05	.94	.06	50.4	0.640	
8. 16th @ K St. SB	.92	.06	.96	.07	50.8	-2.169	++
9. K St. @ 15th EB	.91	.08	.95	.06	48.2	-2.000	+
10. K St. @ 15th WB	.89	.08	.84	.20	34.1	1.161	
11. M St. @ 34th EB	.77	.05	.74	.08	43.6	1.590	
12. M St. @ 34th WB	.99	.02	.99	.02	52.0	0.000	
13. 34th @ M St. SB	NA	NA	NA	NA	NA	NA	
14. Penn. @ 28th EB	.94	.04	.91	.09	35.9	1.523	
15. Penn. @ 28th WB	.56	.16	.47	.28	41.3	1.395	
16. 21st @ Penn. SB	.83	.12	.89	.14	50.8	-1.627	
17. Penn. @ 19th EB	.93	.05	.93	.07	47.1	0.000	
18. Penn @ 19th WB	.66	.10	.71	.19	39.4	-1.164	
19. Penn. @ 18th WB	.82	.10	.81	.15	45.3	0.277	
20. Penn. @ 17th EB	.73	.06	.75	.10	42.6	-0.857	
21. H St. @ 16th WB	.58	.13	.67	.18	47.3	-2.027	++
22. 17th @ H St. NB	.80	.10	.77	.19	39.4	0.699	
23. 17th @ H St. SB	.87	.10	.84	.15	45.6	0.832	
24. L St. @ 20th EB	.78	.05	.75	.16	31.0	0.895	
25. L St. @ 19th EB	.67	.06	.68	.12	38.2	-0.373	
26. L St. @ 15th EB	NA	NA	NA	NA	NA	NA	
27. 20th @ L St. NB	.74	.09	.69	.14	44.4	1.502	
28. 19th @ Penn. SB	.93	.07	.90	.13	39.9	1.016	
29. M St. @ 28th WB	NA	NA	NA	NA	NA	NA	
30. H St. @ 16th EB	.48	.21	.43	.30	46.5	0.683	

* K St. @ 17th and Conn.
\+ significant at 10% level
++ significant at 5% level

Table 26. Comparison of critical lane and total volume primary flow ratios (continued).

Midday Peak Hour

LOCATION & DIRECTION	TOTAL VOLUME		CRITICAL LANE				
	Primary Ratio	Standard Deviation	Primary Ratio	Standard Deviation	Degrees of Freedom	t Statistic	Significance
1. 20th @ K St. NB	.95	.06	.94	.08	48.2	0.500	
2. K St. @ 19th EB	.92	.07	.90	.17	34.6	0.544	
3. K St. @ 19th WB	.84	.09	.82	.10	51.4	0.743	
4. 19th @ K St. SB	.75	.11	.76	.13	50.6	-0.294	
5. K St. @ 18th EB	.81	.18	.85	.10	40.7	-0.971	
6. K St. @ 18th WB	.92	.08	.92	.10	49.6	0.000	
7. K St.* @ 17th WB	.84	.06	.85	.15	34.1	0.309	
8. 16th @ K St. SB	.73	.17	.80	.21	49.8	-1.295	
9. K St. @ 15th EB	.86	.08	.87	.11	47.5	0.368	
10. K St. @ 15th WB	.85	.11	.84	.13	50.6	0.294	
11. M St. @ 34th EB	.79	.09	.76	.13	46.3	0.949	
12. M St. @ 34th WB	.97	.04	.98	.04	52.0	-0.884	
13. 34th @ M St. SB	NA	NA	NA	NA	NA	NA	
14. Penn. @ 28th EB	.97	.04	.99	.04	52.0	-1.768	+
15. Penn. @ 28th WB	.85	.13	.82	.22	42.2	0.587	
16. 21st @ Penn. SB	.80	.12	.83	.14	50.8	-0.813	
17. Penn. @ 19th EB	.95	.04	.98	.04	52.0	-2.652	++
18. Penn @ 19th WB	.91	.08	.89	.11	47.5	0.735	
19. Penn. @ 18th WB	.85	.08	.83	.11	47.5	0.735	
20. Penn. @ 17th EB	.85	.09	.82	.18	38.2	0.745	
21. H St. @ 16th WB	.58	.13	.54	.20	44.6	0.838	
22. 17th @ H St. NB	.68	.15	.78	.21	47.1	-1.937	+
23. 17th @ H St. SB	NA	NA	NA	NA	NA	NA	
24. L St. @ 20th EB	NA	NA	NA	NA	NA	NA	
25. L St. @ 19th EB	.65	.13	.64	.24	40.0	0.183	
26. L St. @ 15th EB	.67	.11	.75	.15	47.7	-2.150	++
27. 20th @ L St. NB	.64	.11	.62	.17	44.5	0.494	
28. 19th @ Penn. SB	NA	NA	NA	NA	NA	NA	
29. M St. @ 28th WB	NA	NA	NA	NA	NA	NA	
30. H St. @ 16th EB	.49	.12	.53	.12	52.0	-1.179	

* K St. @ 17th and Conn.
\+ significant at 10% level.
++ significant at 5% level.

Table 26. Comparison of critical lane and total volume primary flow ratios (continued).

PM Peak Hour

LOCATION & DIRECTION	TOTAL VOLUME Primary Ratio	Standard Deviation	CRITICAL LANE Primary Ratio	Standard Deviation	T TEST Degrees of Freedom	t Statistic	SIGNIFICANCE
1. 20th @ K St. NB	.95	.06	.92	.15	34.1	0.928	
2. K St. @ 19th EB	.80	.14	.89	.12	50.8	-2.440	++
3. K St. @ 19th WB	NA	NA	NA	NA	NA	NA	
4. 19th @ K St. SB	.76	.11	.82	.11	52.0	-1.928	+
5. K St. @ 18th EB	.95	.06	.95	.08	48.2	0.000	
6. K St. @ 18th WB	.94	.06	.93	.08	48.2	0.500	
7. K St.* @ 17th WB	.92	.06	.89	.09	45.3	1.387	
8. 16th @ K St. SB	.75	.08	.75	.15	39.7	0.000	
9. K St. @ 15th EB	.91	.08	.91	.10	49.6	0.000	
10. K St. @ 15th WB	.80	.09	.74	.16	41.0	1.634	
11. M St. @ 34th EB	.88	.08	.76	.18	35.9	3.046	++
12. M St. @ 34th WB	.99	.01	.99	.02	38.2	0.000	
13. 34th @ M St. SB	NA	NA	NA	NA	NA	NA	
14. Penn. @ 28th EB	.92	.07	.88	.13	39.9	1.355	
15. Penn. @ 28th WB	.72	.14	.79	.19	47.8	-1.483	
16. 21st @ Penn. SB	.88	.07	.89	.07	52.0	-0.505	
17. Penn. @ 19th EB	.96	.05	.96	.06	50.4	0.000	
18. Penn @ 19th WB	.88	09	.84	.11	50.4	1.407	
19. Penn. @ 18th WB	.84	.10	.81	.18	40.7	0.728	
20. Penn. @ 17th EB	.73	.15	.77	.08	39.7	-1.176	
21. H St. @ 16th WB	.62	.14	.63	.20	46.5	-0.205	
22. 17th @ H St. NB	.87	.15	.92	.22	45.9	-0.939	
23. 17th @ H St. SB	.83	.29	.76	.24	50.2	0.930	
24. L St. @ 20th EB	.66	.10	.63	.15	45.3	0.832	
25. L St. @ 19th EB	.71	.10	.65	.18	40.7	1.457	
26. L St. @ 15th EB	.78	.06	.72	.11	40.2	2.394	++
27. 20th @ L St. NB	.67	.11	.66	.15	47.7	0.269	
28. 19th @ Penn. SB	.97	.03	.98	.03	52.0	-1.179	
29. M St. @ 28th WB	NA	NA	NA	NA	NA	NA	
30. H St. @ 16th EB	.60	.12	.60	.19	43.9	0.000	

* K St. @ 17th and Conn.

\+ Significant at 10% level.

++ Significant at 5% level.

SINK/SOURCE INVESTIGATIONS

As noted in the procedures discussion, the analysis of the effect of traffic sinks and sources was approached from three perspectives. The first was the determination of the effect of a sink or source on magnitude of lane-changing by vehicles on a link. The second was the determination of the magnitude of change that a sink/source has on link volumes between points upstream and downstream of its entrance. The third was an attempt to translate the effect of total link volumes to the effect on the critical lane alone.

Lane Changing Analysis

Data collection for the lane changing analysis took place during one morning peak hour on L Street eastbound at 19th Street. There are three major parking garages and one parking lot located on either side of this link. Two of these garages are located opposite one another approximately midway between the upstream and downstream intersections. These two garages were isolated for the study of lane changes reduced by sink/sources because they represent apparent "worst" case. The results should, then, conservatively represent a typical condition.

Three data collectors were used, one at the garages, one upstream from the garages and one downstream. The person stationed at the garages counted the number of vehicles turning into either garage and the number of vehicles passing that point by lane and by cycle on L Street. The other two data collectors counted the number of lane changes caused specifically by vehicular activity at either of the garages. One person counted lane changes upstream of the garages while the other counted the downstream lane changes. These data were recorded for 20 consecutive cycles during the period of heaviest a.m. activity at the garage.

A summary of data for the vehicle exchange between lanes 1 and 2 is shown in Table 27 for portions of L Street both upstream and downstream of the sink and source. The lane change between these two lanes was selected for display because it represented by far the most severe case of lane changing on the link. Only seven sink/source related lane changes occurred for the other four combinations of lane changes over the 20 cycles.

From the table 27, it can be seen that there was essentially no lane change effect until Cycle 10. At that time vehicles from the parking garage on the south side of

Table 27. Number of lane changes on a link caused by the presence of a sink/source.

Cycle Number	Upstream of Sink/Source		Downstream of Sink/Source	
	Lane 1 to 2	Lane 2 to 1	Lane 1 to 2	Lane 2 to 1
1				
2				
3				
4				
5				
6				
7				
8				
9				
10	1			3
11	3			6
12				1
13				
14				1
15				1
16	2			2
17				
18		1		
19				
20				

L Street, adjacent to Lane 1, developed a queue of vehicles from its entrance which extended onto the street itself. It was at this time that vehicles upstream from the queue would pull around the queue into lane 2. A portion of these would continue through the downstream intersection in lane 2 while another portion would weave back into lane 1 downstream of the queue. This is evident by the number of vehicles in the lane 2 to 1 exchange in the fourth column in the table. Also, a number of vehicles proceeded past the sink/source in lane 2 and merged back into lane 1 preparing for the right turn at 19th Street. After cycle 11, however, the number of lane changes quickly dropped. This was due to the dissapation of the queue by the beginning of cycle 12. For cycles 12 through 20 only eight lane changes occurred from either lane 1 to lane 2 or lane 2 to lane 1.

Judging from the above data, it would be safe to conclude that lane changes induced from the activity of sinks along a link are noteworthy only when a queue of vehicles entering the garage extends onto the street. This study shows that this occurs at infrequent and unpredictable time intervals during the a.m. peak hour.

Also, it was observed that a vehicle parked along the curb lane for even one or two minutes can have as much or more effect on traffic turbulence as a sink/sourve since that vehicle would have the same effect as a queue of vehicles backed onto the street from a parking garage. Although data were not collected during the p.m. peak hour, it is expected that the effects of sinks and sources on lane changing will be even less than during the a.m. peak hour since vehicles would not queue up on the link but in the garages. Therefore, it is viewed that the sink/source induces only a small number of vehicles on the link to change lanes either upstream or downstream of the sink/source.

Turning Movement Analysis

The second phase of the study involved counting the number of vehicles proceeding into and out of a number of the parking garages in UTCS Section 3. Fourteen garages on eight different links were selected for study. One of these streets, Pennsylvania Avenue between 17th and 18th Streets, was two-way. One link was studied on two consecutive days to observe the effects of a day to day stability. The specific links on which data were collected are shown on Table 28.

The turning movement analysis was expected to answer the following questions: 1) Does a sink or source add or subtract such a number of vehicles from the total link volume

Table 28. Location of sink/source turning movement count studies.

Number	Study Street	Direction of Travel	Nearest Cross Street	Number of Garages
1	18th Street	Northbound	K Street	2[1.]
2	21st Street	Southbound	L Street	1
3	Pennsylvania Avenue	Westbound	18th Street	2
4	Pennsylvania Avenue	Eastbound	17th Street	2
5	L Street	Eastbound	20th Street	1
6	L Street	Eastbound	19th Street	2
7	L Street	Eastbound	17th Street	2
8	L Street	Eastbound	15th Street	2

[1] Data collected for two days for each garage.

as to cause inaccuracies in the computation of volumes, and 2) How stable is the net addition of subtraction of vehicles within short time intervals? Table 29 addresses the first question and shows the net hourly addition or subtraction of vehicles to the traffic stream by the parking garages counted. This figure is indicated under the first column in each of the six time periods. A negative number implies that there are more vehicles entering the garage than leaving. This would be typical of the a.m. peak hour. The converse is generally true for the p.m. peak hour, shown by positive values. In the second column under each time period the ratio of the net addition or subtraction to the total volume upstream of the sink/source (in percent) is given. The denominator includes the through volume plus those vehicles turning into the garage.

In general, the highest percentages occur during the 8:00 to 9:00 a.m. peak period and 4:30 to 5:30 p.m. peak period. In some cases, such as on L Street at 19th Street, the heaviest influx of vehicles translates into approximately 10 vehicles per cycle. The peak values for the exiting of vehicles during the p.m. peak hour are slightly lower than the peak values for the a.m. peak hour. The percentages range as high as 17% in the a.m. peak period and 18% in the p.m. period.

Data taken during the off-peak periods indicated much less impact, as would be expected. During the lunch period there was a net influx of vehicles at most of the garages counted. This ranged from essentially a zero net change to an 18 vehicle net subtraction from link volumes over 1-1/2 hours. Even the highest figure is considerably less than one vehicle per cycle. During the hour of 2:00 to 3:00 p.m. there was a net addition to link volumes of, at most 25 vehicles for one garage. This is also a small number in comparison to the net change in the peak hour. The analysis of the two days of data at 18th Street indicates that there can be a significant fluctuation of volumes entering and exiting a parking garage from day to day. The difference is particularly evident in the 5:30 to 6:30 p.m. range although this may be partially due to the second day of data being collected on a Friday.

Figure 24 shows a plot of the net change in link volumes by five-minute time intervals on one representative link. It appears from the graphs that there is a generalized normal distribution of vehicle entry in the morning and vehicle exit in the evening. However the deviation from this distribution on a five-minute basis is quite high. A longer time average would have to be used before the

Table 29. Summary of sink/source data on an hourly basis.

Approach	Time											
	7 - 8 a.m.		8 - 9 a.m.		9 - 10 a.m.		3:30 - 4:30 p.m.		4:30 - 5:30 p.m.		5:30 - 6:30 p.m.	
	Net Change (vehicles)[1]	% of Total Upstream Volume	Net Change (vehicles)[1]	% of Total Upstream Volume	Net Change (vehicles)[1]	% of Total Upstream Volume	Net Change (vehicles)[1]	% of Total Upstream Volume	Net Change (vehicles)[1]	% of Total Upstream Volume	Net Change (vehicles)[1]	% of Total Upstream Volume
18th @ K St. NB (5/22/75)	-61	7	-146	12	- 70	8	+50	6	+ 95	11	+112	13
18th @ K St. NB (5/23/75)	-70	9	-139	12	-103	10	+36	4	+ 72	8	+ 61	8
21st @ L St. SB	-41	6	- 80	7	- 86	9	+23	3	+ 66	6	+ 61	6
Penn. Ave. @ 18th WB	-33	8	-128	17	-105	15	+30	4	+ 76	10	+ 55[3]	15
Penn. Ave. @ 17th EB	-32	6	-192	16	-179	16	+65	8	+119	12	+ 68[3]	14
L St. @ 20th EB	-29	3	-122	7	- 77	6	+19	2	+ 96	12	+ 40	6
L St. @ 19th EB[2]	-62	5	-205	10	-113	7	+25	3	+174	18	+122	16
L St. @ 17th EB	-97	14	-164	16	-116	14	+63	7	+129	13	+ 83	11
L St. @ 15th EB	-18[4]	4	- 75	6	- 63	6	+49	4	+ 56	5	+ 33	5

1 Minus sign indicates net subtraction of vehicles from link volumes; plus sign indicates net addition.
2 Includes 2 of 4 sink/sources on link.
3 From 5:30 to 6:00 only.
4 From 7:30 to 8:00 only.

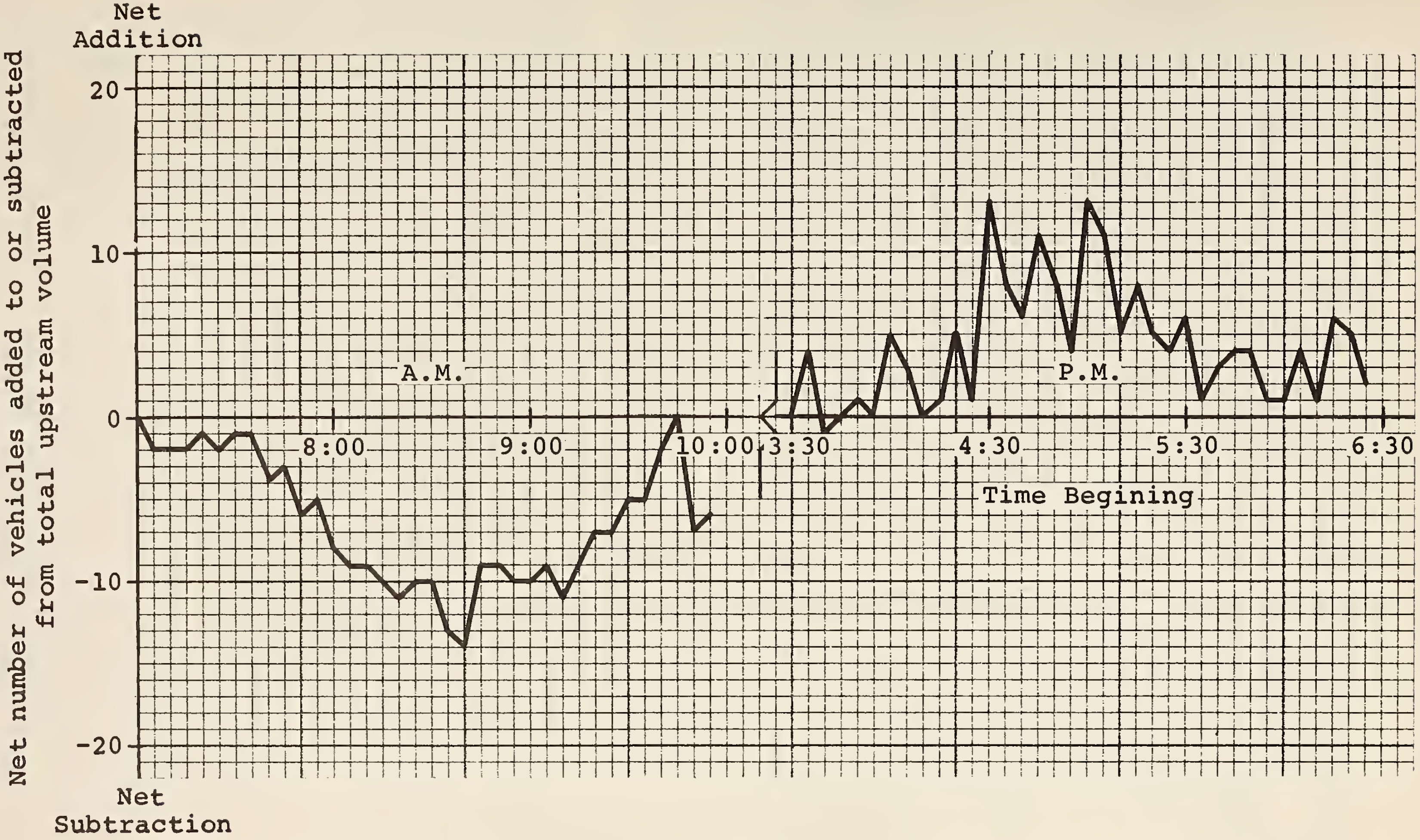


Figure 24. Time variation in volumes entering and exiting one parking garage on L Street eastbound at 20th Street.

patterns would stabilize.

This study shows that a sink/source significantly impacts the total link volumes. This impact must be considered both in light of the location of the sink/source, and in light of the particular control algorithm. Relatively little lane changing was observed immediately upstream from a sink indicating that drivers tended to select the proper lane for entering the sink either upstream from the link or as they entered the link itself. For sinks then, the impact is primarily on the curb lane traffic. The direct impact on the critical lane itself is discussed in the following section. When the garage if functioning as a source, vehicles tended to be proportioned to the turning movements at the downstream intersection. This point is developed more fully in the following section. The question of sink/source stability from day to day is heavily impacted by the sample time period. In general, it appears that the activity is too volatile to be corrected by a time-of-day factor. Again this question must be related to the effect on the critical lane which is the subject of the next section.

Impacts on the Critical Lane

Data collection for the third phase consisted of tracking vehicles exiting from a garage up to the downstream intersection. Two garages on two different links were selected for study during a 4:00 - 6:00 p.m. peak period. One observer was required for each location. Each observer would first note when a vehicle proceeded into the traffic stream from the garage. The lane in which that vehicle crossed the stop bar at the downstream intersection would also be noted. The number of such vehicles in each lane was then recorded on a cycle-by-cycle basis. By doing so, the additional demand exerted on the downstream approach by vehicles from the garage was determined. Specific attention was paid to the effect on the critical lane count.

As would be expected, it was observed during the course of the first two phases of data collection that by far the majority of vehicles entering a garage did so from the curb lane. The activity of vehicles exiting from the garage is somewhat less predictable. If volumes are light, vehicles tend to change lanes more than if volumes are heavy. The availability of turns at the intersection downstream of the sink or source also has a significant effect on the number of vehicles changing lanes after they enter the street. The third data collection phase was designed to determine the impact of vehicles exiting from a sink/source on the critical lane volume count under various conditions.

At the first location, a garage on the west side of 21st Street southbound at L Street, vehicles typically remain in lane 1 (right hand lane) after exiting. This fact is supported by the data shown in Table 30. Approximately 64% of the vehicles crossed the downstream intersection in lane 1 while about 18% utilized each of lanes 2 and 3. The greatest impact on lane 1 for any 15-minute period was an average of less than two vehicles per cycle. The maximum impact on the critical lane, lane 2, was an average of 0.5 vehicles per cycle for a 15-minute period.

Table 30 also displays the results of the analysis of a garage on the north side of L Street at 19th Street. The data reveals a strikingly different pattern from that at 21st Street. Most noticeable is the number of vehicles crossing all four lanes of traffic to turn right at 19th Street, approximately 38% of all those exiting from the garage. Lane 2 received nearly 17% of the vehicles while lanes 3 and 4 carried about 22% each. The highest 15-minute count was in lane 1, an average of 1.3 vehicles per cycle. The maximum value for a 15-minute period did not exceed one vehicle per cycle in any of the other lanes. These results also indicate the amount of lane changing which takes place in the distance between the garage and intersection. The majority of vehicles merged directly into the desired lane before proceeding a significant distance downstream.

In the p.m. peak hour, the paths of vehicles exiting from a garage is much less predictable. Even if a detector is in the curb lane downstream of the garage, there is no assurance that a majority of vehicles will be detected. From the count at the garage on L Street, it can be seen that vehicles may distribute themselves over a number of lanes. The effect on the critical lane is dependent on this distribution. At the two locations studied, the detectorized lane was impacted very little, less than one vehicle per cycle. Even the effect of an additional garage on either of the links would not seriously affect the UTCS algorithms. However, another link may find garage traffic merging into the critical lane in greater quantity. The impact on each link is largely dependent on the turning movements available downstream of the garage and the regional orientation of the traffic.

In summary of the third phase of the analysis, it appears that the impact of a sink/source on the critical lane volume count is site specific, depending on the location of the critical lane on the the prevailing traffic patterns. If the critical lane is not in the curb lane, virtually no impact is felt in the a.m. peak hour since

Table 30. Number of vehicles merging into each lane after exiting from a parking garage.

Time	21 Street SB @ L Street			L Street EB @ 19th Street			
	Lane Number			Lane Number			
	1	2	3	1	2	3	4
4:00-4:15	3	2	4	5	1	2	0
4:15-4:30	9	5	0	3	1	3	1
4:30-4:45	10	2	4	13	2	5	5
4:45-5:00	12	6	2	12	1	6	4
5:00-5:15	19	0	4	5	9	10	10
5:15-5:30	15	4	6	7	7	7	7
5:30-5:45	N.A.	N.A.	N.A.	15	5	5	5
5:45-6:00	N.A.	N.A.	N.A.	8	4	2	8

entries are generally made from that lane. If the detector is in the curb lane, but is downstream of the entrance, still no impact is felt. If the detector is located in the curb lane and upstream of the entrance the demand at the intersection will be overcounted on the a.m. peak hour, essentially by the number of entries into the garage. This is generally less than three to four vehicles per garage per cycle.

A final point to be considered in the location of a detector with respect to a sink/sourve is the effect of weaving maneuvers by exiting vehicles. If a detector is located in an area which is crossed by vehicles merging into other lanes, overcounting in that lane will occur. In addition, the likelihood of detector "clips" is greater.

Discussion of Observed Effects

Priorities for detector placement vary significantly from one sink/source location to another. Some of the factors involved include the location and number of garages on the link, location of the critical lane, size of the garages, availability of turns downstream of the garages and the regional orientation of vehicles using a particular garage. In general, the UTCS system critical lane volume counts at the Q2 location are not significantly impacted by sink/source considerations. Possible exceptions to this may occur where a.m. peak hour queues into a garage are frequent or where a high percentage of vehicles from a large garage merge into the critical lane downstream of a UTCS detector. Judging from this sink/source study, the exceptions are rare.

To optimize induced errors, a critical lane detector should be placed at least 50 feet downstream of the garage entrances. This allows enough distance for many of the exiting vehicles to merge into the desired lane before reaching the detector and without causing severe loop clipping problems.

UTCS/BPS DETECTOR RECOMMENDATIONS

The development of a complete detectorization plan for Sections 1 and 3 was conducted in two stages. The first stage consisted of identifying links that required additional loop detectors and recommending locations for those detectors within the links. A rationale for each group of the recommended additional detectors is presented and specfic comments are made concerning the operation of the detection system. The second stage consisted of a detailed field review of all existing detectors and suggestions are made concerning changes that should be made in the location of the loops if the detectors are to be used for the second and third generation control algorithms.

LANE DETECTION

A critical lane detection scheme is recommended with a single detector located 210 feet from the stop line. The use of a single detector is recommended based on the conclusions drawn in the Longitudinal Comparisons studies. These studies indicated that not only should a single detector at the Q2 location be used for volume and occupancy detection on additional links, but also the current UTCS practice of summing the pulses at Q1, Q2, and Q3 should be discontinued since the mean values are less representative of traffic flow than the single detector measures. This recommendation may have to be altered to include a Q1 detector dependent upon current queue algorithm development efforts.

ADDITIONAL DETECTOR RECOMMENDATIONS

Nine additional detectors are recommended for installation in Section 1. Eight of these detectors are required for CIC operation of congested intersection identified in the Cycle Failure Measure studies:

- M Street at Whitehurst Freeway

    1) Whitehurst NB lane 2
    2) M Street EB lane 2

- M Street at 34th Street

    1) 34th Street SB lane 1
    2) M Street WB lane 2

- M Street at 33rd Street

    1) 33rd Street SB lane 2
    2) 33rd Street NB lane 2

3) M Street EB lane 2
4) M Street WB lane 2

One additional detector is recommended on M Street for the arterial control algorithms LSTSQS and CYRANO:

- M Street at 29th Street

  1) M Street EB lane 2

Thirty-seven additional detectors are recommended for Section 3. Because virtually any intersection in this section can become critical during a typical peak hour, either as a source of congestion, or as an intersection impacted by a source intersection (a conlusion reached from the Cycle Failure Measure studies) it is recommended that every link internal to Section 3 be detectorized. This recommendation is based on the concern for the surveillance required for third generation CYRANO and CIC/QMC.

If only first generation or second generation were to be evaluated, then detectors would only have to be installed at the five additional intersections identified as "major." Fortunately, from an economic standpoint, the majority of internal links in Section 3 have adequate existing detectors. Additional detectors must be added on links that have been disrupted by METRO construction. In addition, H Street will require new detectors because of the new traffic operation plan of making H Street and I Street a one-way pair. This will change H Street from being one-way westbound to one-way eastbound thus rendering the existing detectors obsolete.

At certain locations, the Critical Lane Analysis indicated that the lane with the second highest volume exerted the major demand on a cycle-by-cycle basis for more than ten percent of the cycles. Because this condition was found to be prevelent on K Street, L Street, and 17th Street, detectorization of the alternate lane is recommended and software should be altered to automatically select the lane exerting the major demand.

On 16th Street between K Street and L Street, lane 2 was found to be critical during the peak hours. Because of frequent double parked vehicles, however, lane 3 carried the major traffic flow during non-critical hours. Since this link is relatively minor, it is recommended that a software change be implemented that would enable the use of historical data except during peak traffic flow times.

Additional detectors recommended for Section 3 are as follows:

- Eye Street/20th Street
    1) Eye Street WB lane 2
    2) 20th Street NB lane 2
- Eye Street/19th Street
    1) Eye Street WB lane 2
- Eye Street/18th Street
    1) Eye Street WB lane 2
- Eye Street/17th Street (west)
    1) Eye Street WB lane 2
- Eye Street/17th Street (east)
    1) Eye Street WB lane 2
- H Street/18th Street
    1) H Street EB lane 2
- H Street/17th Street
    1) H Street EB lane 2
- L Street/Connecticut Avenue
    1) Connecticut Avenue SB lane 2
    2) Connecticut Avenue NB lane 2
- K Street/Connecticut Avenue/17th Street
    1) Connecticut Avenue SB lane 2

Detectors for the alternate lanes on K Street, lane 1 for both EB and WB, and L Street, lane 3 for EB approaches, are recommended at the following intersections:

- 21st Street
- 20th Street
- 19th Street
- Connecticut Avenue
- 17th Street
- 16th Street
- 15th Street
- Vermont Avenue
- 14th Street (EB only on K Street)

Relocation of Existing Detectors

A field review of all existing detectors in Sections 1 and 3 was made to identify traffic operational problems associated with specific locations. Not all detectors could be reviewed because of recent pavement overlays or Metro construction. The latter item most affected Eye Street and Connecticut Avenue and 17th Street N. W. in the vicinity of Eye Street.

The field review was designed to identify problems such as placement within a line, side friction factors caused by major driveways, lane discipline, etc. The observations are, therefore, judgmental in nature but reflective of the objective data gathered as part of the overall task effort. The items identified are summarized in Table 31. The suggestions should be reviewed in the field with those responsible for system maintenance, for implementation decisions. The suggestions presume an overall detection logic similar to that now used and are intended for immediate implementation.

One comment does not fit into the tabular format and is presented here. Severe lane discipline problems occur in Section 1, especially on M Street west of Wisconsin Avenue. In this area, there are old trolley tracks and many sections of cobblestone. The striping and pavement markings are virtually non-existent. This leads to significant variation in placement of a vehicle in a lane(s). It is impractical to develop precise detector plans in this area until a method can be found to make striping and pavement markings more permanent. Further, the tracks should be removed so that lanes can be better defined. This problem is most evident at the "M" Street - Key Bridge Intersection where eastbound "M" Street must shift significantly as they proceed through the intersection. The overall problem is compounded with the lane reversals which occur on "M" Street in the morning and evening peak hours.

Table 31. Suggestions for immediate changes to existing detectors.

Street/Direction	Link Num.	Det. Num.	Observation	Suggestion
Key Bridge NB	2	44	The Q1 detector is at the stop bar.	Relocate 35 ft. southerly to be consistent with other Q1 locations
Key Bridge NB	2	46	The Q3 detector is located just beyond a major exit. Traffic does not "fill-in" until the Q2 location, distorting average counts.	Eliminate detector from processing.
Key Bridge NB	3	47	The Q1 detector is at the stop bar.	Relocate 35 ft. southerly to be consistent with other Q1 locations.
"M" Street WB	7	56	The Q3 detector is too close to the upstream signal. Lane changing and acceleration are prevelant.	Relocate 50 ft. westerly.
Wisconsin Ave. NB	8	57	The Q1 detector is too close to the parking area.	Relocate westerly to Lane 3 near centerline.
Wisconsin Ave. NB	8	58	Q2 detector, same above.	Same as above.
"M" Street	9	53	The Q2 detector is located in a parking space (peak hour through land).	The detector should be relocated 30 ft. easterly to a little used driveway or shifted to Lane 2.
Wisconsin Ave. SB	11	63	The Q1 detector is actually in Lane 3, the left turn lane and is too close to the lane line.	Relocate 2 ft. easterly.

Table 31. Suggestions for immediate changes to existing detectors (continued).

Street/Direction	Link Num.	Det. Num.	Observation	Suggestion
Wisconsin Ave. SB	11	64	The Q2 detector extends approximately 2 ft. into the opposing left turn area.	Restripe of relocate loop 3 ft. westerly.
"M" Street WB	17	71	The Q2 detector is located in a curve at the intersection. Poor lane discipline occurs in this area.	Relocate approximately 200 ft. easterly in Lane 2 or Lane 3.
Penn. Ave. SEB	18	72	The Q2 detector is not centered in travel path.	Relocate 2 ft. southerly.
Penn. Ave. SEB	52	75	The Q2 detector is located beyond a bridge in a lane offset to the right and in a future driveway.	Relocate laterally to Lane 3.
Penn. Ave. NWB	54	77	The loop is not centered in the travelled path.	Relocate approximately 2 ft. southerly.
25th Street	55	78	The loop is not centered in the travelled path.	Relocate approximately 5 ft. westerly.
Penn. Ave. SEB	56	79	The Q2 detector is in Lane 2 and is suspect for greatest activity.	Relocate to Lane 3.
17th Street NB	67	198	Q1, Q2 and Q3 detectors are in Lane 2 near parked cars.	Relocate detectors to Lane 3 near centerlin.
		199 200	In off-peak, traffic moves nearer the centerline.	

Table 31. Suggestions for immediate changes in existing detectors (continued).

Street/Direction	Link Num.	Det. Num.	Observation	Suggestion
22nd Street NB	120	313	The detector is not centered in the travelled path.	Relocate approximately 3 ft. westerly.
New Hampshire Ave. NEB	121	314	The Q2 detector is in Lane 2 which is for right turns only.	Review for count requirements to determine whether through or right turn movements are needed and relocate to Lane 3 for through movement.
L Street EB	122	315	The detector straddles 2 lanes (may be old WB detector).	Field check to determine if the detector is #315 which should be centered in Lane 2.
L Street EB	137	347	The Q2 detector is impacted by a high driveway.	Relocate approximately 50 ft. westerly.
L Street EB	137	348	The Q3 detector is as above.	Same as above.
L Street EB	143	356	The detector is impacted by a high volume driveway.	Relocate approximately 50 ft. westerly
L Street EB	146	361	The Q1 and Q2 detectors are impacted by a high volume driveway.	Relocate detectors approximately 25 ft. westerly.
19th Street SB	147	366	The Q3 detector is impacted by a high volume driveway.	Relocate approximately 50 ft. southerly.
L Street EB	151	375	The Q2 detector is impacted by a high volume driveway.	Relocate approximately 50 ft. westerly.

Table 31. Suggestions for immediate changes in existing detectors (continued).

Street/Direction	Link Num.	Det. Num.	Observation	Suggestion
Jackson Ave. NB	173	423	The Q2 detector is not centered in the travelled path.	Relocate to center of striped lane.
Connecticut Ave. SB	174	426	The Q2 detector is straddling two lanes.	Relocate approximately 5 ft. easterly.
Vermont Ave. SB	181	437	The detector is impacted by a high volume driveway.	Relocate approximately 50 ft. northerly.

APPENDIX A

APPENDIX A

DISCUSSION OF PROCEDURES

The data collection procedures are grouped into two categories, automated data collection and manually conducted traffic surveys. Both methods were used to support conclusions within the three categories forming the hierarchical structure of detector placement requirements (i.e. intersection, link, and within link). The data collection procedures used for the project are discussed below.

AUTOMATED DATA COLLECTION

Several sets of data were used in this analysis. The data were obtained either directly from detectors on the UTCS system or from the simulation of traffic within the network. These data bases and their uses are described below.

MOE Data

One source of data used in the detector placement study was the measures of effectiveness (MOE) data collected during the evaluation of the first generation control strategy. MOE data were used in each of the three detector requirement categories. The data selected for analyses were obtained the week of April 14, 1975, a time period in which system control was in the traffic responsive mode. A further discussion of the use of these data tapes is contained later in this chapter under computer processing.

Data Collection with Supplementary Detectors

To evaluate the benefits of various detector configurations within a link, it was necessary to supplement system detectors with temporary detectors on several of the links within Section 3. The data obtained from these detectors could then be compared to actual traffic conditions on the street to determine which configurations resulted in data most representative of real-world conditions.

To accomplish the above task, 62 temporary detectors were installed during the week of Monday, April 14, to Friday, April 18, 1975. The detectors were placed on ten links. Detector installation took place in two stages. The first stage took place between the hours of 10:00 p.m., April 13, and 6:00 a.m. April 14, during which time 37 temporary detectors were installed on six different links. In the second stage, 25 additional loops were installed on four links during the same hours on April 16 and 17. Also in the second stage, the loops placed on three of the original links were repaired and retained for use on the last two days of data collection. Links were selected for analyses based on suitability of the location by the availability of amplifiers and terminal connections in the local controllers.

For each link, detectors were placed at the Q1, Q2, and Q3[1] locations on all available lanes. System detectors were used where possible. In addition, multilane detectors (covering 2 or more lanes) were placed at selected locations, either immediately upstream or downstream of the Q2 locations. Overall, a total of 53 detectors were available for use in the first three days of data gathering and 67 were available during the final two. Detector configurations used on each link are shown in Figures 25 through 34.

Installation Procedures

The following paragraphs give a detailed description of the techniques and materials used in the detector installation process.

Placement of the loops was performed between 10:00 p.m. and 6:00 a.m. to avoid heavy traffic conditions. The loop detectors were taped to the roadway and connected to the existing amplifiers and communication equipment in the local controller cabinets.

The loops were installed and connected to the controller by two crews. One crew was responsible for actually placing the loops in the roadway and the other crew installed the lead-in cable and prepared the lead-in for connection to the amplifier. The procedures for both crews are discussed below.

Loop Placement Crew - The loop placement crew consisted of 3 persons equipped with the following materials:

14 AWG wire
wire cutters
electrician's tape
loop tape
loop jig
wire spool rod
identification labels
tape pressing boards
brooms
flares

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[1] See Figure 2 for detector designations.

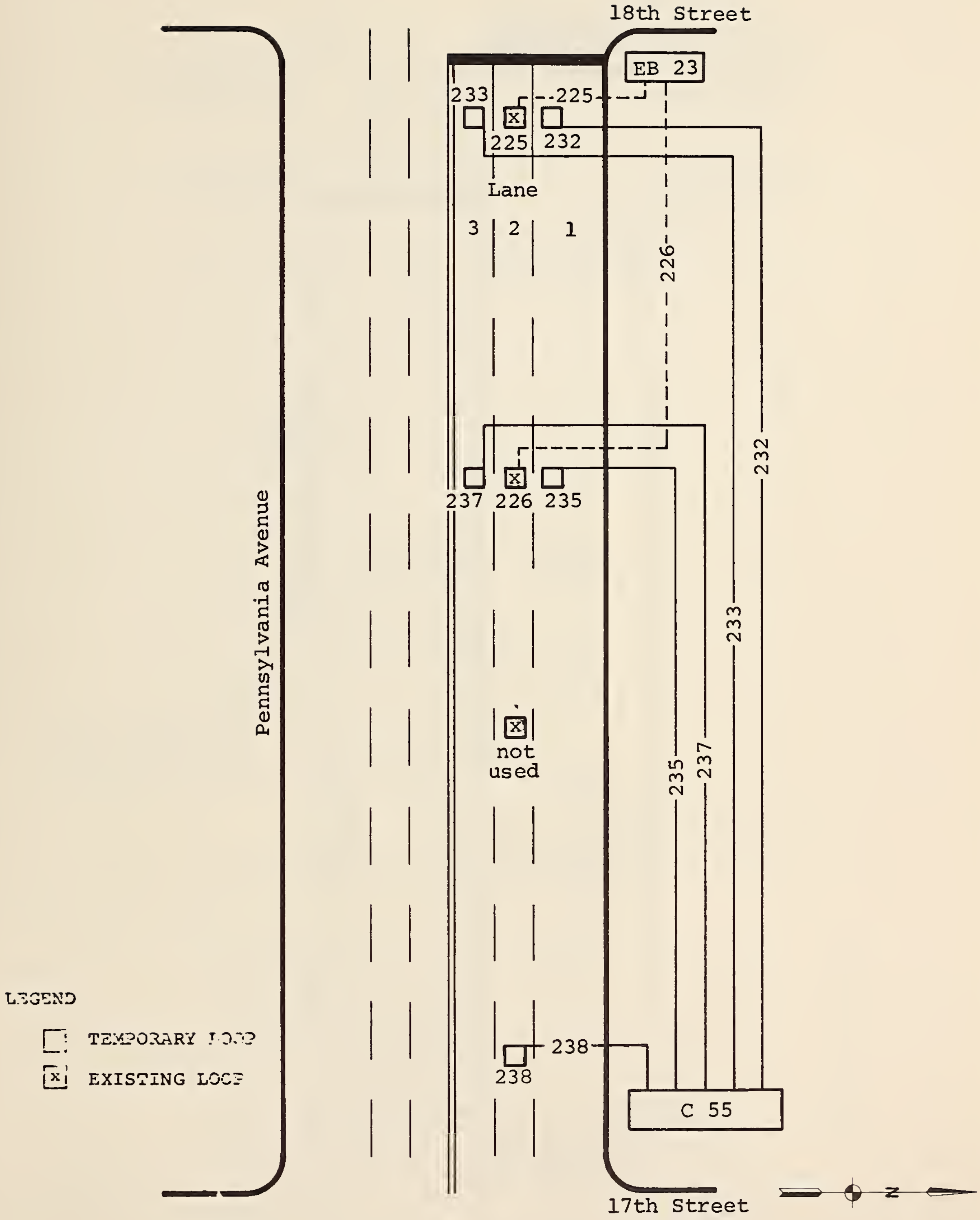


Figure 25. Detector configuration, Pennsylvania Avenue westbound between 17th Street and 18th Street.

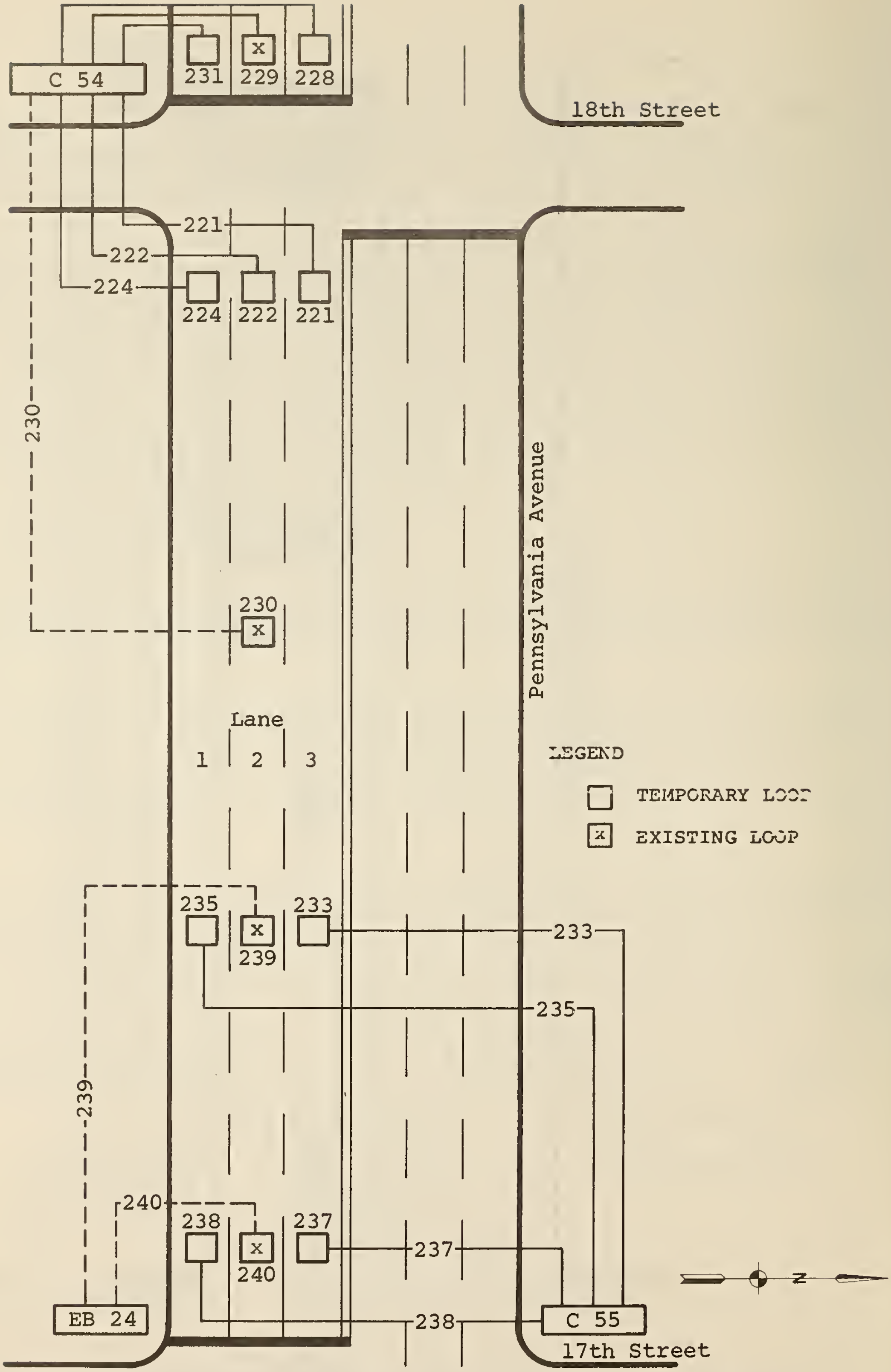


Figure 26. Detector configuration, Pennsylvania Avenue eastbound between 17th Street and 18th Street.

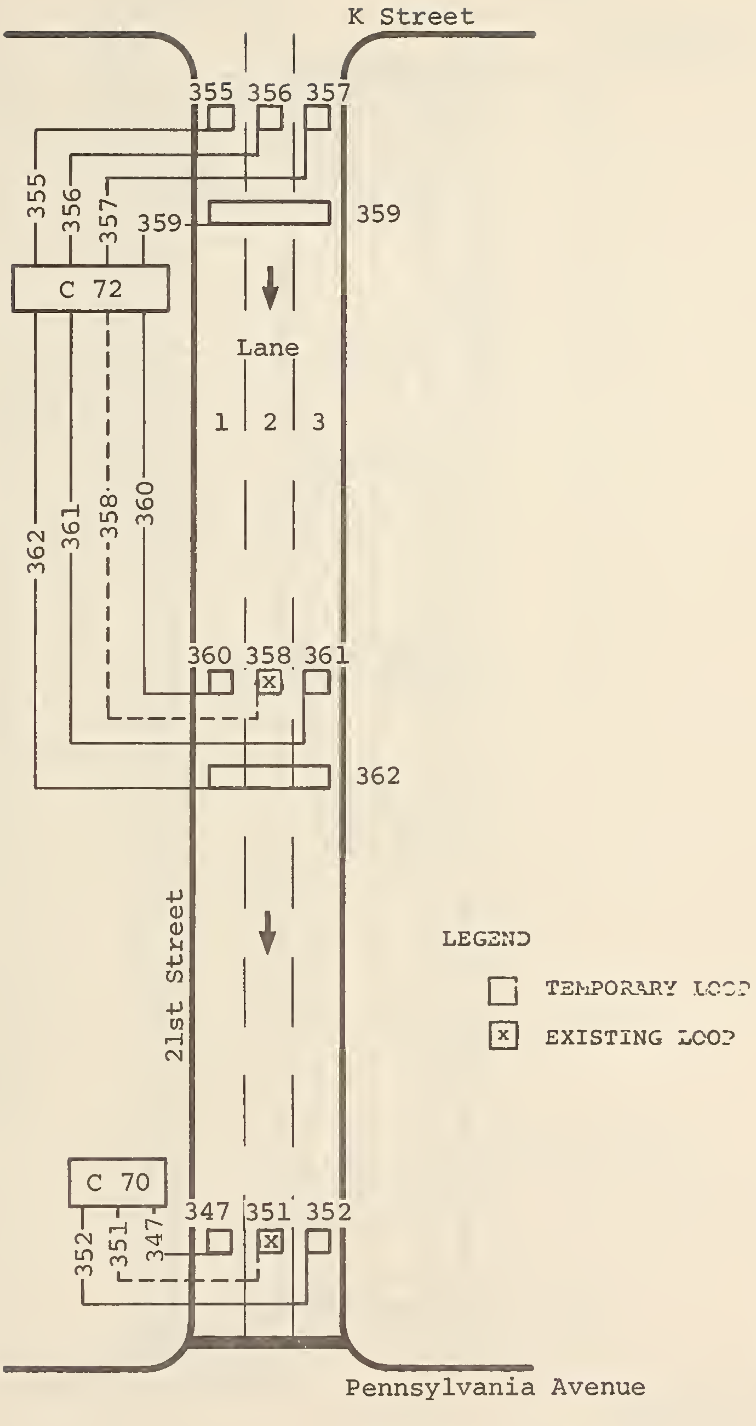


Figure 27. Detector configuration, 21st Street between Pennsylvania Avenue and K Street.

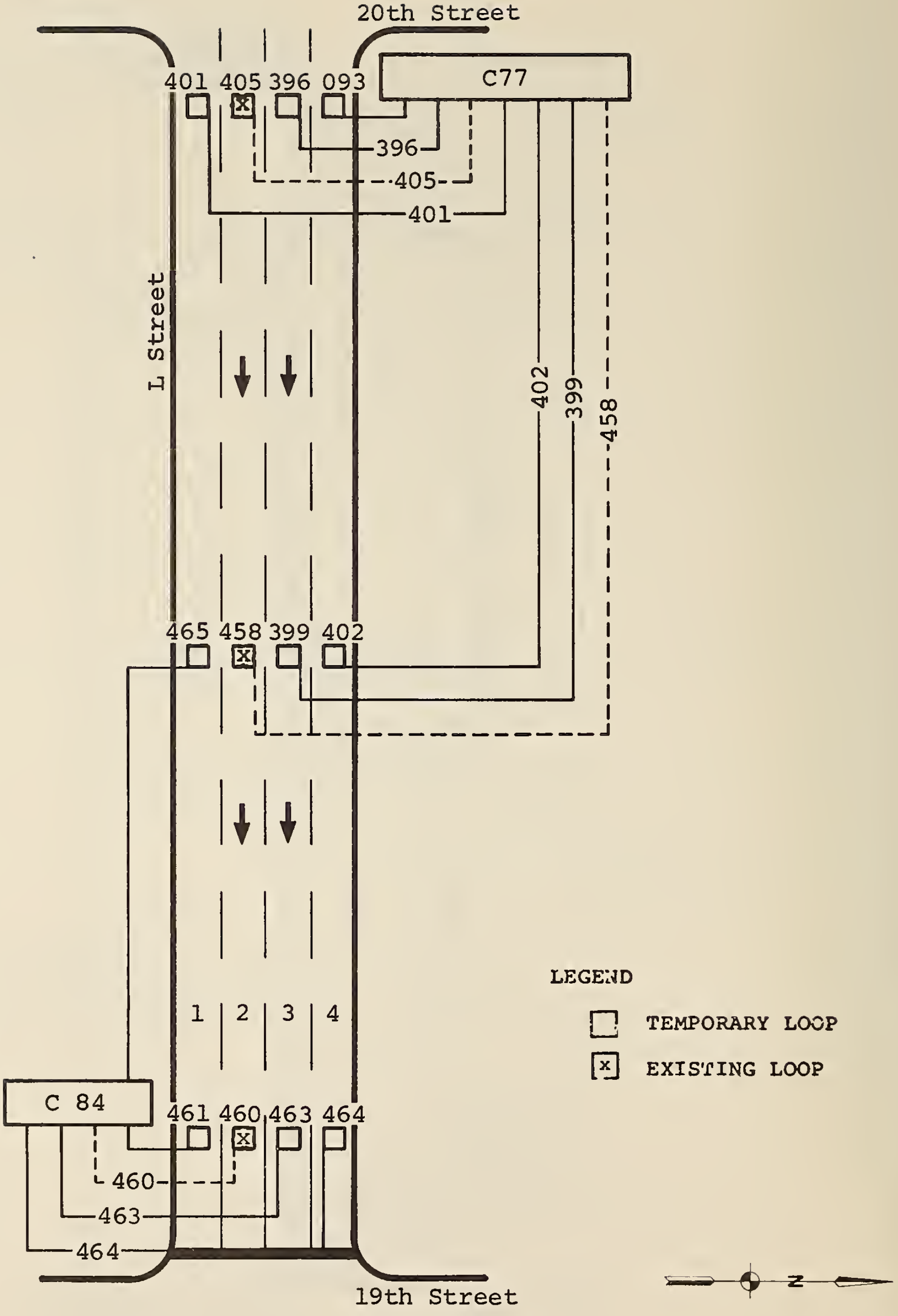


Figure 28. Detector configuration, L Street between 19th Street and 20th Street.

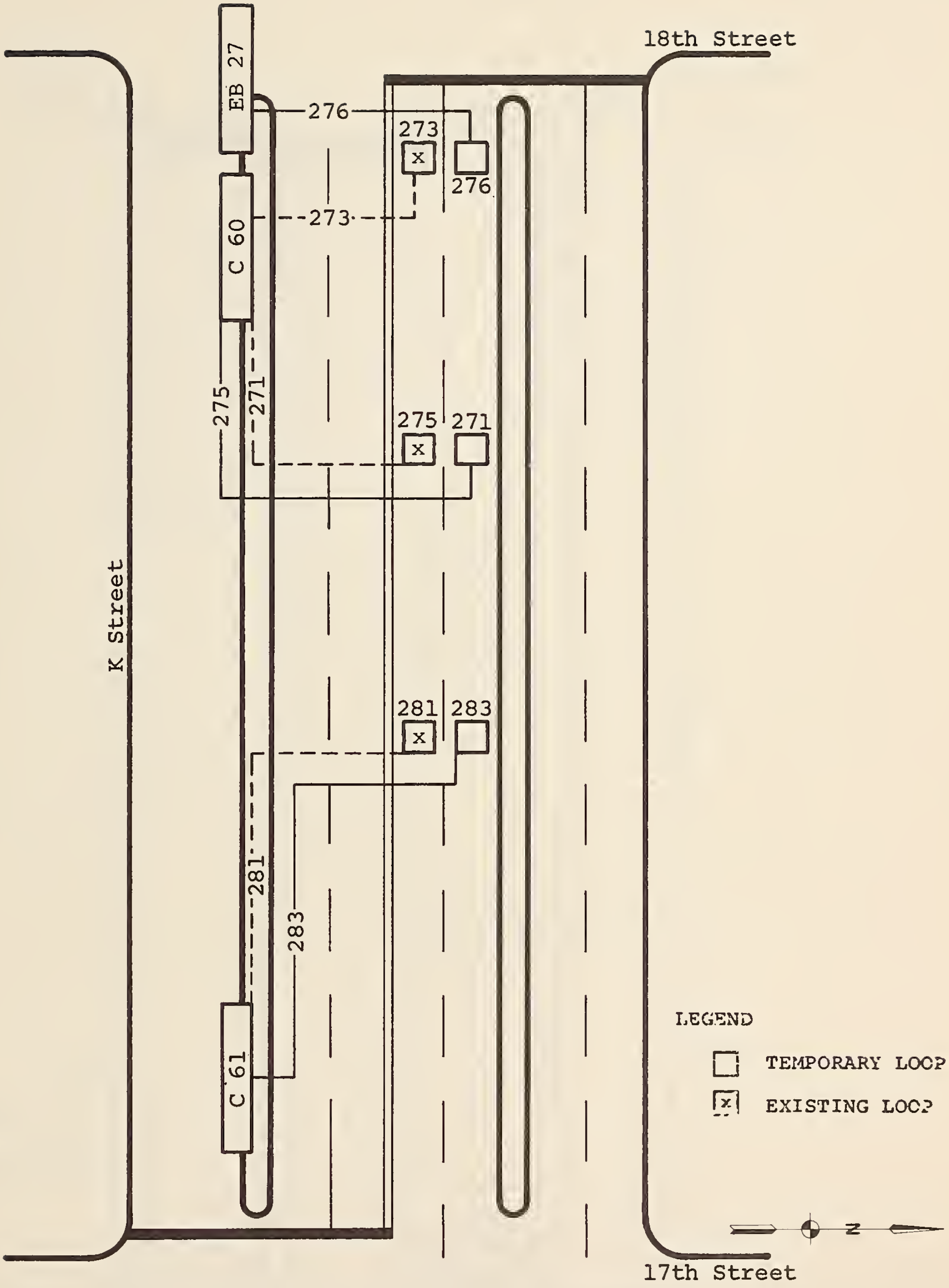


Figure 29. Detector configuration, K Street westbound between 17th Street and 18th Street.

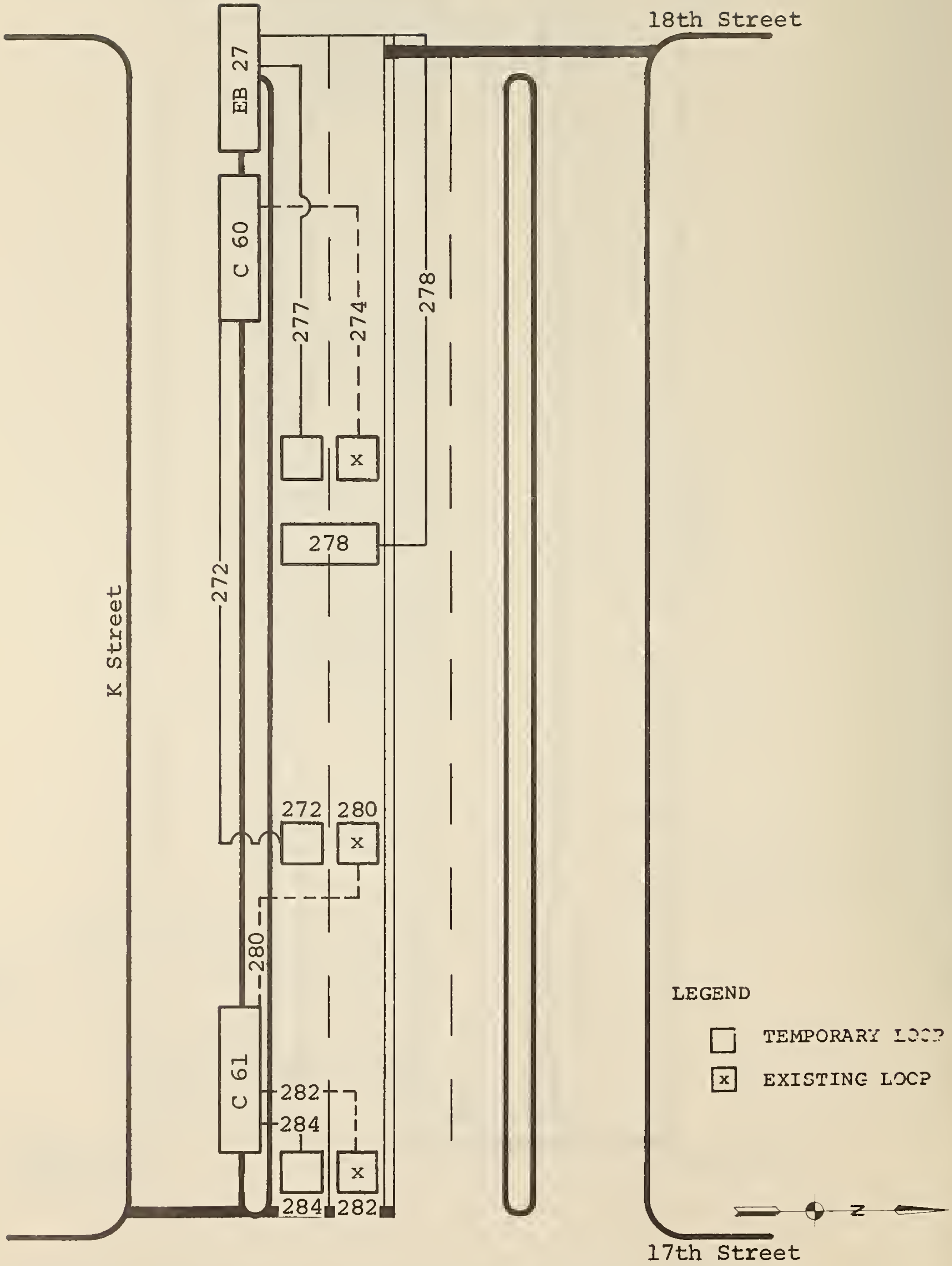


Figure 30. Detector configuration, K Street eastbound between 17th Street and 18th Street.

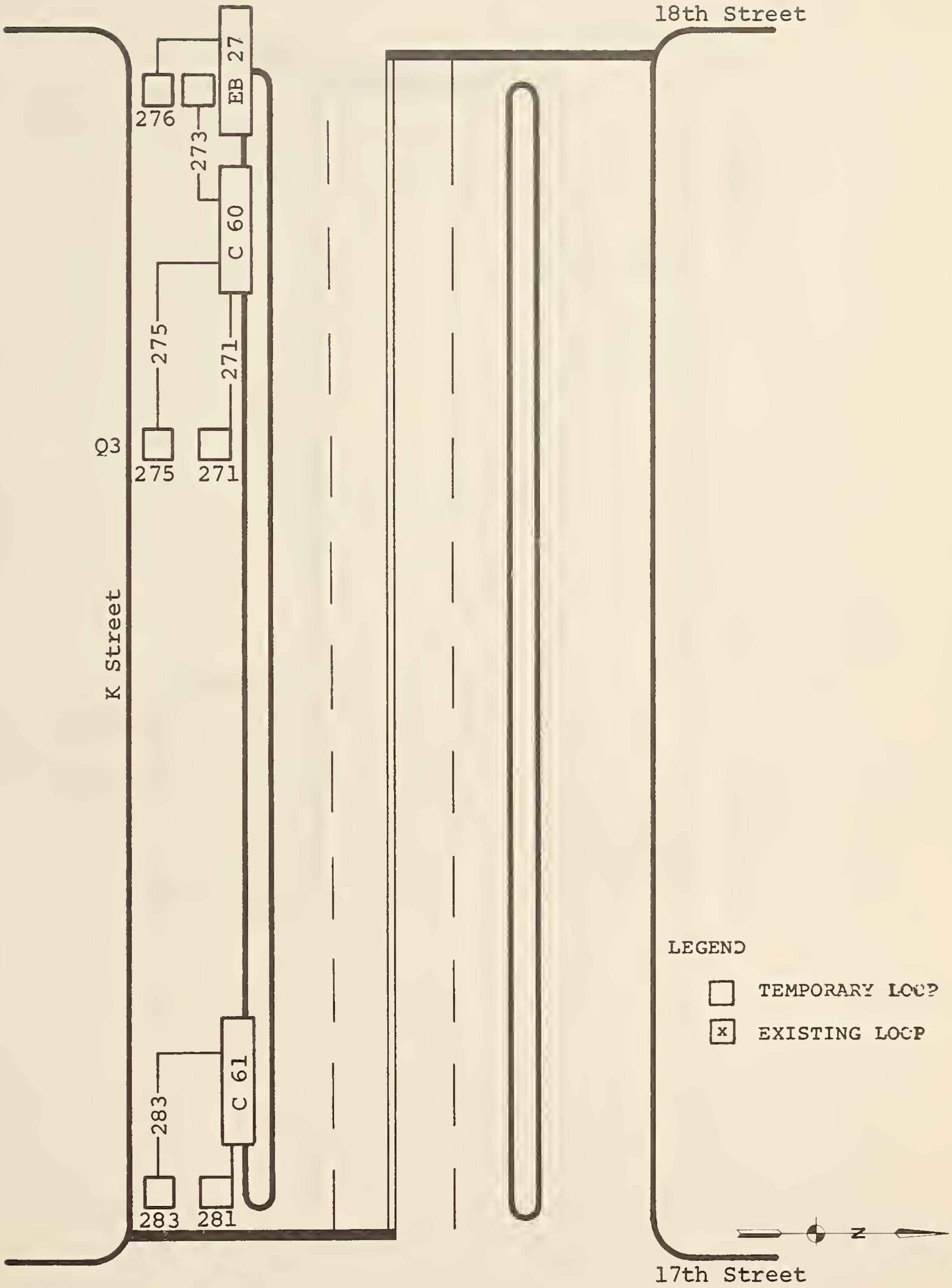


Figure 31. Detector configuration, K Street eastbound frontage road between 17th Street and 18th Street.

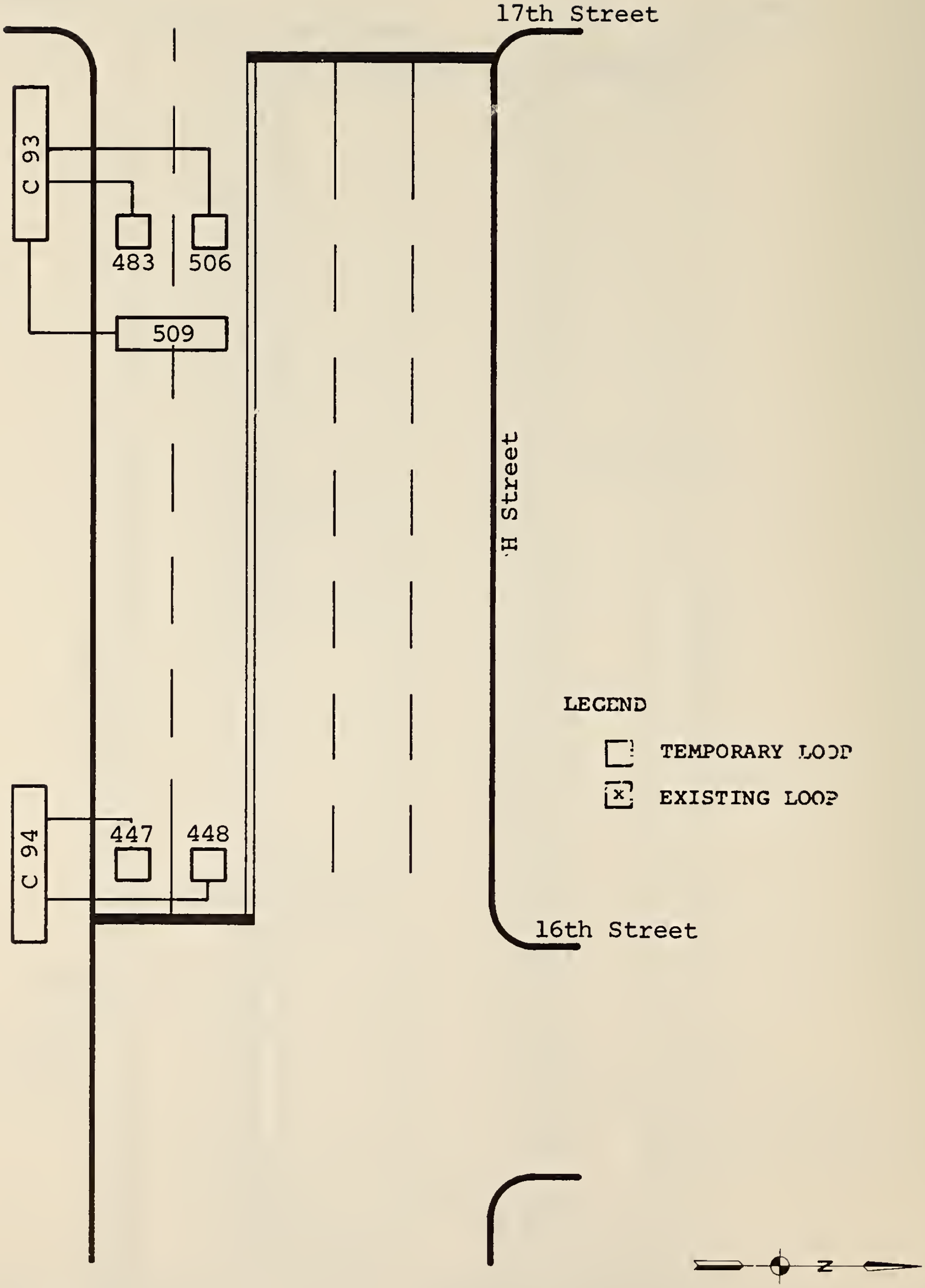


Figure 32. Detector configuration, H Street eastbound between 16th Street and 17th Street.

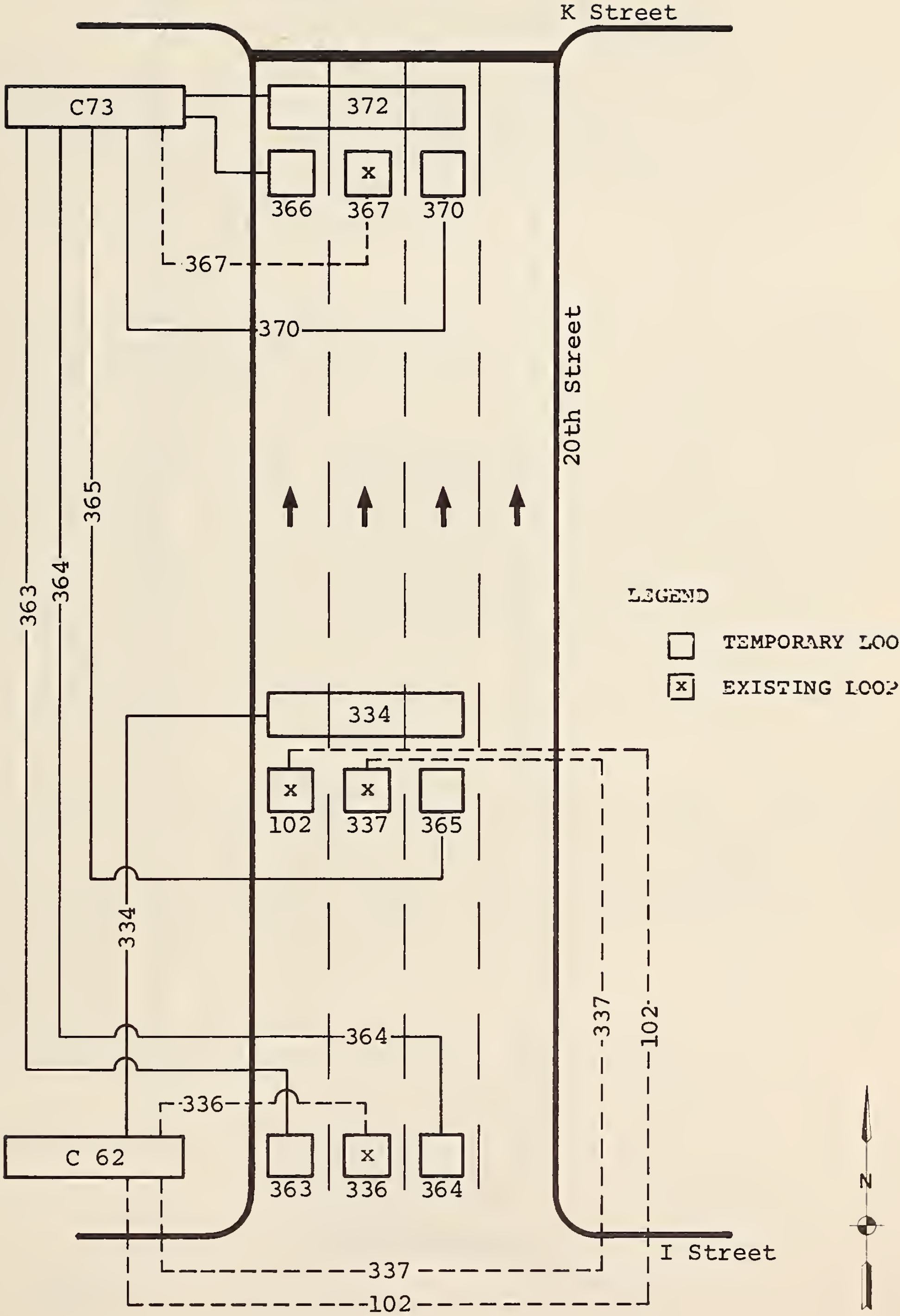


Figure 33. Detector configuration, 20th Street between I Street and K Street.

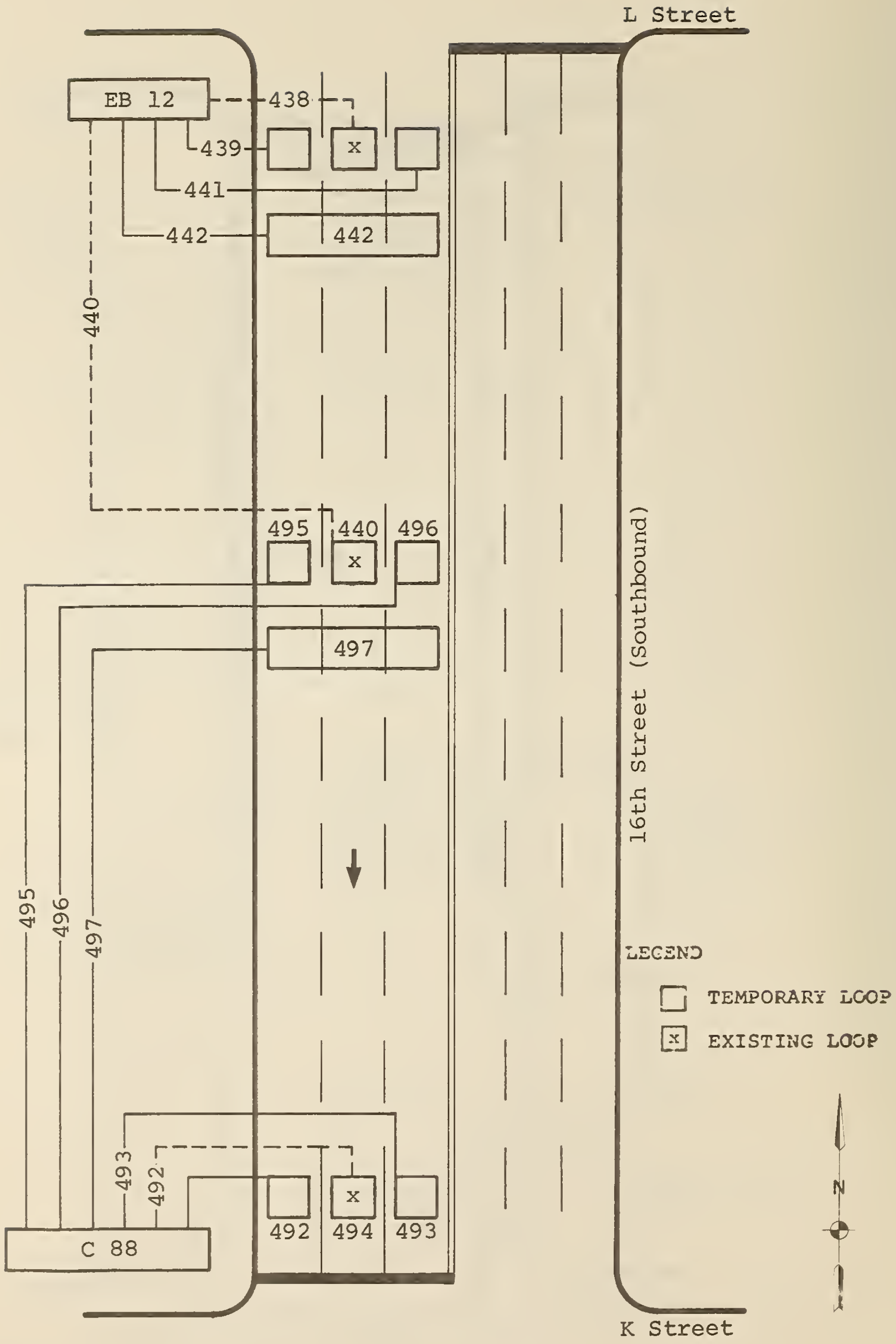


Figure 34. Detector configuration, 16th Street southbound between K Street and L Street.

traffic cones
safety vests
pickup or station wagon

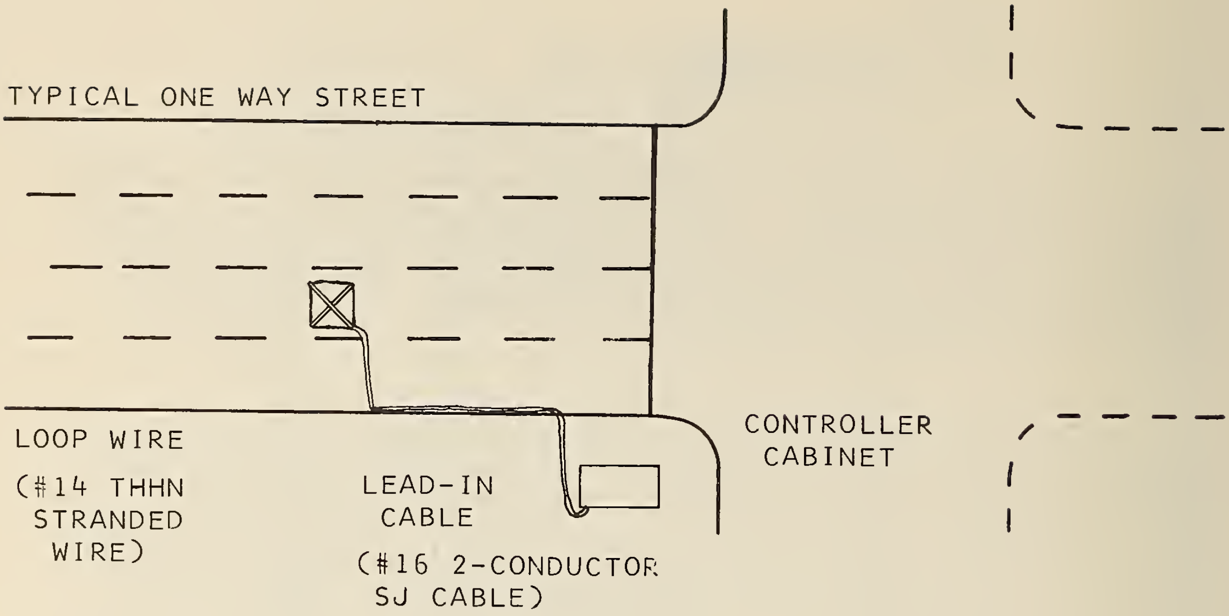
A special jig was constructed to facilitate the stringing of the loop wire. This jig was made with 2" x 4" lumber with two hinged sides. Typical layout of the loop and a sketch of the jig are shown on Figure 35. Use of the jig and the three-man crew was able to install a 6 foot by 6 foot loop in ten to fifteen minutes.

After the area has been thoroughly swept, the jig was placed in the lane and opened. One man would stand on the curb with the end of the wire. Another man would walk to the jig, unrolling the wire from the spool to measure the appropriate lead-in length to the curb. The third man would then hold the jig in place and also hold the wire tight at the starting corner of the jig. The man with the spool of wire then walked around the jig three times, looping the wire on the ends of the jig. Upon completing the three loops, two men would tape the three wires together, using electrician's tape, at approximately two-foot spacing, while the other twisted the lead-in wires, taped at two-foot spacings, and attached a label at the end denoting the sensor number to which the detector was eventually to be connected in the controller cabinet.

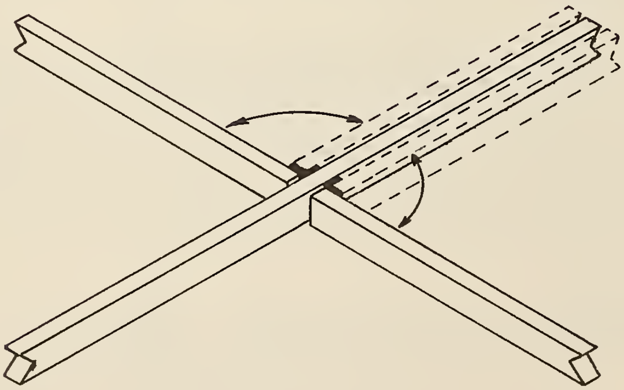
The loop was centered on the lane and taped to the pavement using Permacel, 2-inch waterproof cloth tape. Loop wires running parallel to the moving traffic required only 2" width of tape while the wires crossing traffic flow had one extra strip of tape on the leading edge. For obvious reasons, it was found best to use more tape if the loop was not very secure.

Lead-in Crew - The lead-in crew consisted of either 3 or 4 men equipped with the following materials:

14 AWG 2 conductor SJ wire
electrician's tape
loop tape
wire stripper
wire spool rod
identification labels
brooms
pickup or station wagon
twist caps
spade lugs
screwdrivers
crimping tool



TEMPORARY LOOP INSTALLATION



FOLDING JIG

Figure 35. Typical loop installation and diagram of installation jig.

This crew was responsible for laying the lead-in cable, from the loop to the controller and preparing it for connection to the amplifier. The connection itself was performed by a signal technician following the lead-in crew.

First the lead-in cable was spliced to the loop with twist caps and taped with a generous amount of electrician's tape as a safety measure. The opposite end of the lead-in cable was then labeled with the identical number as the loop to which it was being attached, and the cable was strung in the gutter from the loop to the controller. Spade lugs were attached to the other end of the cable for connection onto the terminal board in the controller cabinet.

Loop Connection

Following the laying of the lead-in cable, the system technician connected the spade lugged ends of the cable to the appropriate terminal. Existing detector loops on adjacent streets were disconnected and the temporary detectors were hooked up in their place. The amplifiers were then turned to the temporary loops. In most cases, the doors could be closed and locked. A tape seal was placed around the door for waterproofing. At the end of the data collection period each day, certain of the detectors were disconnected from the controller cabinets and left in the curb gutters in order to alleviate any hazards caused by cables crossing pedestrian paths. Each night, all splices were checked for continuity. Any loose detectors were retaped to the roadway, but generally only minor patching was involved.

After all data had been collected, the detector loops and lead-in cables were removed from the street. The lead-in cable was coiled in cable groups retaining the identification tags where possible. The tape was removed from the loops which were then coiled separately. All wiring was in excellent shape and was reuseable. The existing detectors in the street were reconnected to the amplifiers at a later time.

Materials Used

Approximately 14,000 feet of No. 14 THHN stranded wire was used for the 62 temporary loop detectors for an average of 225 feet of wire per detector. A total of 9,000 feet of two conductor No. 16 ST cable was used for the lead-in from the detector to the controller cabinet. Four cartons of Permacel tape, P-672, were used for the detectors and lead-in wire. A total of 34,500 feet of this two-inch wide tape was used for an average of approximately 550 feet per detector. Miscellaneous materials used on the project include approximately 30 rolls of electrician's tape, several boxes of spade lugs, and two boxes of twist nut electrical connectors.

Collection of Loop Data

An evaluation of the detector installation procedures indicated that the technique was quite successful. The loops tuned easily even with the long lead-ins and, using olive drab tape, the appearance was not too noticeable to the drivers. The tape generally held firm to the pavement up to a full week with only minor patching required. Wet weather on one day caused the tape to break loose on several detectors, which had to be abandoned, but a great majority of installations remained intact.

Data collection efforts using the installed detectors and a field crew at each detectorized location commenced on Monday, April 14. Two men were stationed at the UTCS control center along with the system operator to record raw pulse detector data onto magnetic tape. Prior to 7:00 a.m. each morning a magnetic tape would be loaded on the BPS tape drive. The drive was activated at approximately 7:10 and ran for approximately 35 minutes, recording pulses generated from all system and temporary detectors. After the first tape had finished another was loaded. A third tape was loaded once the second was completed, so that data collection ended at approximately 8:50 a.m. The five days of data collection produced 15 tapes containing the raw data history of traffic passing over each of the detectors of interest. All efforts were coordinated with the traffic surveys being conducted simultaneously on the street as will be discussed in the following section.

TRAFFIC SURVEYS

Link Volume and Queue

The first of the several traffic surveys conducted in this study was the measurement of link volumes and queues on each of the links that had been instrumented with additional detectors. The data obtained from the field survey was to be compared on a cycle-by-cycle basis to volume counts over the Q2 detectors and queue counts in the critical lane.

One team of field observers was assigned to each link. There were six teams on Monday, Tuesday, and Wednesday and seven teams on Thursday and Friday. Each team of observers was supervised by a permanent staff member and the team consisted of two or three temporary employees. The temporary employees were enlisted through notices posted in the student placement office of nine colleges and universities in the Washington Metropolitan area. The notices stated that persons were needed to observe and count traffic in the central business district of Washington, D.C. between the hours of 6:00 a.m. and 9:00 a.m. for four or five days beginning April 14, 1975. In total, the data collection effort used 19 temporary employees, seven permanent employees, and the principal investigator. Each team was composed of a downstream queue observer, and one traffic counter for each two lanes at the Q2 location.

Downstream Queue Observations

The crew supervisor served as the downstream queue observer. It was his responsibility to select the cycle to begin the study, generally at about 6:30 a.m. each morning. The crew supervisor started the stop watch on the beginning of green at the downstream intersection. At that time he would count the numbers of vehicles in the critical lane between the stop line and the upstream end of the Q2 detectors. At 20 seconds into the green period and at the beginning of yellow, the vehicles within this queue zone were recounted. All these data were recorded in the appropriate column on the recording form. During the red period, the supervisor reset his stop watch to zero and watched for the next green indication to repeat the above procedure. It was also his responsibility to report to the upstream queue observer and the volume counters as the beginning of the green occurred. It is important to note that the queue consisted of moving vehicles as well as stationary (link content).

Upstream Queue Observations

At the beginning of green, twenty seconds into green, and at the beginning of yellow, the upstream queue observer counted the number of vehicles in the critical lane between the upstream crosswalk and the Q2 detector. These data were recorded in a similar fashion to the downstream queue data.

Volumes Counts

The volume counter tallied the number of vehicles passing over the Q2 detectors by cycle, by lane, and by vehicle type. He was informed by the crew supervisor of the initiation of green on each cycle. Each cycle count ran from the beginning of green to the beginning of green. Some judgment was required on the part of each counter as to the lane assignment for vehicles making turning movements and lane changes. Vehicles were classified as shown below:

- Automobiles - Passenger cars, pickup trucks, small vans, motorcycles

- Trucks - Vehicles with more than four tires
- Buses - Any type of bus including city transit and interstate

During the data collection periods, the project director checked with each field crew at least three times. During each check, the director gave the crew supervisor the exact time at the beginning of one of the cycles, which was then recorded opposite that cycle number. The watch from which these times were taken and the digital clock on the UTCS display map were referenced to the correct time as given by the C&P telephone service. This was the all-important link between the detector data and the field data.

Once the data had been collected, it was returned to the office and coded onto forms in preparation for keypunching. A sample coding form is shown in the handbook. All coding was thoroughly checked after which it was delivered for keypunching.

Turning Movement Counts

The first method attempted for determining critical intersection locations was a simplified capacity analysis using turning movement counts. The practicality of this procedure is discussed later in the report. The turning movement counts were primarily obtained from the computerized file of counts generated for the UTCS-1 simulation studies. Others were obtained through the D.C. Department of Highways and Traffic.

Intersection Geometrics Inventory

A geometric inventory of the UTCS instrumented approaches was taken for use in the capacity analysis and in the portion of the study entailing link related detector requirements. A major portion of this task consisted of a photo inventory of the approaches.

On each approach photos were taken approximately 110' and 220' upstream from the stop line. At the 220' distance, the detector on queue instrumented approaches was just visible at the bottom of the camera viewfinder. Most pictures were taken on Sunday under light traffic conditions.

Each photo was identified as to approach, street, name, and direction of traffic, and a reference file was established for each intersection. The photos were useful as a reference for determining the number of lanes, CIC locations, etc.

Another data requirement in this category included the measurement of street widths. These were useful in the determination of pedestrian walk times at each intersection. This inventory was made from maps of the Washington, D.C. street system. The distances obtained were sufficiently accurate to calculate walk times to within 1 second.

Queue Length and Cycle Failure

This study was the second approach used to determine critical intersections within Sections 1 and 3 of the UTCS system by means of field observation. This provided a comparison with other means of determining critical intersections which used system data and was intended to provide verification for the location of intersections presently under CIC control. According to this definition, the two principal items of data required to identify a critical intersection are queue and cycle failures. A cycle failure was defined as any cycle in which there was a vehicular demand at an intersection approach when the signal indication changed to yellow. A queue failure was said to occur at times during which there were stationary vehicles within 50 feet of the upstream intersection.

To provide the above data the queue length and cycle failure criteria were examined in the field for all of the intersections in Sections 1 and 3. (Except for several intersections along I Street that were impacted by construction activities.) Data were collected during the hours of 7:30 to 10:30 a.m. and 3:30 to 6:30 p.m. using 12 observers, one at each intersection being examined. The observer was asked to record all cycle failures and queue failures for the two dominant conflicting approaches at each intersection. A mark was put in the appropriate column if a queue or cycle failure occurred on any lane of an approach. A cycle failure could be defined only at the time that the signal turned to yellow, but a queue failure could occur at any time during the cycle. All of the above data were collected between May 12-21, 1975.

Once stationed at the intersection, the observer would record the time that data collection commenced. The time was also recorded on each successive sheet so that the times and cycle numbers could be checked against each other. The observer would then judge which two conflicting approaches placed the most dominant demand on the intersection. Cycle and queue failures were marked across from the appropriate cycle number and approach for each three hour data collection period. If the dominant approaches shifted sometime during the data collection period the observer would observe the alternate approach. However, if dominance shifted frequently, only the approach which appeared to be more dominant would be examined. At least one supervisor circulated among the observers to insure uniform application of criteria.

Flow Discharge Rate

Data collected involving the discharge of vehicles from an intersection was used in the analysis of placement of detectors within a link. The data were collected on a cycle-by-cycle basis as follows. Two links were selected for microscopic analysis by means of sampling over the entire day. These two links were L Street eastbound at 19th Street, and Pennsylvania Avenue westbound at 18th Street. For these two links, 20 cycle samples were collected each half hour from 7:45 to 10:15 a.m., from 10:45 to 12:45 p.m., from 2:00 to 3:00 p.m. and from 4:00 to 5:00 p.m. These time periods were chosen as representative of the morning peak, morning off-peak and lunch hour, afternoon off-peak, and evening peak period conditions. This yielded data for approximately seven hours for each link for each day on these links. The data were used to determine if any change in critical lane was consistent by time of day over a period of days.

For the remaining 28 links, discharge data were collected for a 20 cycle period during either the a.m. or p.m. peak period but only when the discharge of vehicles was not influenced by congestion downstream from the intersection. This data allowed the selection of the critical lane to be made for one peak period for each approach.

Another function of the platoon discharge study was to provide the input necessary, along with primary and secondary volumes, to determine critical lanes. Inherent in the use of this data is the assumption that the rate of discharge is reasonably constant over time.

The data collection procedure for both the all-day and 20-cycle sample studies required one observer per lane stationed near the link stop line. The observer was equipped with a stop watch, recording forms, and a clip board. The watch was started when the green was displayed at the intersection. The following data were obtained: the time that the third vehicle cleared the stop line, the number of vehicles in the platoon, and the time that the last vehicle in the platoon cleared the stop line. The observer also recorded whether the phase was loaded and whether the discharge rate was influenced by congestion downstream of the intersection. Any data influenced by downstream congestion were not used in the calculation of platoon discharge time. For the purposes of this test, a platoon was defined as a group of vehicles in one lane with headways of five seconds or less as they passed the stop line. Also, if a vehicle was within 50 feet of the stop line and was either stopped or moving slowly, that vehicle would be included into the platoon even though his headway was greater than five seconds. All of the above data were collected in the months of April and May.

Primary Secondary Volumes

Primary and secondary volumes were used in several of the categories of detector location research conducted, as discussed elsewhere. For most of the uses, it was necessary to have the primary and secondary volume counts stratified by lane.

To obtain this data, volume counts by lane were taken for 30 representative approaches during a.m. peak, mid-day, and p.m. peak conditions. The counts were taken in close proximity to the upstream intersection so that the primary and secondary flows could be separately identified. Observers were equipped with counters equal in number to the lanes to be counted, recording forms and a clipboard. It was usually possible for one person to collect all the required data at each location. All samples taken were for 25 consecutive cycles. The morning counts were taken between the hours of 7:30 and 9:30 a.m., mid-day counts were taken between 1:00 p.m. and 3:30 p.m., and evening counts were obtained between 4:30 and 6:00 p.m.

Sink/Source Counts

The analysis of the effect of traffic sinks and sources (primarily parking garages) was approached from three perspectives. The first was the determination of the effect of a sink or source on magnitude of lane-changing by vehicles on a link. The second was the determination of the magnitude of change that a sink-source has on link volumes between points upstream and downstream of its entrance. The third was an attempt to translate the effect on total link volumes to the effect on the critical lane alone.

Data collection for the first phase, the lane changing analysis, took place during one morning peak hour on L Street eastbound at 19th Street. There are three major parking garages and one parking lot located on either side of this link. Two of these garages are located opposite one another approximately midway between the upstream and downstream intersections. These two garages were isolated for the study of lane changes produced by sink/sources.

Three data collectors were used, one at the garages, one upstream from the garages and one downstream. The person stationed at the garages counted the number of vehicles passing that point by lane and by cycle on L Street. Six columns on the data sheet were required to record all of the data, four for volumes on the through lanes and one column each for turns into the garages. A negligible number of vehicles were proceeding out of the garage in the morning peak hour and these were ignored.

The other two data collectors counted the number of lane changes caused specifically by vehicular activity at either of the garages while the other counted the downstream lane changes. As an example of the lane change, if a vehicle moved from lane 1 to lane 2 upstream of the sink and attempting to avoid a vehicle, it would be recorded by the upstream observer in the column designated for such a lane change. Six combinations of lane changes were possible among the four lanes on L Street: one to two, two to three, three to four, and their respective reverse movements. The other observer recorded the identical information when it occurred downstream of the garage. These data were recorded for 20 consecutive cycles during the period of heaviest inbound activity at the garage.

The second phase of the study involved counting the number of vehicles proceeding into and out of a number of the parking garages in UTCS Section 3. Fourteen garages on seven different links were selected for study. One of these links, Pennsylvania Avenue between 17th and 18th Streets, was two-way. The others were one-way links. In addition, one link was studied on two consecutive days to determine whether the impact of a sink/source would be somewhat stable from day to day. The specific links on which data were collected are listed below:

1. 18th (one-way) at K Street NB (2 garages, data collected for 2 days)
2. 21st (one-way) at L Street SB (1 garage)
3. Pennsylvania Avenue (two-way) at 18th WB and Pennsylvania Avenue at 17th EB (2 garages each side)
4. L Street (one-way) at 20th EB (1 garage)
5. L Street (one-way) at 19th EB (2 garages)
6. L Street (one-way) at 17th EB (2 garages)
7. L Street (one-way) at 15th EB (2 garages)

The data were collected in a form similar to a standard turning movement count. Where one garage was counted on a one-way street, three basic movements were examined: 1) the total through movement on all lanes excluding those vehicles turning into the garage; 2) the number of vehicles turning into the garage; and, 3) the movement turning out of the garage. On Pennsylvania Avenue it was necessary to record two additional movements, namely the left turn vehicles into the garage and the traffic volumes on the opposite side of the street from the garage. In general one person was assigned to one garage but where two garages were opposite one another, it was possible for one person to count both garages.

Data were recorded for the periods of greatest activity of the parking garages. In most cases, this was 7:00 to 10:00 a.m. and 3:30 to 6:30 p.m. For two of the links, Pennsylvania Avenue and L Street at 15th Street, the collection time was extended later in the morning and included lunch hour and mid-afternoon data collection periods. It was expected that from these two all-day samples the magnitude of activity during these off-peak periods could be quantified. Data were recorded for five minute increments during each of the above time periods.

Data collection for the third phase consisted of tracking vehicles exiting from a garage up to the downstream intersection. Two garages on two different links were selected for study during a 4:00 - 6:00 p.m. peak period. One observer was required for each location. Each observer would first note when a vehicle proceeded into the traffic stream from the garage. The lane in which that vehicle crossed the stop bar at the downstream intersection would also be noted. The number of such vehicles in each lane was then recorded on a cycle-by-cycle basis. By doing so, the additional demand exerted on the downstream approach by vehicles from the garage was determined. Specific attention was paid to the effect on the critical lane count.

APPENDIX B

APPENDIX B

DATA PROCESSING

A total of 12 special purpose programs were prepared for this portion of the detector placement study. In addition, the statistical post processor developed during the evaluation of first generation strategies was used extensively. In so far as possible, most of the programs were developed with a degree of generality, so that additional use could be made at another time or they could be adapted to other applications.

All of the programs that were executed multiple times were compiled into a load module named PMMCO.UTCS.PULSE LOAD stored on disk OTHERS at the FHWA/DoT computer center. The two "one-shot" programs, QUICK and PATVAR, were run as compile-link-edit-and-go procedures and load modules were not saved. The statistical post processor remained as program POSTPROC in load module PMMCO UTCS.LOAD2, also on OTHERS.

A brief User's Guide to each of the programs follows:

PROGRAM PULSE2

Function - Convert raw history tapes to MOEs for all detectors associated with a single controller and signal phase.

Operation

1. Read control cards including link identification
2. Read header (null operation at present) and skip first data record
3. Set up pointers for selected detectors
4. Read records searching for first AOG
5. A. Read for end of AOG if on at start
   B. Read for end of AOG if AOG not selected interval
6. Read for additional intervals until one desired
7. Initialize all tables
8. If detector 'ON' at start of data collection period, set flag
9. Check for start, end, or continuing vehicle
10. If continuing, increment pulse length
11. If start of vehicle, correct pulse length
12. If end of vehicle:
    A. Correct pulse length
    B. Compute raw MOE's
    C. Save speed correction if flag set
    D. Update queue counts
13. Print detailed summary if desired
14. Reset status words for next record
15. Check for AOG recurrence
16. Compute queue at 20 seconds
17. Read new record
18. If new interval:
    A. If end of green compute queue
    B. Write interval duration
19. If end of cycle:
    A. Update time
    B. Compute link statistics
    C. Compute final detector statistics
    D. Print output summary
    E. Write output data set
    F. Update queue for beginning of green
    G. Reset tables
20. Recycle until desired number of cycles or end of file

Special Requirement

PULSE2 uses an assembly language bit-shifting routine SLLR from the FHWA urban planning battery. This was incorporated by link-editting with an object deck.

Inputs

1. Unit: FT08
   Description: Raw history pulse tape
2. Unit: FT05
   Description: Control cards (follows)

Outputs

1. Unit: FT09
   Description: MOE summary (follows)

Control Cards

1. Label card
   Number: 1
   Format: 20A4
   Description: Any alphanumeric data
2. Program control card
   Format: Namelist
   Number: As required
   Description:

```
&PARAM            Namelist identifier
NC          = N CB number
NTP         = N Total number of intervals
NP          = N First interval for data;
                1 is beginning of AOG
ND          = N Number of detectors 1≤N≤20
NLN         = N Number of links 1≤N≤10
NCYCLE      = N Number of cycles desired
PRINT       = N Number of cycles for detailed
                printing, assumed 0
NIG         = N Number of intervals during AOG
NIS         = N Number of intervals for data
                collection
DS          = N Data set identicier (for concate-
                nated output files)
TIME(4)     = H,M,S,D Hour, Minute, Second, Half
                Second of start time
NI(ND)      = N Detector numbers
NPL(NTP)    = N Nominal interval lengths
&END              Namelist terminator
```

3. Link identifier cards(s)
   Number: NLNX
   Format: 20I4
   Description:
   Field 1: Number of detectors on link
   Fields 2 - NLNX+1: Detector serial numbers of the sequence specified in table NI; recorded from stopline upstream

Note: If NI = 137, 141, 162, 163, 191 and 162, 163 and 137 were the detectors on the link, the card would be coded: ---3---3---4---1

Output Data

1. Organization: 1 record/cycle
2. Form: Binary, combination of integer and real data as noted below
3. Fields (Variables):
   - A. Time (TIME)
     Number: 4
     Structure: Integer
     Description: hours, minute, second, half second
   - B. Data Set (DS)
     Number: 1
     Structure: Integer
     Description: User-supplied identifier from control card
   - C. Cycle Number (CYCLE)
     Number: 1
     Structure: Integer
     Description: Sequential cycle number
   - D. Length of cycle (LCYCLE)
     Number: 1
     Structure: Integer
     Description: Cycle length in 1/2 seconds (i.e., 160 is normal value)
   - E. Detector measures of effectiveness
     Number: ND sets of four
     - E1: Detector number (NI)
       Structure: Integer
     - E2: Volume (VOL)
       Structure: Integer
     - E3: Speed (SP)
       Structure: REAL
     - E4: Occupancy (OCC)
       Structure: REAL
   - F. Link measures of effectiveness
     Number: NLNX sets of three
     - F1: Volume (LVOL)
       Structure: Integer
     - F2: Speed (LSP)
       Structure: REAL
     - F3: Occupancy (LOCC)
       Structure: REAL
   - G. Queue Data
     Number: 9*NLNX PAIRS

Description: First of pair is queue with zeroeing at end of green; second is cumulative values

Organization: Three-dimension tables (see illustration)

4. Fortran Description

```
INTEGER TIME (5), DS,CYCLE,LCYCLE, NI(15), VOL(15),
LVOL(10), GA(10,3,3), QB(10,3,3)
REAL SP(15), OCC(15), LSP(10), LOCC(10)
TIME, DS, CYCLE, LCYCLE,
(NI(I), VOL(I), SP(I), OCC(I), I=1,ND),
(LVOL(J), LSP(J), LOCC(J), J=1,NLNX),
(((QA(K,L,M), QB(K,L,M), M=1,3), L=1,3), K=1,NLMX)
```

5. Queue Organization

Variable	Type	Time	Location	Link
1	Zeroeing	Beg.of green	Q1-Q2	1
2	Cumulative	Beg.of green	Q1-Q2	1
3	Zeroeing	20 sec.intogreen	Q1-Q2	1
4	Cumulative	20 sec.intogreen	Q1-Q2	1
5	Zeroeing	End of green	Q1-Q2	1
6	Cumulative	End of green	Q1-Q2	1
7-12	-	-	Q2-Q3	
13-18	-	-	Q3-Q4	(usually zero)
19-36	Repeat for link 2 (if required)			
37-54	Repeat for link 3 (if required)			
55-72	Repeat for link 4 (if required)			

6. Table of Parameters

Location	Controller	Number of Detectors (ND)	Number of Links (NLNX)
EB L @ 19	84	12	4
NB PA @ 18	54	7	3
SB 16 @ K	88	11	3
SB 21 @ PA	70	11	3
NB K @ 18	60	6	2
EB K @ 17	61	7	2
NB 20 @ K	73	8	2*
EB PA @ 17	55	14	3
EB H @ 16	94	5	2

* Only 1 link coded for April 16, 2 links for April 17 & 18.

7. Controller Sequence

Date	Controllers (CB)
April 14	84, 54, 88, 70, 61, 60
April 15	84, 70, 60, 61
April 16	84, 54, 73, 88, 70
April 17	84, 55, 54, 73, 94
April 18	84, 55, 54, 73, 94

Notes: Sequence shown in (7) above also holds for data sets created by programs ASSEM, CARDS and ASSEM2.

See figure 8 for controller locations.

PROGRAM ASSEM

Function - Select PULSE data for paired comparisons with field data.

Operation

1. Read header and control cards
2. Read lane and detector identification cards for each data set (street)
3. Initialize parameters
4. Read data from PULSE2
5. Extract volume data by lane, for total approach, and link volume for critical lane
6. Extract queue data
7. Adjust queue for count in Q0 if count in Q1 exceeds 4 vehicles
8. Set negative queue values to zero and sum for link content
9. Repeat (4)-(8) for each data set, incrementing output array
10. Write and print output array
11. Reset tables
12. Skip any unmatched PULSE records
13. Repeat (3)-(12) for 'B' part of data set
14. Repeat (3)-(10) for 'C' part of data set

Input

1. Unit: FT05
   Description: Control Cards (see below)
2. Unit: FT08 → FT13 (as required)
   Description: Data sets from PULSE2 for each controller (street) required

Output

Unit: FT23
Organization: Binary integer whole words
Description:

Field 1: Observation (cycle) number (sequential)
2→N+1 Lane volume, where N=Number of lanes on first link
N+2: Total approach volume
N+3: Link volume in critical lane
N+4: Downstream queue at beginning of green
N+5: Upstream queue at beginning of green
N+6: Total queue at beginning of green
N+7: Downstream queue at 20 seconds
N+8: Upstream queue at 20 seconds
N+9: Total queue at 20 seconds
N+10: Downstream queue at end of green

N+11: Upstream queue at end of green
N+12: Total queue at end of green
N+13 ETC. Repeat field 2-N+12 for each data set (street), generally with different values of N

Control Cards

1. Label Card
   Format: 20A4
   Number: 1
   Description: Any alphanumeric data
2. Program control card
   Format: Namelist
   Number: As required
   Description:

&PARAM		Namelist Identifier
NDS	= N	Number of data sets (streets)
NOBS 1(I)	= N	Value of number of matched observations from the 'A', 'B', and 'C' portions of the input data sets
&END		Namelist terminator

3. Lane and detector identification cards
   Format: 20I4
   Number: NDS
   Description:

Field 1: Number of lanes for output
2: Number of detectors
3: Total number of detectorized lanes (normally same as field 1)
4: Critical lane/link
5: Total number of observations on 'A' portion of data set, ≤ NOBS1(1)
6: Total number of observations on 'B' portion of data set, ≤ NOBS1(2)
7: Total number of observations on 'C' portion of data set, ≤ NOBS1(3)
8: Detector serial number for Q2 location in lane 1
9: Detector serial number for Q2 location in lane 2 (if required)
10: Detector serial number for Q2 location in lane 3 (if required)
11: Detector serial number for Q2 location in lane 4 (if required)

Note: See "Data Set Parameters" under program cards for further information and "note" under program PULSE2.

PROGRAM CARDS

Function - Convert field data into MOE's for paired comparisons with PULSE data.

Operation

1. Read label and control card
2. Read card data
3. Compute total volume across vehicle type for each lane
4. Compute total approach volume
5. Select volume in critical lane
6. Compute total link content
7. Perform (2)-(7) for each data set used as input
8. Write output data set
9. Print output summary

Input

1. Unit: FT05
   Description: Control cards (see below)
2. Unit: FT08
   Description: Card data for first data set (street)
3. Unit: FT10
   Description: Card data for first data set (street)
4. ETC., up to (theoretically) 10 data sets

Output

Unit: FT23
Organization: Binary integer whole words
Description:

Field	
1:	Observation number (sequential)
2→N+1	Total lane volume, where N=number of lanes for first link
N+2	Total approach volume
N+3	Total volume in critical lane fields 2-N+1
N+4	Downstream content at beginning of green
N+5	Upstream content at beginning of green
N+6	Total content at beginning of green
N+7	Downstream content at 20 seconds
N+8	Upstream content at 20 seconds
N+9	Total content at 20 seconds
N+10	Downstream content at end of green
N+11	Upstream content at end of green
N+12	Total content at end of green
N+13	Etc. - Repeat fields for each data set (street), generally with different values of N (See summary table below)

Control Cards

1. Label card
   Format: 20A4
   Number: 1
   Description: Any alphanumeric data
2. Program control card
   Format: Namelist
   Number: As required
   Description:

&PARAM		Namelist identifier
NDS	= N	Number of data sets (streets) on input
NQBS	= N	Number of observations (cycles)
NL(I)	= N	Number of lanes for each data set
CL(I)	= N	Critical lane for each data set
&END		Namelist terminator

Data Set Parameters

Date	Number of data sets (NDS)	Number of observations (NOBS)	Number of lanes (NL)	Total Variables
April 14	6	65	4,3,3,3,2,2	83
April 15	4	65	4,3,2,2	55
April 16	5	64	4,3,1,3,3	69
April 17	5	64	4,3,3,2,2	69
April 18	5	53	4,3,3,2,2	69

Notes

1. Further information on Field data set numbers, cycles selected for matching, etc., given in figure 6 in the previous section and under "note" in program PULSE2.
2. Critical lane is lane 2 except for third data set on April 16 when only 1 lane was used.

PROGRAM VPLOT

Function - Plot detector vs field volumes from summary data sets.

Operation

1. Read control card
2. Set counters
3. Read detector and field data
4. Store data for graphs desired
5. Read and store graph labels
6. Read alphanumeric number cards
7. Zero graph
8. Index counts in graph
9. Convert graph to alphanumeric form for printing
10. Print graph
11. Repeat (7)-(10) for each link graph
12. Perform (7)-(10) for each approach graph

Inputs

1. Unit: FT05
   Description: Control cards (see below)
2. Unit: FT08
   Description: PULSE data summary from ASSEM
3. Unit: FT10
   Description: Field data summary from CARDS

Outputs None except graphs

Control Cards

1. Program control card
   Format: Namelist
   Number: As required
   Description:

&PARAM		Namelist identifier
NOBS	= N	Number of observations (matched cycles)
NDS	= N	Number of streets on input data set, $1 \leq N \leq 6$
NLX(I)	= N	Number of links (lanes) on each street
LL(I)	= N	Location of input data set set of all link graphs, NDS + ΣNLX entries
LA(I)	= N	Location on input data set of approach totals; NDS entries
&END		Namelist terminator

2. Header cards
Format: 20A4
Number 1 per graph; 2*NDS + ΣNLX
Description: Any alphanumeric data; labels for all link graphs followed by labels for each approach graph.

3. Alphanumeric number cards
Format: 20A4
Number 4
Description: Integers 1-66, 20 per card, coded in columns 1-2, 5-6, 9-10, etc.
E.G. -1---2---3---4---5---6

PROGRAM QPLOT

Function - Plot detector vs field queues/content from summary data sets.

Operation

1. Read control card
2. Compute number of graphs
3. Read data and CB labels
4. Read alphanumeric number cards
5. Set up output labels
6. Set up location pointers & print result
7. Read detector and field data
8. Store data for graphs desired
9. Zero graph
10. Index & increment counts in graph
11. Convert graph to alphanumeric form for printing
12. Print graph
13. Repeat (9)-(12) for each graph

Inputs

1. Unit: FT05
   Description: Control cards (see below)
2. Unit: FT08
   Description: PULSE data summary from ASSEM
3. Unit: FT10
   Description: Field data summary from CARDS

Outputs None except graphs

Control Cards

1. Program control card
   Format: Namelist
   Number: As required
   Description:

&PARAM		Namelist identifier
NOBS	= N	Number of observations (matched cycles)
NDS	= N	Number of streets on input data set; $1 \leq N \leq 6$
NLS(I)	= N	Number of links (lanes) on each street
NV	= N	Total number of variables (less cycle number) on input data set
&END		Namelist terminator

2. Date and CB Labels
   Format: 20A4
   Number: 1
   Description:

cc 1-2	Date (14, 15, 16, 17, 18)
5-6	First CB number
9-10	Second CB number
Etc.	for NDS CB numbers

3. Alphanumeric number cards
   Format: 20A4
   Number: 4
   Description: Integers 1-66, 20 per card, coded in columns 1-2, 5-6, 9-10, etc.
   E.G. -1---2---3---4

Note: Bulk of graph labels, except for date and CB number, generated internally.

PROGRAM QUICK

Function - Create dummy LUSE and OBUSE vectors for POST runs.

Operation

1. Write 200 word OBUSE vector
2. Write 200 word LUSE vector

Input None

Output

1. Unit: FT09
   Organization: Binary integer half-words
   Description: 200 words with value of 1 to use as OBUSE vector for up to 100 observations for 2 alternatives.
2. Unit: FT11
   Organization: Binary integer half-words
   Description: 200 words with value of 1 to use as LUSE vector for up to 100 links.

Control Cards None

PROGRAM ASSEM2

Function - Assemble PULSE data for longitudinal analysis along links (lanes).

Operation

1. Read label and control card
2. Read detector specification cards
3. Read PULSE summary data from PULSE2
4. Build output data sets
5. Write output data sets
6. Print output data sets

Inputs

1. Unit: FT05
   Description: Control Cards (see below)
2. Unit: FT08 → FT13 (as required)
   Description: PULSE summary data sets for different streets (CB's).

Output

1. Unit: FT23
   Organization: Binary integer identifier, followed by binary real whole word data
   Description:
   Field 1: Observation number (sequential)
   2: Volume at Q1 location
   3: Volume at Q1 location
   4: Volume at Q2 location
   5: Speed at Q1 location
   6: Speed at Q1 location
   7: Speed at Q2 location
   8-13: As (2)-(7) for second street
   Etc. For all streets (see note below)
2. Unit: FT24
   Organization: As FT23
   Description:
   Field 1: Observation number (sequential)
   2: Volume at Q2 location
   3: Volume at Q3 location
   4: Volume at Q3 location
   5: Speed at Q2 location
   6: Speed at Q3 location
   7: Speed at Q3 location
   8-13: As (2)-(7) for second street
   Etc. For all streets (see note below)

Control Cards

1. Label
   Format: 20A4
   Number: 1
   Description: Any alphanumeric data
2. Program control card
   Format: Namelist
   Number: As required
   Description:
   &PARAM Namelist identifier
   NOBS = N Number of observations
   NDS = N Number of data sets (streets)
   NLX(I) = N Number of links on each data set
   &END Namelist terminator
3. Detector identification card
   Format: (20I4)
   Number: 1
   Description:
   Field 1: Q1 detector, data set 1
   2: Q2 detector, data set 1
   3: Q3 detector, data set 1
   4: Q1 detector, data set 2
   5: Q2 detector, data set 2
   6: Q3 detector, data set 2
   Etc. As required, for all data sets

Note ASSEM2 was run in three sets, first for critical lane, then for lanes 1, then for lanes 3 and 4.

Data Set Parameters:

Date	Number of observations (NOBS)	Number of Data Sets (NDS)			Tot.Number of Variables (incl. ID)		
		Run1	Run2	Run3	Run1	Run2	Run3
April 14	65	6	6	5	37	37	31
April 15	65	4	4	3	25	25	31
April 16	64	5	4	5	31	25	31
April 17	64	5	4	5	31	25	31
April 18	53	5	4	5	31	25	31

The following streets were included

Date	Run	Streets
April 14	1	L, PW, 16, 21, KE, KW
	2	L, PW, 16, 21, KE, Kw
	3	L, PW, 16, 21, L-lane 4
April 15	1	L, 21, KW, KE
	2	L, 21, KW, KE
	3	L, 21, L-lane 4

April 16	1	L, PW, 20, 16, 21
	2	L, PW, 16, 21
	3	L, PW, 16, 21, L-lane 4
April 17	1	L, PE, PW, 21, H
	2	L, PE, PW, H
	3	L, PE, PW, 21, L-lane 4
April 18	1	L, PE, PW, 21, H
	2	L, PE, PW, H
	3	L, PE, PW, 21, L-lane 4

L Street was duplicated on a temporary data set for run 3 so that both lane 3 and lane 4 data could be extracted.

Note See figure 8 for mnemonics

PROGRAM PULSE1

Function - Summarize raw history volumes and speeds.

Operation

1. Read control card
2. Read header and skip first data record
3. Set up pointers for selected detectors
4. Read records searching for first AOG
5. A. Read for end of AOG if'ON' at start
   B. Read for end of AOG if AOG not selected interval
6. Read additional intervals until one desired
7. Initialize all tables
8. If detector 'ON' at start, set flag
9. Check for start, end, or continuing vechicle
10. If continuing, increment pulse length
11. If start of vehicle:
    A. Correct pulse length
    B. If more than 5 seconds from end of previous vehicle, set 5 second flag.
    C. If more than 10 seconds from end of previous vehicle, set 10 second flag.
12. If end of vehicle:
    A. Correct pulse length
    B. Compute raw MOE's
    C. Save speed correction if flag set
    D. If 5 second flag set, update volume, speed and pulse history.
    E. If 10 second flag set, update volume, speed, and pulse history.
13. Print detailed summary if required
14. Reset status for next record
15. Check for AOG recurrence
16. If end of interval, write interval length
17. If end of cycle:
    A. Compute raw 5 and 10 second MOE's
    B. Compute final MOE's
    C. Print cycle summary
    D. Reset tables
18. If end of 4 cycle period:
    A. Compute cumulative volume and speed for 5 and 10 second gap MOE's
    B. Print summary
    C. Reset tables
19. If end of 11 cycle period, do as (18)
20. Recycle until desired cycles or end of file
21. Print 5 and 10 second gap pulse histories

Special Requirement

PULSE1 uses an assembly language bit-shifting routine SLLR from the FHWA urban planning battery. This was incorporated by link-editting with its object deck.

Inputs

1. Unit: FT08
   Description: raw history PULSE tape
2. Unit: FT05
   Description: control cards (see below)

Outputs

None except printing

Control Cards

1. Label card
   Number: 1
   Format: 20A4
   Description: Any alphanumeric data
2. Program control card
   Format: Namelist
   Number: As required
   Description:

&PARAM Namelist identifier
NC = N CB number
NTP = N Total number of intervals
NP = N First interval for data; 1 is beginning of AOG
ND = N Number of detectors, $1 \leq N \leq 20$
NCYCLE = N Number of cycles desired
PRINT = N Number of cycles for detailed printing
NIG = N Number of intervals during AOG
NI(ND) = N Detector numbers
NI(NTP) = N Nominal interval lengths
&END Namelist terminator

PROGRAM AGVS

Function - Compute 4-cycle and 11-cycle summaries or total volume and speed for all vehicles, to supplement data from PULSE1.

Operation

1. Read control card
2. Read first vehicle speed corrections
3. Index to proper place on input data set (if required)
4. Read summary data from PULSE2
5. Compute weighted average speed
6. Accumulate volumes
7. If multiple of 4 cycles, print summary
8. If multiple of 11 cycles, print summary
9. Continue for all cycles

Input

1. Unit: FT05
   Description: Control cards (see below)
2. Unit: FT08
   Description: Summary data from PULSE2

Output Nothing except printed report

Control Cards

1. Program control card
   Format: Namelist
   Number: As required
   Description:

&PARAM		Namelist identifier
NLX	= N	Number of links on input data set
ND	= N	Number of detectors on input data set
NOBS1	= N	First observation desired
NOBS2	= N	Last observation desired
&END		Namelist terminator

2. First vehicle speed corrections
   Format: 15F5.4
   Number: 1
   Description: Speed correction factor for each detector if it was "ON" at the beginning of data collection; values are reciprocal of pulse length from detailed PULSE1 printout; coded for each detector, in sequence.

PROGRAM PATVAR

Function

1. Compute means, variances, standard deviations, and variance to mean ratios for all field volume data by cycle and 4-cycle aggregations.
2. Dump all field data with 11-cycle averages.

Operation

1. Read control card
2. Read card data
3. Compute total lane volumes and total approach volume
4. Computes sums and sums of squares by cycle and 4-cycle aggregates
5. Compute volume and queue averages by 11-cycle periods
6. Dump data on cycle-by-cycle basis and 11-cycle aggregates
7. Compute means, variances, standard deviation, and variance to mean ratio by cycle and 4-cycle aggregates of volume by lane and approach
8. Print statistics
9. Repeat (2)-(8) for each data set

Input

1. Unit: FT05
   Format: Namelist
   Number: 1
   Description:

```
&PARAM                        Namelist identifier
 IY                     = N   Number of first data
                              set (usually set to 1)
&END                          Namelist terminator
```

2. Card data
   Unit: FT08
   Format: (I2, I6, I4, IX, ¢I3, 12I3)
   Number: As required
   Description: See figure 2

Output

None except printed summary

Control Cards

Only as input #1 above

PROGRAM HIST

Function - Format volume and occupancy from Phase I 15 minutes summary data.

Operation

1. Read LUSE vector
2. Read header record
3. Read volume record
4. Read occupancy record
5. Store selected values
6. Repeat (2)-(4) for all records
7. Compute average values for all days in each 15 minute period
8. Print output summary

Inputs

1. Unit: FT08
   Description: LUSE vector for an alternative/time period
2. Unit: FT10
   Description: Header records for an alternative/time period
3. Unit: FT12
   Description: Volume data (MOE2) for an alternative/time period
4. Unit: FT14
   Description: Occupancy data (MOE3) for an alternative/time period

Outputs Nothing except printed summary

Control Cards

None

PROGRAM SIMI

Function - Compute statistics and regression parameters for prespecified pairs of links, using the Phase I volume and occupancy data.

Operation

1. Read label and control card
2. Read pairs of links
3. Set flag if link appears in list
4. Read LUSE vector
5. Set second flag if LUSE indicates bad link
6. Initialize tables
7. Read volume data on first and third passes
8. Read occupancy data on second and third passes
9. Compute sums and sums of squares for each selected link
10. Compute cross products for each test pair
11. Compute means and standard deviations of both links in each test
12. Compute R, R^2, standard error, B-coefficient and A-coefficient
13. Compute T-statistic for B against value of 1 and T-statistic for A against value of 0
14. Print summary for each test

Inputs

1. Unit: FT05
   Description: Control cards (see below)
2. Unit: FT08
   Description: Volume data (MOE2) for an alternative/time period
3. Unit: FT10
   Description: Occupancy data (MOE3) for an alternative/time period
4. Unit: FT14
   Description: LUSE vector for an alternative/time period

Outputs None except printed report

Control Cards

1. Label
   Format: 20A4
   Number: 1
   Description: Time period label (AM,PM,or MD)
2. Program control card
   Format: Namelist
   Number: 1

Description:

&PARAM		Namelist identifier
NT	= N	Number of tests, $1 \le N \le 175$
&END		Namelist terminator

3. Pair specification

Format: 20I4

Number: (NT/20)+1

Description: Pairs of links for testing, 10 pairs per card

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