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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT **113**

OPTIMIZING FLOW ON EXISTING STREET NETWORKS

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT 113

OPTIMIZING FLOW ON EXISTING STREET NETWORKS

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EDWARDS AND KELCEY
NEWARK, NEW JERSEY

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
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WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
- TRANSPORTATION ADMINISTRATION
- TRAFFIC CONTROL AND OPERATIONS
- TRAFFIC FLOW
- TRAFFIC MEASUREMENTS
- URBAN TRANSPORTATION ADMINISTRATION

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1971

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NCHRP Report 113

Project 3-14 FY '68
ISBN 0-309-01903-6
L. C. Card No. 75-610979

Price: $15.60

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the Federal Highway Administration. Individual fiscal agreements are executed annually by the Academy-Research Council, the Federal Highway Administration, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of effective dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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2101 Constitution Avenue
Washington, D.C. 20418

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FOREWORD

By Staff
Highway Research Board

This report will be of special interest to traffic engineers, public works administrators, and other city officials interested in improving the traffic-carrying ability of city streets. The project is unusual in that it is one of the few research endeavors that has actually demonstrated methods of improving traffic flow on complex networks of city streets as compared with spot or arterial improvements. Dozens of traffic engineering improvements were implemented and evaluated. Newark, N. J., and Louisville, Ky., were selected as test cities for this $1 million study, which sought ways for middle-sized cities to expedite their local traffic without expensive reconstruction. The research clearly indicates through examples the benefits that can be achieved for the motoring public by application of traffic engineering knowledge to improve traffic flow and prevent costly travel delays.

A professionally produced, 20-minute, color motion picture (with sound) suitable for television audiences is available for use by state highway departments. The film, "Relief for Tired Streets," demonstrates for lay audiences the results that can be obtained by applying sound traffic engineering practices to urban traffic problems and is offered as a tool that can be used by highway departments to help explain the value of good traffic engineering to others.

A need exists to improve the traffic-carrying capability of existing urban street networks. Limited experiments have demonstrated that substantial improvements can be achieved by modifications of such factors as changes in speed of signal progression, turning and parking control, one-way streets, unbalanced lane operation, platoon dispersion, intersection controls, use of available green time, pedestrian controls, and minor geometric or structural changes within the right-of-way. The effects of these and other traffic engineering measures on the flow of traffic in the network are not clearly understood, and there are no accepted methods of measuring network efficiency. It was with these thoughts in mind that this project was initiated during the fall of 1967.

The primary objectives were (1) to develop a practical method of measuring the degree of change in network traffic flow resulting from various system modifications, and (2) to evaluate the degree of change resulting from practicable modifications. Expensive sophisticated systems for computer control of traffic and electronic guidance were not considered to be within the scope of this research endeavor because they are being studied elsewhere.

In this comprehensive and well-documented study, the consulting firm of Edwards and Kelcey conducted 37 major experiments to quantify the effect of traffic engineering measures. These experiments involved six major categories, as follows: directional control and lane use, curb lane controls, channelization, signal controls, inclement weather effects, and bus operations.

Study of the limitations of a direct capacity-volume approach for analysis of downtown traffic flows led to investigation of other means for quantifying and describing traffic flow in the downtown area. The research includes studies of

acceleration noise, mean velocity gradient, and travel time, together with several elements related to travel time, such as delay time, average speed, running speed, number of stops, and the number of saturated cycles at signalized intersections. Using the voluminous travel time and intersection study data accumulated on the project, regression analyses were performed to determine the relationships between various elements of travel time. It was also demonstrated that these relationships are fairly constant for arterial streets of the two study areas, in spite of their widely differing characteristics. The delay ratio—the ratio of delay time to total travel time—was developed, and is proposed by the agency to be used in a level-of-service definition for arterial roadways in downtown areas.

A statistical evaluation of flow data collected by the agency describes the variance and distribution of many elements of traffic flow. The report includes the effect of seasonal, daily, and hourly variations on traffic flow parameters.

A network analysis study was conducted to evaluate various models for use in analysis of downtown area traffic flows; validation tests were performed and a model is proposed for this use. A signal analog model was also evaluated for use in studying offset relationships between adjacent signals. This model, together with conventional time-space diagramming techniques and the FHWA-SIGOP program, was used to evaluate offset relationships between adjacent signals.

A fine-grain network assignment model was developed for the downtown Newark study area, using the Bureau of Public Roads' assignment system. The report describes the calibration and use of the model for analysis of the functional use of downtown streets. The authors suggest that the network assignment model be used to determine the over-all efficiency of the city street network. The over-all average travel speed developed from total trip time and total trip mileage is proposed to be used to develop a network level of service. It is anticipated that the network level of service may become a useful measure for determining priorities for the allocation of funds in relation to need.

Although this research has resulted in useful means for benefitting the motoring public, there is still room for much additional work. Future research could involve road use behavioral studies, enforcement of regulations, additional bus studies, development of a traffic data coordination system, and the extension of the level-of-service concept for downtown roads.

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 3-14 by the Traffic and Transportation Division of Edwards and Kelcey, Inc., under the direction of Wesley La Baugh, Vice President, and Walter E. Pontier, Associate, as Principal Investigator, assisted by Paul W. Miller, Project Manager and Walter H. Kraft, Assistant Project Manager. Important contributions also were made by Nathan C. Ficklin, Principal Engineer; Merwyn A. Kraft, Principal Engineer; Donald P. Downes, Senior Engineer; Wallace W. Sanders, Sr., Senior Engineer; and other staff members.

Consultants in special fields were T. T. Wiley and Associates, Gordon Gravelle, Dr. Walter Helly, and Dr. Thomas J. Boardman, Statistician.

In addition, the assistance of the following is gratefully acknowledged: Kentucky Department of Highways; New Jersey Department of Transportation; the staff of the Louisville Metropolitan Comprehensive Transportation and Development Program; the Departments of Traffic Engineering, Planning, and Police, and the privately owned transit companies of Louisville and Newark.

OPTIMIZING FLOW ON EXISTING STREET NETWORKS

SUMMARY

This project has investigated the benefits to traffic flow in downtown areas that can be achieved by application of traffic engineering measures. Experimentation to quantify the effect of road improvements was carried on in two study areas—the downtown portions of Louisville, Ky., and Newark, N.J. Data developed for control and analysis of these experiments were subjected to statistical evaluation to describe those controlling conditions that influence measurements in the downtown area. Meaningful relationships were developed that describe the quality of traffic flow, attaining a level-of-service definition for downtown streets. Methods were developed for application of the results of this research to streets of other areas.

Experimentation

Thirty-seven experiments were conducted to quantify the effect of traffic engineering measures. These experiments can be grouped into six major categories, as follows:

- Directional control and lane use.
- Curb-lane controls.
- Channelization.
- Signal controls.
- Inclement weather effects.
- Bus operation.

Initial investigations included systematic gathering of control data describing traffic flows in the area and operational studies to describe opportunities for meaningful experimentation. Improvements were designed, including description of the physical changes to be made, and the approval of city officials was secured. Experiments were designed, survey methods for description of "before" and "after" conditions were selected, and the experiments were implemented. Finally, the local effects, as measured by the survey data, were analyzed.

Comprehensive reports for each experiment are compiled as Appendix G and are summarized in Chapter Two under the heading "First Level Analysis."

Directional Control and Lane Use.—The portion of the experimental program involving directional control and lane use includes investigation of benefits of one-way street patterns, reversible lane operations, and various lane-control markings. In Louisville, Experiment E30, conducted at Mellwood and Story Avenues, and Experiment E31, which established a one-way rotary pattern of travel at River Road and Fort Nelson Way, showed substantial benefits because of the decreased conflicts resulting from one-way operation. For instance, at Fort Nelson Way, average vehicular speed was increased by 25 percent, from 13.8 to 17.2 mph; stops were reduced by 55 percent; and delay time was diminished by 65 percent. In Newark, Experiment B78, Broad Street Reversible Lanes (com-

bined with Experiment B100, Broad Street Signal Progression), showed substantial benefits resulting from reversing the center two lanes of a 0.6-mile section of roadway. Although trip time for the off-peak direction increased, computations show that the benefits to peak-direction traffic were almost five times as great as the loss in the off-peak direction.

Lane markings and other lane controls used to organize traffic were investigated in four experiments, which showed varying results. Experiment B86, which measured the effect of revised lane markings on Central Avenue in Newark, showed some benefits to the organization of traffic but little or no change in volume or speed during peak hours. Experiment A47, designed to increase the capacity of the critical Central Avenue and West Market Street intersection by providing an additional lane at the Central Avenue westbound approach, showed a reduction of saturated cycles from 97 percent to 4 percent for this approach and a corresponding reduction from 52 percent to 17 percent on the West Market Street northbound approach. It was during Experiment A47 that use was first made of delay time measurements; these are discussed in Chapter Four.

The intersection of Main and Third Streets in Louisville involves major turning movements. Experiment D67 investigated the benefit of lane-control markings and curb-mounted signs to organize traffic at this location. Violations of mandatory left-turn lane regulations were significantly reduced, resulting in better organization of traffic.

At Raymond Boulevard and McCarter Highway in Newark, separate left-turn lanes were provided in a lane-marking program that was implemented in conjunction with signal changes providing separate signal phases for turning traffic. Some benefits to flow were recorded in this experiment; however, much of the potential benefit was nullified by the high level of traffic violations. This experiment, more than any other, emphasizes the need for strict enforcement if traffic engineering modifications are to be successful in relieving downtown congestion.

Curb-Lane Control.—Five experiments, two in Newark and three in Louisville, investigated the results obtained from curb-lane parking and loading restrictions. The three Louisville experiments indicate that no significant advantage is gained by providing an additional lane at the approach to an intersection where traffic volumes are light. Public reaction to parking restrictions was very vigorous and indicates that extreme care should be taken in implementing such regulations. It was only in locations where traffic flow was experiencing some unusual delay that significant benefits were obtained. In Experiment A68 conducted at the Central Avenue-High Street intersection in Newark, where traffic volumes were much heavier and delays were severe, both the delay time and the number of saturated cycles were significantly reduced.

Experiment C123 investigated the benefits to be attained by truck loading restrictions on University Avenue in Newark, which eliminated a bottleneck at the University Avenue-Orange Street intersection. Travel time through this area was reduced as a result of this experiment.

Channelization.—Three experiments conducted in Louisville for the purpose of organizing traffic at locations where an expressway ramp joined the street system were successful in producing significant improvements to traffic flow. These experiments involved the ramps from I-65 to St. Catherine, Brook, and Jefferson Streets. A decrease in delay time of as much as 55 percent was realized at some locations during peak periods of flow.

At the intersection of Ninth and Jefferson Streets, channelization was used in conjunction with signs and lane markings to prevent wrong-way movement of

traffic between the two-way and one-way portions of Jefferson Street. A significant reduction of wrong-way movements was measured as a result of this experiment.

Experiment A33, Pedestrian-Vehicle Conflict Control, Market Street at Washington Street in Newark, investigated the use of painted islands and movable barriers in relocating pedestrian crosswalks to eliminate conflicts between pedestrians and turning vehicles. As a result, significant decreases in delay and frequency of stops were measured.

Signal Controls.—Eleven experiments, five involving single intersections and six investigating coordination of signals on arterials or in networks, were performed. As a result of these experiments it can be concluded that the signal system of a downtown area is the most important element of all the control media available. Very minor changes of signal timing produced large improvements to traffic flow. Implementation of signal progressions increased average speed of traffic sometimes more than 100 percent. Many other types of improvements should be considered subservient to the needs of the signal system. Among these are restrictions to parking at intersection approaches, locations of bus stops, provision of separate left-turn lanes, mandatory lane use, and channelization—all of which, at least to some degree, are involved in optimizing the signal system. The ability to organize successful progressions depended largely on the reduction of frictions by use of these other control media. If parking regulations, turning movements, conflicts at critical intersections, etc., were not controlled, platoons would rapidly disperse because of these frictions. Maintaining traffic platoons in a signal progression also depends on successfully adjusting the speed of progression to the speed that can be maintained by most vehicles in the platoon—the fastest vehicles being slowed down while only the stragglers are cut off. In Experiment B88, Springfield Avenue Signal Progression, platooning of traffic was not as successful at 30 mph as at 25 mph. Evidently, too many drivers found it difficult to maintain the higher speed.

Inclement Weather Effects.—Comparisons of travel time, delay time, and number of stops for clear weather and inclement weather conditions were made in two experiments, one conducted in Newark, the other in Louisville. In the Louisville experiment conducted on Walnut and Liberty Streets, traffic volumes were light in comparison to the Newark experiment on McCarter Highway. In this lighter traffic, the effect of inclement weather was to increase the number of stops, indicating that average speeds were reduced and that more drivers found difficulty in maintaining the designed speeds of progression. A small reduction in speed of progression probably would be desirable under these conditions. In the McCarter Highway experiment, a significant increase in measured travel time, delay time, and number of stops could be attributed to inclement weather.

Bus Operation.—Three major subjects were investigated in the field of bus operations—the optimum location of a bus stop, passenger service operations, and the organization of a major bus stop.

Studies of the optimum location of a bus stop demonstrated significant advantages for far-side stop locations if signal delay was not encountered at the intersection. Near-side stops held an advantage if signal delay was incurred, due to the fact that a certain amount of delay time could be allocated to passenger service operations. This finding indicates that the time position of a bus with respect to the green-time band of the coordinated signal system should be a subject when designing a bus route. Studies of passenger service operations produced equations and graphs for the estimation of time required for boarding and alighting of passengers. If the average number of passengers boarding and alighting

at a given stop is known, the range of passenger service operation time required may be estimated from these graphs. A 3½-block portion of Market Street was organized as a downtown bus stop, servicing more than 100 buses per hour in the afternoon peak period. Nearly 1 min per bus was saved and all other traffic through the area was expedited.

Analysis of Flow Data

Consideration of the limitations of a direct capacity-volume approach to analysis of downtown traffic flows led to investigations for developing other means for quantifying and describing traffic flow of a downtown area. These included studies of acceleration noise, mean velocity gradient, and travel time, together with several elements related to travel time, such as delay time, average speed, running speed, number of stops, and the number of saturated cycles at signalized intersections. These analyses indicated that a comprehensive analysis of travel time was the best medium for understanding and classifying traffic flow of the downtown area. Using the voluminous travel time and intersection study data accumulated on the project, regression analyses were performed to demonstrate the relationships that exist between various elements of travel time. It was also demonstrated that these relationships are fairly constant for arterial streets of the two study areas, in spite of their different characteristics. The delay ratio (the ratio of delay time to total travel time) was developed and used in a level-of-service definition for arterial roadways of the downtown area.

A statistical evaluation of flow data described the variance and distribution of many elements of traffic flow. This study also described the effect of seasonal, daily, and hourly variations of traffic flow, developing information for control of surveys in the downtown areas.

Network Analysis

A study was conducted to evaluate various models for use in analysis of downtown area traffic flows. As a result of this study, Newell's Intersection Model (see Appendix B) was selected for use in estimating delays at an intersection. Validation tests were performed and the model was accepted for this use. The Signal Analog Model (see Appendix D) was used in studying offset relationships between adjacent signals. This model, together with conventional time-space diagramming techniques and the SIGOP program, was used in optimizing signal relationships for downtown networks. The major benefit experienced from use of this model was that the network offset relationships are made visible to the designer in three dimensions, so that the effect of any adjustment may be immediately seen at adjacent intersections.

A fine-grain Network Assignment Model was developed for the downtown Newark study area, using the Bureau of Public Roads' assignment system. This model was calibrated and was found to be useful for analysis of the functional use of downtown streets. In accuracy, this model is comparable to similar models commonly used for analysis of urban area traffic problems. The development and calibration of this model is described in Appendix C.

The Network Assignment Model may be used to determine the over-all efficiency of the network. The over-all average travel speed developed from total trip time and total trip mileage outputs of the network can be used to develop a network level of service. For PM peak hours, the Newark network indicates a level of service D; using inventoried data from travel time and traffic volume surveys,

the Louisville network indicates a PM peak-hour level of service C. Using the criteria of Table 10.14—Levels of Service for Downtown Streets, of the *Highway Capacity Manual* (p. 334), it is anticipated that the network level of service may become a useful measure for determining priorities for the allocation of funds in relation to need.

Convenience and Safety

Throughout the initial operational studies and the experimental program, observations were made concerning the convenience and safety of the traveling public. In the initial studies an accident analysis was performed for each study area. This analysis was used during design of improvements, so that hazardous conditions, noted in the accident study, could be eliminated. In general, it can be said that those improvements that expedite traffic flow tend to eliminate traffic frictions and therefore enhance safety. Convenience of the traveling public is reflected in the system of travel time measurements that has been adopted for analysis purposes in this project. Reduction of delays, reduction in the number of stops, and increased running speed are all responsible for reducing travel time and are directly aligned with the roadway users' desires for efficient travel. The reliability and predictability of trip time are also important elements when considering convenience. These are directly reflected in the decrease in variance of flow data usually experienced as a result of improvements.

An extremely significant finding of this study was the importance of strict enforcement of traffic regulations to eliminate those violations that can nullify traffic engineering efforts. In Experiment A7, conducted at the intersection of Raymond Boulevard and McCarter Highway (the highest accident location in Newark), the level of violations was very severe. The experiment became a contest between the traffic engineers and the traveling public, with the engineers attempting to design a violation-proof system and the public finding ingenious ways to violate the regulations, creating hazardous conditions and nullifying the traffic engineering. Obviously, in such a situation enforcement is the only answer. Traffic engineering must be backed up by vigorous enforcement.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

The project statement that initiated Project 3-14, Optimizing Flow on Existing Street Networks, stated the objective of developing "a practical method of measuring the degree of change in network traffic flow resulting from various system modifications." This objective was based on recognition of the need to better understand the effect that many commonly used system modifications have on traffic flow.

The complex nature of the problem of analysis of traffic flow on an urban street network suggests the need for some generalized single measurement or coordinated system of measurements that may be used to express the interacting effects of the many impediments to traffic flow that may be influencing any given segment of roadway at a particular time. Such a measurement or system of measurements would form a common denominator by which not only the deleterious effects of traffic impediments but also the beneficial effects of modifications designed to control and counteract these detrimental influences may be expressed.

Strict adherence to the use of capacity criteria commonly used for analysis of highways and rural roads is hardly possible in the urban environment. Computations covering limited areas may be practical, but the drastic changes in conditions that occur within very short road sections preclude the use of such techniques to compute an over-all roadway capacity. Also, even at a single location, rapid changes of conditions, such as the number of turns at an intersection or the composition of the traffic stream, may produce a considerable change in capacity within a short period of time.

While this need was recognized at the inception of the project, the entire significance of developing such a system of measurements to be used, not only to express the results of road improvements but also to assess the relative value and interaction of these improvements, was not fully appreciated. How are the benefits to be derived from the restriction of parking to be compared with those obtained by the improvement of a signal progression, or perhaps those resulting from better control of pedestrian interference? Measurements that attempt in any way to relate these elements to the static physical characteristics of the road system do not have the required flexibility to express the many variables involved. Obviously, these relationships require consideration of the dynamics of the situation and must focus more on the characteristics of the traffic stream than on the physical elements of the transportation network. In other words, the importance of any modification to the road network must be measured and evaluated in the result it produces on the flow of traffic.

In the search for an appropriate system of measurement, many variables relating to the dynamics of traffic flow were tested, including average vehicle speed, travel time, acceleration noise, mean velocity gradient, and various delay time measurements. Systematic measurements of volume were also made; but, except in those situations where demand exceeded capacity, these proved to be of little use in assessing the effect that improvement of road conditions had on traffic flow. However, volume measurements were necessary for control purposes.

Results of the tests performed to evaluate several of these variables appear in Appendix A. These tests and other related experience of the project finally focused attention on a family of measurements related to travel time through which the effects of the various internal and external influences on flow could be evaluated. The measurement of applicable elements of travel time data and the application of these data to both local and network analyses appear in Chapters Three and Five.

BACKGROUND

The project statement describing the research needs and related objectives of this project stated the following:

> A need exists to improve the traffic-carrying capacity of existing urban street networks. Limited experiments have demonstrated that substantial improvements can be achieved by modifications of such factors as: changes in speed of progression, turning and parking control, one-way streets, unbalanced lane operation, platoon dispersion, intersection controls, utilization of available green time, pedestrian controls, and minor geometric or structural changes within right-of-way. The effects of these and other traffic engineering measures on the flow of traffic in the network are not clearly understood and there are no accepted methods of measuring network efficiency.
>
> Development of sophisticated systems for computer controls and electronic guidance are being studied elsewhere and would not be included.
>
> *Objectives:*
>
> The objectives of the research are to develop a practical method of measuring the degree of change in network traffic flow resulting from various system modifications and to evaluate the degree of change resulting from practicable modifications. Consideration should be given and examples studied where one or more changes are made simultaneously.
>
> Specifically, the research shall:
>
> 1. Develop the quantitative potential effects of all pertinent factors upon the total flow in network systems. For each factor, the effect of incremental changes in the factor upon the flow should be determined throughout the range of possible flows. The interdependence between factors should also be quantitatively defined.
> 2. Develop practical methods of measuring the effects of system changes.
> 3. Test experimentally the effects of the factors by adequate field observations and measurements and indicate the level of confidence for each of the major factors and their interrelationships insofar as traffic flow is concerned.
> 4. Outline a procedure for the practical application of the results of this research to street networks in general.

Objectives 1 and 3 were investigated through the experiments conducted in the downtown areas of Louisville, Ky., and Newark, N.J. Thirty-seven experiments, each of which investigated one or more of the foregoing modifications, were conducted in these study areas. Chapter Two summarizes the findings of each experiment. Appendix G describes the experiments in greater detail. Each report in Appendix G specifies the objectives of the investigation, describes the conditions under which the experiment was conducted, summarizes data obtained from field measurements, describes pertinent features of the analysis, and lists the findings of the experiment. Objectives 2 and 4 are more closely related to the network analysis and the related studies of models to be used in network analysis. These subjects are developed in Chapters Three and Five.

RESEARCH PLAN

The research plan for this project was designed to investigate and quantify the effect of traffic engineering modifications to a street system and to develop methods for measuring both the local and network effects of these modifications. The research was conducted in the real-life environment of the downtown areas of Newark and Louisville. Operational studies, conducted as one of the first elements of project work, investigated the potential for traffic engineering improvements to the street systems of these study areas. These initial investigations included study of traffic operations, mass transit operations, and accident records. The operations studies produced brief reports

describing deficiencies in the existing system and proposed remedial measures based on these limited observations.

Meetings were held with the city traffic engineers of both study areas to review these reports and to receive comments and criticism of the measures proposed. The project regulations required that the local governments having jurisdiction over the roads involved in the proposed experiments pay for construction of permanent improvements. Engineering costs were to be supported by the project. Many of the improvements that involved considerable cost (such as major channelizations or replacement of signal equipment) were rejected because of budgetary considerations. Another factor in this matter was the imminence of the Traffic Operations Program to Increase Capacity and Safety (TOPICS). Both cities were interested in this program and naturally wanted at least the more expensive improvements deferred so that they could be paid for under the sharing formula of the TOPICS program. The final program of experimentation for each city was controlled by these considerations:

1. The opportunities offered by each study area for improvements of the type that were specified to be of interest to this project.
2. Approval of the improvement by the public officials, including acceptance of the costs of permanent improvements.
3. Time and budget limitations of the project itself.

Concurrently with the traffic operations studies, inventories of street and transit facilities were compiled, and files of existing traffic engineering data were reviewed to obtain background information for the project.

Traffic counting stations were established at key points in each study area to provide a continuing record of traffic volume data for experimental control and for detection of possible changes in traffic flow due to the experiments. A continuing travel time study also was developed to yield information describing trip time, amount of delay time, number and cause of delays, average speed, and the number and severity of accelerations. These data were used during the development of the trip assignment model of the Newark downtown area and for analysis of network effects of experimentation in both study areas. Volume and travel time measurements were also made both before and after most experimentation, concentrating measurements on and near the experimental site.

Project experimentation (reported in Appendix G) includes an analysis of the local effects of the improvement, which has been generally referred to as the First Level Analysis. The Second Level Analysis, detailed in Chapters Three and Five, includes consideration of the network effect of experimental improvements on roads of the study areas and a generalized analysis to develop methods for application of this information to other areas. The applications, given in Chapter Six, develop a methodical approach to analysis of traffic flows in a congested urban environment. This chapter is intended to develop the framework for functional and operational studies of the traffic problems of a downtown area.

PROJECT DEMONSTRATIONS

The initial work plan recognized the potential for demonstrating the betterment of traffic flow inherent in this project. The results of experimentation and analysis performed on this project could demonstrate to public officials the benefits of similar programs in their cities. To rely entirely on the published material to develop this awareness would not be realistic. The technical nature of these publications, the time required to read them, and their limited distribution all indicate the need for a more acceptable medium for the demonstration aspects of the project.

To provide for this need, a 16-mm film was produced; it briefly describes several interesting portions of the project work. This film, "Relief for Tired Streets," may be purchased from Coleman Productions, Inc., 45 West 45th Street, New York, N.Y. 10036, for $80. Running time is approximately 18 min. Short-term loan copies may be obtained from the Program Director, NCHRP.

STUDY AREAS

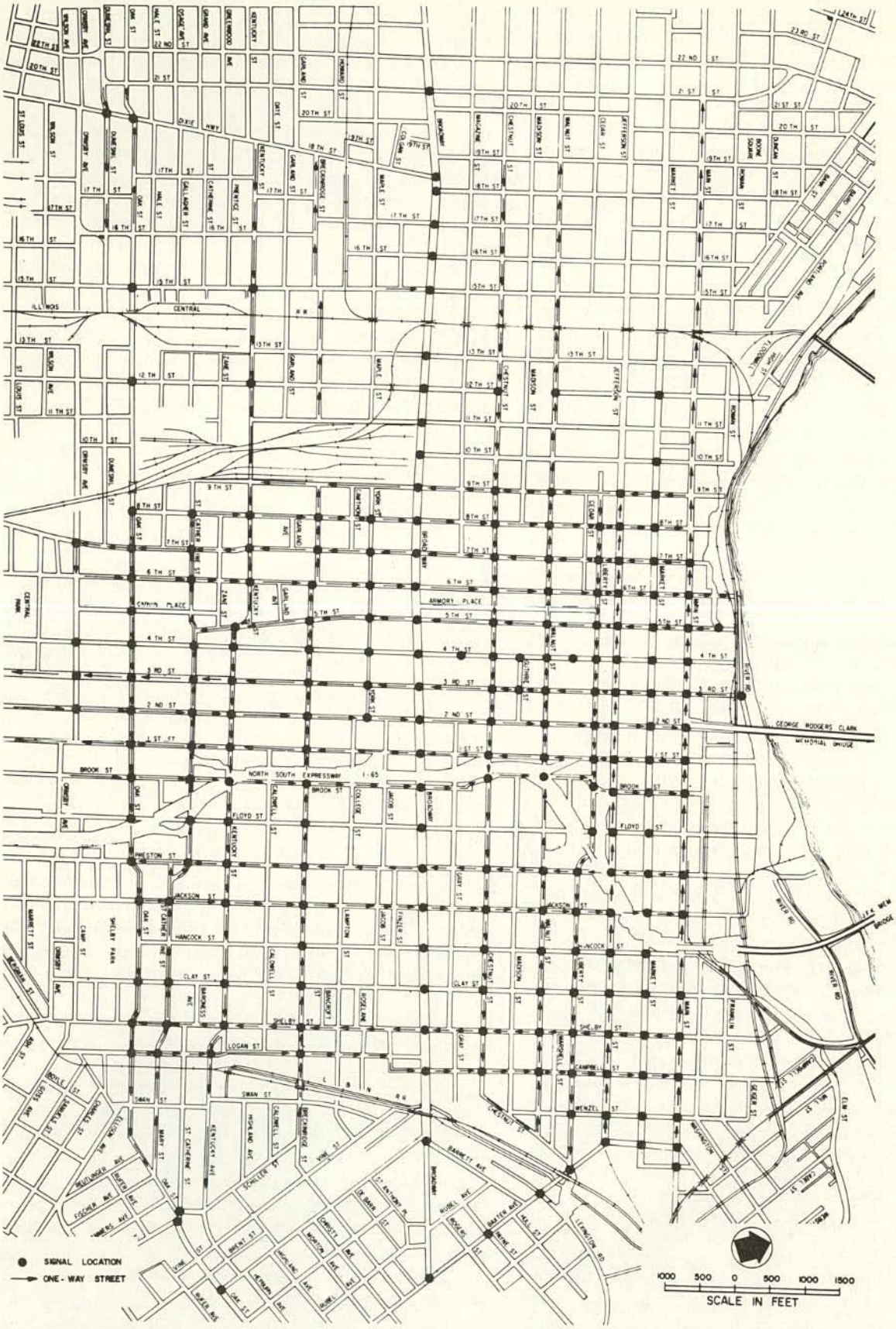
To obtain a real-life environment in which to conduct the experimentation of this project, the downtown areas of two cities were selected as study areas. These cities are Louisville, Ky., and Newark, N.J. The project statement indicated a preference for research to be conducted in cities within the population range of 250,000 to 600,000. Cities within this population range generally fall into two major classifications: (1) the community that is the metropolitan center of a large suburban and rural area furnishing the necessary business, commercial, and industrial services to this area, and (2) the community that is part of a huge metropolitan area and has the characteristic of a satellite focal point within the larger complex.

Louisville, Ky.

Louisville is the center of more than 1,000 diversified industries and 5,000 retail firms. The 1960 census indicates a population of approximately 391,000—an increase of about 22,000 from 1950. It is the market area for over a million people; a large urban renewal program is under way; and programs for civic improvement include much work on the street system, as well as highway construction that is currently being implemented. Louisville is served by an extensive transportation system. Two new Interstate highways, six commercial airlines, seven major railroads, and five commercial barge lines provide for the transportation needs of commerce and industry. Centrally located in the Ohio River Valley, it is only 150 miles from the population centroid of the United States. The administration of this city has cooperated enthusiastically with the project, providing much of the required background information, assisting in the acquisition of additional data, reviewing plans for improvement, and cooperating in the implementation of field experiments.

A comprehensive transportation study of the Louisville metropolitan area has just been completed. This study was the source of many data required for project purposes.

The street system of this study area (Fig. 1) is a fairly


SIGNAL LOCATION
ONE-WAY STREET
1000 500 0 500 1000 1500
SCALE IN FEET
NORTH SOUTH EXPRESSWAY I-65
GEORGE ROGERS CLARK MEMORIAL BRIDGE
ILLINOIS CENTRAL R.R.
ARMORY PLACE
CAMPAIGN PLACE
BROOK ST
FLOYD ST
PRESTON ST
JACKSON ST
HANCOCK ST
CLAY ST
LOGAN ST
SWAN ST
SHELBY ST
CAMPBELL ST
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LEXINGTON RD
SHELBY PARK
CENTRAL PARK
RIVER RD
ELM ST
MARKET ST
MAIN ST
FRANKLIN ST
GEORGE ST
WASHINGTON ST
CHESTNUT ST
MADISON ST
WALNUT ST
JEFFERSON ST
BROADWAY
BRECKINRIDGE ST
MAPLE ST
GARLAND ST
KENTUCKY ST
OAK ST
CATHERINE ST
HALE ST
GALLAGER ST
PRENTICE ST
DUMESNIL ST
CRESCENT AVE
WILSON AVE
ST LOUIS ST
DIXIE HWY
ZANE ST
YORK ST
COLLEGE ST
JACOB ST
BRIGHAM WAY
FLOYD ST
CALDWELL ST
LAMPTON ST
FINZER ST
GURLEY ST
HIGHLAND AVE
SCHILLER ST
FISCHER AVE
BOSTON AVE
MORTON AVE
OAK AVE
BURKE AVE
BRENT ST
ST ANTHONY PL
OLD BARN CT
BOOKER ST
BRAXTON AVE
HALL ST
LEXINGTON RD
BROADWAY
BURKE AVE
MARY ST
ST CATHERINE ST
KENTUCKY AVE
HIGHLAND AVE
CALDWELL ST
BRECKINRIDGE ST
YORK ST
CHESTNUT ST
MARSHALL ST
CAMPBELL ST
MENZEL ST
SHELBY ST
BRAMCROFT ST
ROSELANE
BARONESS AVE
CAMP ST
CRESCENT AVE
MARKET ST
MAIN ST
FRANKLIN ST
GEORGE ST
WASHINGTON ST
ELM ST
MILL ST
CAGE ST
BOYLE ST
CHARLES ST
CONRAD ST
ELLIOTT AVE
BOULDER AVE
BOATING AVE
FISCHER AVE
BOSTON AVE
OAK ST
BURKE AVE
BRENT ST
HIGHLAND AVE
MORTON AVE
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BURKE AVE
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KENTUCKY AVE
ST CATHERINE ST
MARY ST
OAK ST
SCHILLER ST
BOULDER AVE
BOATING AVE
FISCHER AVE
BOSTON AVE
MARKET ST
MERIDIAN ST
CRESCENT AVE
CAMP ST
SHELBY PARK
OAK ST
CATHERINE ST
HANCOCK ST
CLAY ST
BARONESS AVE
CALDWELL ST
BRAMCROFT ST
ROSELANE
LAMPTON ST
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YORK ST
COLLEGE ST
JACOB ST
BRIGHAM WAY
FLOYD ST
CALDWELL ST
LAMPTON

regular grid, traversed in the north-south direction by I-65. Present planning for construction of expressways in this area includes the construction of I-64 along the Ohio River waterfront in the northern extremity of the city. A section of I-64 extending northeastward from the city has been completed and is in use at present.

Five bus companies operate a total of 282 buses in the study area. The largest company is the Louisville Transit Company, with a total of 216 buses operating on 14 separate routes, making 1,803 daily round trips in the study area.

Newark, N.J.

The 1960 census reported that Newark had a population of about 405,000—a decline of 8 percent below its high point experienced in 1930. However, Newark is the center of an expanding metropolitan area. It is anticipated that the population of this area will grow from an estimated 1,000,000 in 1964 to approximately 2,300,000 in 1980. Newark is the location of approximately 33 percent of all the jobs in the Standard Metropolitan Statistical Area (SMSA), the transportation hub and financial center of the region. It has a major airport and seaport, has extensive bus service, and is served by five railroads. The street system consists of major arterials of a roughly radial pattern focusing on downtown Newark superimposed on a discontinuous rectangular grid of land service and collector roads (Fig. 2).

There are 34 bus companies operating an estimated 2,945 buses in the study area. The largest company is Public Service Coordinated Transport, with a total of 2,537 buses operating on 84 separately numbered routes, making 7,474 daily round trips in the study area.

The highest daily volume of buses in one direction (1,979) occurs northbound on Broad Street between Clinton and Commerce Streets.

The Newark Transportation Study of 1961 found that an estimated 2,000 buses leave the central business district (CBD) daily during the period from 2:00 to 6:00 PM. During the period from 3:00 to 6:00 PM, some 51,000 passengers use bus transit facilities; approximately 30,000 of these passengers are outbound.

At present, Newark is engaged in an extensive urban renewal program and has applied for a Demonstration City grant. The city administration has welcomed the opportunity to cooperate with this project, and the researchers have received much assistance from city officials.

NOMENCLATURE

A need has developed in performing the investigations of this project to define several terms that have become necessary to these studies. In general, those definitions that are in common use and that are defined in the *Highway Capacity Manual* (*1*, pp. 4-21) have not been altered. However, it has been necessary in several instances to add terms that extend and refine the various aspects of a category of definitions. This is particularly the case with traffic operations definitions that involve description of speed, running time, and delay. These definitions have become very important, as this study includes a system for describing a level of service for downtown streets based on analysis of travel time.

The importance of describing traffic flows at an intersection in terms more readily related to the over-all concept of travel time analysis has also become apparent. Therefore, several terms are defined that fill this need. Study of downtown traffic operations is largely a study of intersections. Most of the critical restrictions to flow occur at intersections, and most of the relief measures must deal with intersection problems. Because of this, there is a need for measurements that can be used to describe these critical elements of traffic movement and that can also be easily surveyed and analyzed.

These traffic operations definitions are brought together into three categories: route measurements, intersection measurements, and general. Where it was thought to be beneficial to complete the measurement system, definitions given in Chapter Two of the *Highway Capacity Manual* are repeated. The slight alteration of using the term "roadway" in place of "highway" has made the definitions applicable to downtown street systems.

Route Measurements

TRAVEL TIME (T). The total time required for travel over a specified section of roadway for a single trip or the average of several trips, including all delay time.

RUNNING TIME (T_R). The time the vehicle is in motion.

DELAY TIME (T_D). The time the vehicle is not in motion ($T = T_R + T_D$).

OVER-ALL TRAVEL SPEED (S). The total distance traversed divided by the total time required, including all traffic delays.

RUNNING SPEED (S_R). The total distance traversed divided by the running time. If D is the distance traversed, $S = D/T$ and $S_R = D/T_R$.

INTERSECTION DELAY (T_I). Delay time occurring due to interference to flow at an intersection.

MIDBLOCK DELAY (T_M). Delay time occurring due to interferences to flow between intersections ($T_D = T_I + T_M$).

SIGNAL DELAY (T_S). Delay time occurring due to interference to flow by traffic signals at an intersection.

TURN DELAY (T_T). Delay time occurring due to interference to flow by turning vehicles at an intersection.

PEDESTRIAN DELAY (T_P). Delay time occurring due to interference to flow by pedestrians at an intersection.

Other interferences to flow may be similarly defined so that $T_I = T_S + T_T + T_P + \ldots$

Midblock delay may also be divided into its various components related to each cause of delay.

DELAY RATIO. Delay time divided by travel time.

Intersection Measurements

DELAY TIME ON RED. The total time lost by all vehicles during a predetermined time interval from the moment each arrives at the intersection or queue during the red signal interval until the signal indication changes to green. For each vehicle held for more than one cycle,

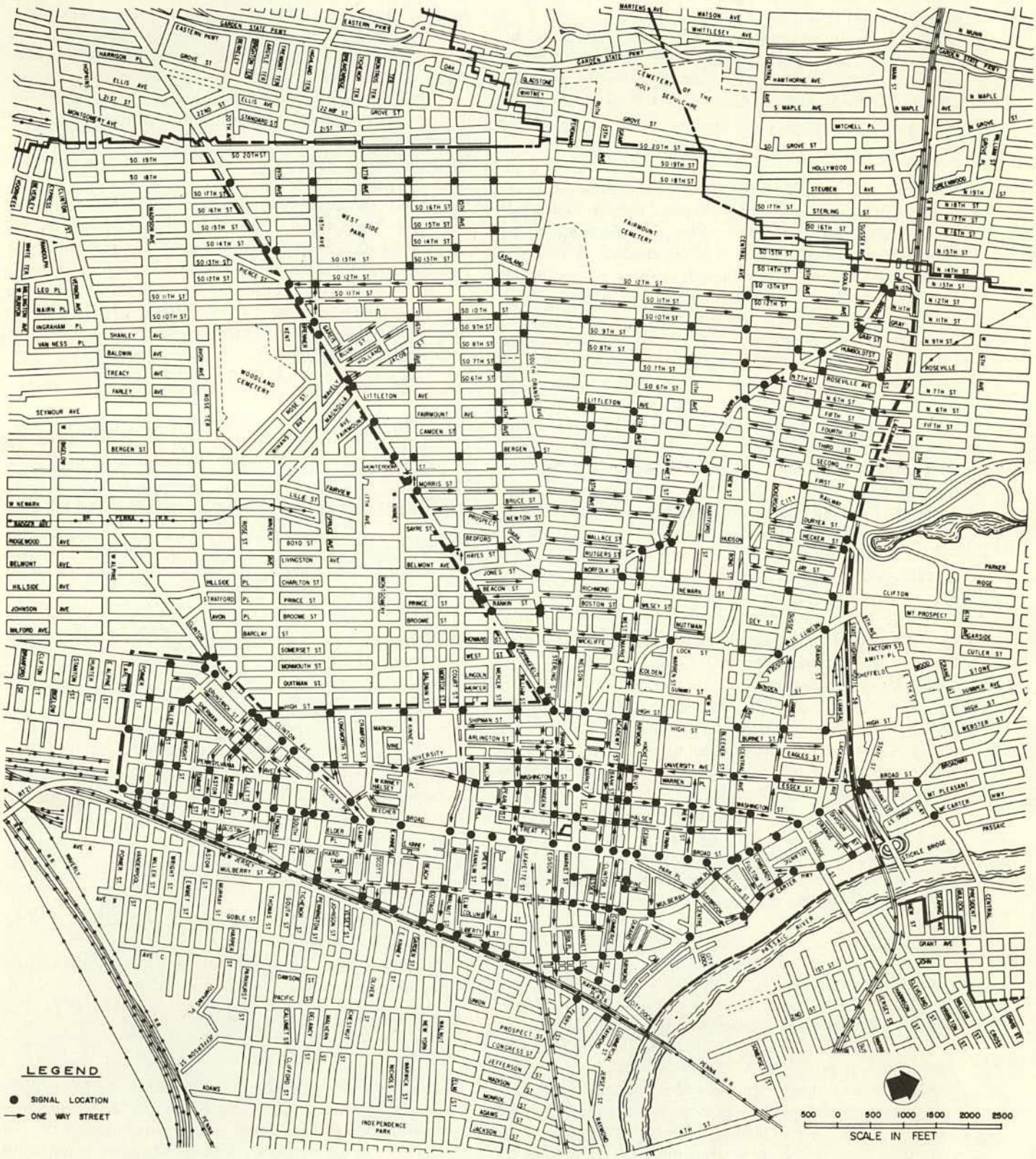


Figure 2. Newark study area.

the green and amber times for the appropriate number of cycles are added.

DELAY TIME PER VEHICLE STOPPED. A ratio quantity of delay time divided by the number of vehicles stopped during a predetermined time interval.

DELAY TIME PER VEHICLE THROUGH. A ratio quantity of delay time divided by the number of vehicles through during a predetermined time interval.

VEHICLES QUEUED. The total number of vehicles in all lanes of an approach stored at the end of the red interval.

VEHICLES STOPPED ON RED. The number of vehicles stopped in all lanes of an approach at the end of the red interval.

The term "vehicles stopped" means the same as the term "vehicles queued" when data are reported by cycle. When longer time periods are considered, "vehicles stopped" is defined as the number of vehicles stopped per cycle minus the number of vehicles stopped more than one cycle.

VEHICLES HELD OVER ONE CYCLE. The number of vehicles not clearing the intersection during the first green interval after arrival.

VEHICLES THROUGH. The number of vehicles passing through a signalized intersection from a given approach during the green interval.

SATURATED CYCLE. A signal cycle for which, at a given approach to a signalized intersection, the number of vehicles stopped at the end of the red interval is greater than the number of vehicles through on the following green interval.

Turning lanes with significant traffic volumes may be considered as separate approaches.

General

Although definitions are given for "rate of flow," "interrupted flow," and "uninterrupted flow," no definition is given for "flow" in Chapter Two of the *Highway Capacity Manual.* This term has been widely used but is subject to a variety of interpretations, varying from definitions almost synonymous with "volume" to definitions that imply quality as well as quantity. Such an implication is indicated by the words "interrupted" and "uninterrupted." When traffic movement on a freeway is considered, there may be little difference of meaning between the words "volume" and "flow." However, when the study of downtown streets is involved, the need for a "quality" as well as a "quantity" component in defining flow becomes very apparent. Under uninterrupted flow conditions a certain quality of travel is implied by the quantity measurement. This relationship is much less apparent under the extreme conditions of interrupted flow experienced on downtown arterials. Therefore, it becomes necessary under these conditions to retain specific measurements of the quality as well as the quantity components of flow. In consideration of this need, the following definition for flow is used:

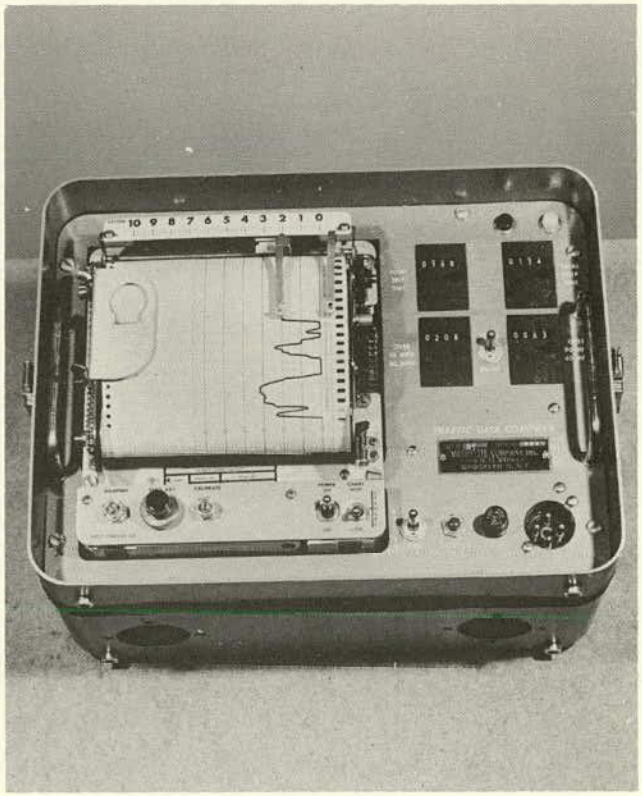
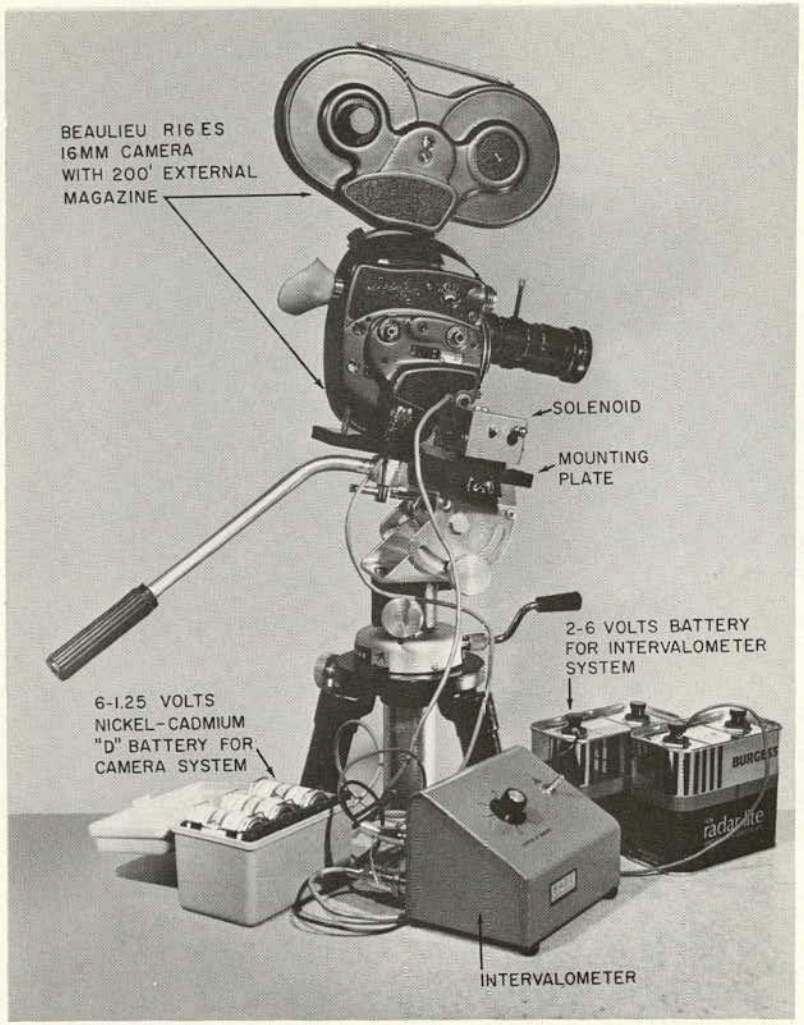


Figure 3. Traffic data compiler.



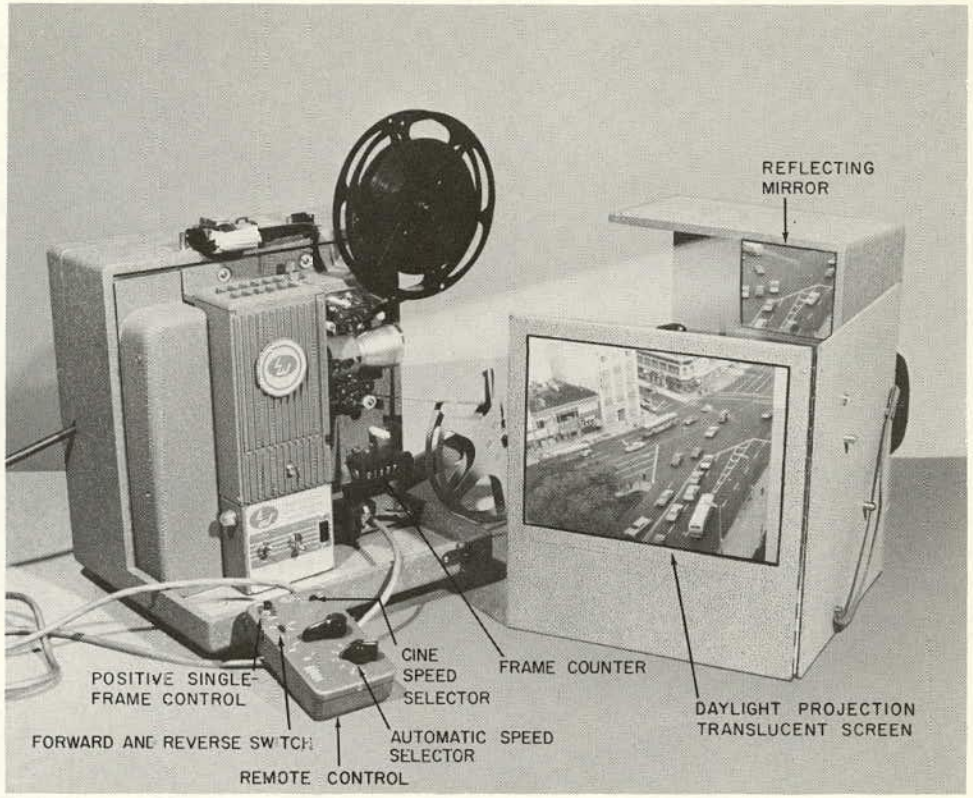


Figure 4. Time-lapse photographic equipment and time-lapse data analysis equipment.

TRAFFIC FLOW (under interrupted conditions). Traffic flow is measured as a volume of traffic moving at an average over-all travel speed during a specified time period.

For the special purpose of this research, two terms are used to describe the phases of analysis:

FIRST LEVEL ANALYSIS. The direct evaluation of the experimentation conducted in the study areas, sometimes referred to as the "local" analysis.

SECOND LEVEL ANALYSIS. The investigation of methods to be used in the application of the results of this research to other areas. This includes the testing and validation of models to be used for network analysis, development of measurements and measurement systems to be used for measuring network efficiency, and methods for application of the results of this research.

To establish the time position of conditions affecting experimentation with regard to the implementation of improvements, the following terms are commonly used:

"BEFORE." Applied to conditions or measurements before implementation of the improvement.

"AFTER." Applied to conditions or measurements after implementation of the improvement.

In the study of bus transit, the following terms are used:

BUS STOP OPERATIONS. All actions or events occurring between the beginning of deceleration into the bus stop and the end of acceleration out of the bus stop, excluding the providing of passenger service.

PASSENGER SERVICE OPERATIONS. All actions or events occurring while passengers are permitted to alight or board the bus.

BUS ROUTE OPERATIONS. All actions or events occurring between bus stop operations at succeeding stops.

SURVEILLANCE EQUIPMENT

Equipment used for obtaining traffic flow measurements is described in detail in Appendix E. Vehicles equipped with Marbelite Traffic Data Compilers (Fig. 3) were used in most travel time surveys. As necessary, these units were supplemented by additional vehicles using an observer who made manual recordings with a stopwatch.

In local areas, time-lapse photography was often used to measure traffic activity. A Beaulieu, R 16 ES, 16-mm movie camera was equipped with an intervalometer specially designed for this project (Fig. 4). Photography was often used to supplement surveys at intersections or bus stops. This method was valuable in those situations where interaction of many elements had to be studied. Also, it was very useful in demonstrating various aspects of the traffic problems to public officials.

Traffic volume counts made on a continuing basis for control purposes and as part of the surveillance at and adjacent to experiment sites were made using Streeter-Amet, Model RCT, Trafficounters.

CHAPTER TWO

FINDINGS—FIRST AND SECOND LEVEL ANALYSES

To develop an understanding of the effect of traffic-engineering modifications to the road network on the flow of traffic, it was necessary to proceed from evaluation of local situations to study of network effects. The basic information describing the effect of these modifications was obtained by conducting 37 experiments in the project's two study areas. The measurements of change in traffic flows were evaluated to determine whether the changes measured were statistically significant, and the computed results for each experiment were tabulated and displayed. A separate report was prepared for each experiment; these reports appear in Appendix G. The findings of these reports are summarized in the first section of this chapter entitled "First Level Analysis." Also included in this chapter are sections summarizing the investigations that have been conducted to determine those measurements that are most meaningful in describing traffic flows on downtown arterials, the factors that influence these measurements, and the relationships between these measurements. A system of analysis, based on the foregoing measurements, has been developed. Finally, a summary description of the investigations for use and development of models to assist in functional and operational analysis is presented.

FIRST LEVEL ANALYSIS

The research problem statement for this project listed many factors that influence traffic flows. These factors were given as examples of the traffic-engineering measures to be investigated in this project. Where practical, experiments designed to investigate these and other similar factors were organized and the resulting measurements were evaluated to determine the effect of the modifications. These results are summarized in the following.

Directional Control and Lane Use

In this section of the experimental program, the use of one-way street patterns, reversible lane operations, and various lane markings and controls were investigated.

One-Way Streets

The desired effect of establishing one-way flow is to eliminate conflicts between left-turn and through vehicles at intersections and also to eliminate the conflicting requirements of opposing traffic, making possible more efficient signal progressions. One-way operation is most applicable when adjacent roadways can be paired to serve both directions of travel. One-way operation is most desirable under situations where large turning movements conflict with two-way operation, where signals are closely spaced, making signal progressions inefficient, and where directional distribution of flow is fairly balanced during all time periods of the day. Therefore, development of a one-way street system is particularly applicable to the needs of a downtown area, with its large volumes of circulating traffic and closely spaced intersections.

The radial pattern of arterial streets in Newark limited the potential for one-way revisions to the minor arterials and collector streets that form a grid pattern in the downtown area. Most of these streets were already designated as one-way at the inception of this project. In Louisville, all of the arterial streets within the downtown area were already one-way, with the exceptions of Fourth Street, Market Street, and Broadway. Two-way movement had been retained on these streets because of the need for circulation through the shopping area.

Experiment E30 instituted a one-way operation on Mellwood and Story Avenues in Louisville. These arterials are east of the downtown area. The "before" and "after" conditions for this experiment are shown in Figure 5. A statistical analysis of "before" and "after" travel times indicated a significant increase ($\alpha = 0.05$) of travel speeds for each time period on both Mellwood and Story Avenues. A summary of these measurements is given in Table 1.

Experiment E31, at River Road and Fort Nelson Way, developed a one-way rotary pattern of travel around a one-block area between Fifth and Sixth Streets. This travel pattern was instituted to eliminate several conflicting traffic movements and thereby reduce delays. The prior travel patterns and those proposed in this experiment are shown in Figure 6. Travel time studies were made for eastbound trips beginning at Main and Sixth Streets, ending at River Road and Third Street, and for westbound trips beginning at River Road and Third Street, ending at Main and Seventh Streets. Comparison of "before" and "after" measurements indicated substantial saving in travel time, number of stops, and delay time. Average vehicle speed was increased by 25 percent from 13.8 to 17.2 mph. Stops were reduced by 55 percent; and a 65-percent reduction in delay time, from 94 to 33 sec per vehicle, was experienced.

Experiment A21, Revision of Flow Directions in the Vicinity of Broad Street, Lincoln Park, and Pennington Street in Newark, involved the change of flow directions from two-way to one-way eastbound on Pennington Street, as shown in Figure 7. Traffic signals at the intersection were placed under pedestrian actuation to provide a crossing between office buildings on the east side and a bus stop on the west side of Broad Street. During the morning peak hour when the traffic situation is most critical at this location, approximately 18 signal cycles were not actuated by the pedestrians, resulting in a reduction of approximately 195 vehicles stopped at the Pennington Street intersection.

These experiments have illustrated three separate applications for the use of one-way streets. First, improvement to traffic flow in a major travel corridor was accomplished by pairing two streets in a one-way system. Second, at Fort Nelson Way and River Road a one-way pattern was used to eliminate conflicting traffic movements at intersections;

TABLE 1

EXPERIMENT E30, SPEED AND DELAY RUN DATA

		"BEFORE"						"AFTER"					
		MELLWOOD AVE.			STORY AVE.			MELLWOOD AVE.			STORY AVE.		
DIRECTION	TIME PERIOD	NO. OF OBS.	MEAN SPEED (MPH)	VARIANCE (MPH)	NO. OF OBS.	MEAN SPEED (MPH)	VARIANCE (MPH)	NO. OF OBS.	MEAN SPEED (MPH)	VARIANCE (MPH)	NO. OF OBS.	MEAN SPEED (MPH)	VARIANCE (MPH)
EB	AM	12	16.6	7.7	10	20.9	10.5	26	20.2	9.1	—	—	—
	PM	4	17.3	20.8	3	18.1	7.7	46	20.6	9.6	—	—	—
	All	16	16.9	10.5	13	19.5	10.0	72	20.4	9.4	—	—	—
WB	AM	10	17.9	4.3	12	18.2	19.7	—	—	—	27	27.3	13.1
	PM	3	17.7	0.6	4	21.2	21.0	—	—	—	46	26.9	6.8
	All	13	17.8	3.6	16	19.7	19.9	—	—	—	73	27.1	9.1
All	AM	22	17.2	6.2	22	19.6	15.5	53[a]	2.38[a]	11.1[a]	—	—	—
	PM	7	17.5	12.7	7	19.6	15.7	92[a]	23.8[a]	8.2[a]	—	—	—
	All	29	17.4	7.5	29	19.6	15.6	145[a]	23.8[a]	9.3[a]	—	—	—

[a] Mellwood Avenue and Story Avenue combined.

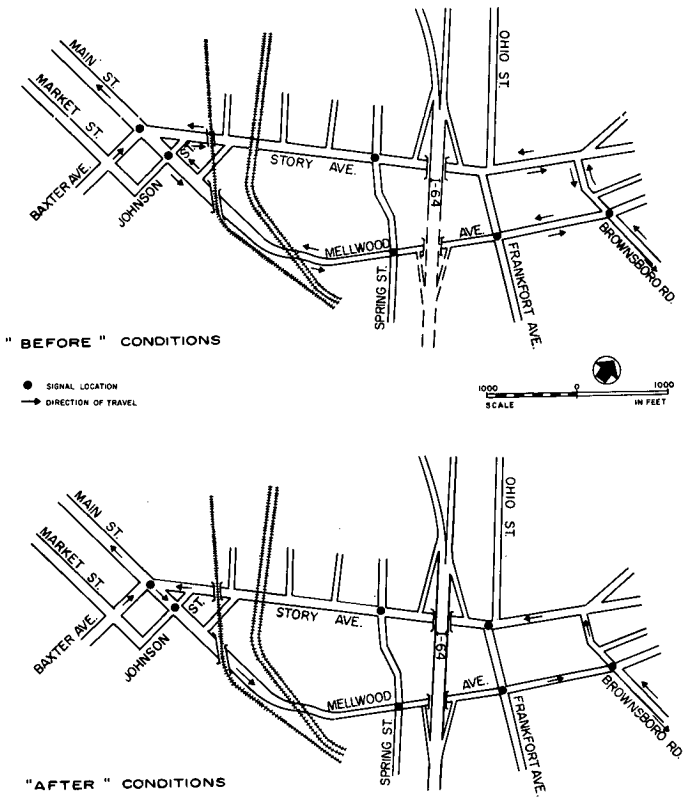


Figure 5. Experiment E30, vicinity map.

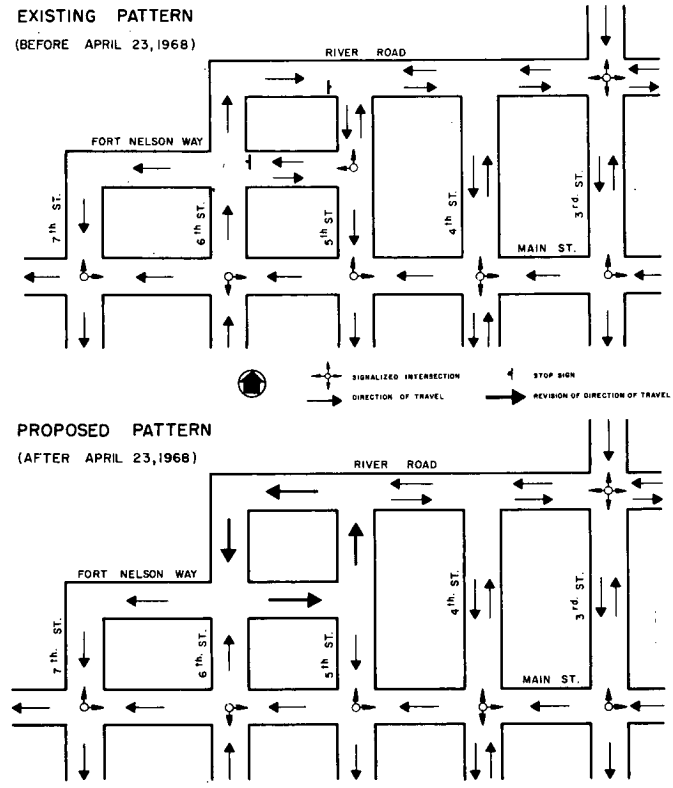


Figure 6. Experiment E31, vicinity map.

and, third, at Pennington Street in Newark a minor street was diverted to eliminate interference with major flows on a downtown arterial. In all three experiments significant improvements were measured.

Reversible Lane Operation

The primary purpose of establishing reversible lanes is to meet the short-term, highly directional traffic demands experienced in many urban areas during periods of peak traffic flow. The demands of heavy inbound morning commuter traffic to center-city areas and the evening exodus in the reverse direction may be expedited by signal progressions that favor the peak flows and by banning parking and truck loading to free more lanes for moving traffic. When such measures are inadequate to meet the demands, reversible lane operations on one or more arterial roadways may provide the required additional capacity. Preferably, streets with reversible lanes should be at least 50 ft wide, providing for five 10-ft lanes. Considering bus stops, breakdowns, and other stopping vehicles (which may be in violation of city ordinances but which stop nevertheless), a minimum of two 10-ft lanes is required for the off-peak direction.

The grid pattern of predominantly one-way streets in Louisville did not provide any potential for experiments with reversible lanes.

Experiment B78, Broad Street Reversible Lanes, in-

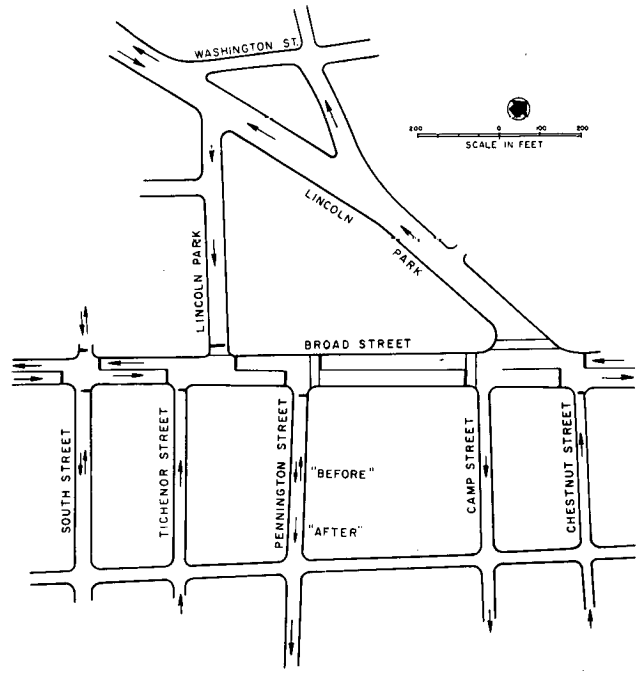


Figure 7. Experiment A21, vicinity map.

Figure 8. Experiment 78, Broad Street reversible lanes.

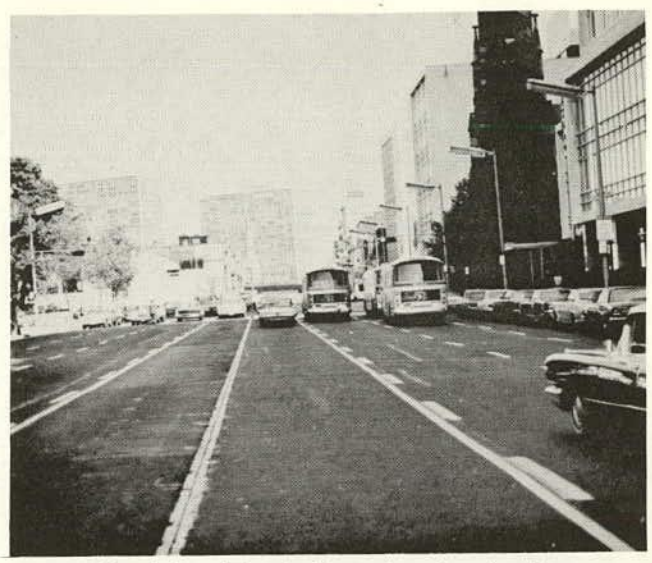
vestigated the effects of reversing the two center lanes of a 0.6-mile section of this Newark arterial. Broad Street varies from 90 ft in width, from Central Avenue to Washington Street, divided for eight lanes of moving traffic during peak hours, to 62.5 ft in width used for six lanes between Washington and Clay Streets. With parking permitted during nonpeak periods, these sections of roadway were reduced to six and four lanes, respectively. Approximately 37,000 vehicles use this facility daily in the area where this experiment was conducted. Peak-hour, peak-direction volumes average approximately 1,070 vehicles southbound in the morning and 2,060 vehicles northbound in the afternoon. Figure 8 shows Broad Street, looking south from its intersection with Washington Street. The two center lanes were used outbound in the evening, inbound in the morning, and divided for one lane in each direction during the off-peak period.

Tables 2 and 3, summary of analysis for northbound and for southbound directions, respectively, give data developed from field measurements for comparison of "before" and "after" conditions. Computations show a definite improvement for the southbound AM peak period. Signal delays encountered in the northbound direction prevented a similar improvement in the PM peak period. These delays were substantially eliminated during Experiment B100. This illustrates the controlling influence of the signal system in realizing benefits from an improvement.

Lane Controls

Lane markings and other lane controls are used primarily to organize traffic flow. Both turn lanes (to accommodate large left-turn movements) and lane markings (which offer guidance to conflicting traffic streams, delineate areas of

Looking South From Park Place



Looking North From Lombardy Place

Looking South at Lackawanna Avenue

Looking North From Grant Street

usable pavement, and restrict flow from areas allocated to other uses) reduce the internal frictions of the traffic stream. The roadway width, parking conditions, number of large vehicles, turns, bus stop locations, and traffic volumes must all be considered in the application of lane control markings. Lane markings are often a necessary supplement to operational controls. The interrelation between lane markings and other controls, such as reversible lane operations, parking controls, channelization, turn controls, and signal progressions, must be carefully considered. While minor revisions of lane markings were incorporated in many other experiments, four experiments were completed that involved major lane marking revisions.

Experiment B80, a fifth, partially completed, experiment, was organized to investigate the effect of marking a 2½-mile section of McCarter Highway in Newark with five lanes, providing a center lane for opposing left turns at intersections as well as two lanes in each direction. Experiment B80 was delayed due to uncertainty concerning the jurisdiction over McCarter Highway. A discussion between city and state officials was finally settled, with the state accepting the financial responsibility for the improvements. However, this decision was made too late for the work of the experiment to be completed within the period of this project. Observation of the final results of this improvement indicate a substantial organization of flow at critical intersections along the highway. At these locations, which previously were bottlenecks, the improvement was very apparent. McCarter Highway provides for a very heavy truck movement bypassing the downtown area of Newark. Prior to the implementation of this experiment, McCarter Highway was striped with a center line only. The 25 ft of

TABLE 2

EXPERIMENT B78, SUMMARY OF ANALYSIS, BROAD STREET NORTHBOUND

CONDITION	SPEED AND DELAY			VEHICLES PER CYCLE					
	TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	AT ORANGE ST.		AT CENTRAL AVE.		AT PARK PLACE	
				THROUGH	STOP	THROUGH	STOP	THROUGH	STOP
(*a*) AM time period									
"Before"	137.3	46.5	1.6	26.0	12.0	16.1	3.7	6.6	4.7
"After"	184.0[a]	70.3	2.3	25.7	12.2	15.9	4.5	5.6	4.3
Net change	+46.7	+23.8	+0.7	−0.3	+0.2	−0.2	+0.8	−1.0	−0.4
Percent change	+34.0	+51.2	+43.8	−1.1	+1.7	−1.2	+21.6	−15.2	−8.5
Sig. level	0.0005	0.005	0.005	NS	NS	NS	NS	0.05	NS
(*b*) Midday time period									
"Before"	76 8	29.7	1.0						
"After"	88.2	32.5	1.3						
Net change	+11.4	+2.8	+0.3						
Percent change	+14.8	+9.4	+30.0						
(c) PM time period									
"Before"	120.4	50.3	1.8	68.2	29.2	29.2	12.0	19.5	—
"After" Wk. 1	118.5	53.4	1.9						
Wk. 2	113.2	51.1	1.8	68.8	34.6	31.1	7.7	21.0	—
Wk. 3	110.1	53.1	2.0						
Net change (Wk. 3)	−10.3	+2.8	+0.2	+0.6	+5.4	+1.9	−4.3	+1.5	—
Percent change	−8.6	+5.6	+11.1	+0.9	+18.5	+6.5	−35.8	+7.7	—
Sig. level	NS	NS	NS	NS	0.05	NS	0.05	NS	—

[a] Variability increased ($\alpha = 0.05$).
NS = not significant ($\alpha = 0.10$).

TABLE 3

EXPERIMENT B78, SUMMARY OF ANALYSIS, BROAD STREET SOUTHBOUND

CONDITION	SPEED AND DELAY			VEHICLES PER CYCLE			
	TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	AT ORANGE ST.		AT CENTRAL AVE.	
				THROUGH	STOP	THROUGH	STOP
(*a*) AM time period							
"Before"	267.8	127.1	4.1	44.0	33.5	44.2	10.2
"After"	158.6[a]	53.9[a]	1.8[a]	48.6	25.3	44.4	12.0
Net change	−109.2	−73.2	−2.3	+4.6	−8.2	+0.2	+1.8
Percent change	−40.8	−57.6	−56.1	+10.5	−24.5	+0.5	+17.6
Sig. level	0.0005	0.0005	0.0005	0.05	0.05	NS	NS
(*b*) Midday time period							
"Before"	67.5	21.3	0.9				
"After"	79.6	29.3	1.1				
Net change	+12.1	+8.0	+0.2				
Percent change	+17.9	+37.6	+22.2				
(*c*) PM time period							
"Before"	81.9	27.5	0.9	18.7	11.9	28.1	7.8
"After" Wk. 1	106.9	46.3	1.4				
Wk. 2	95.7	36.9	1.2	18.9	15.3	30.7	10.5
Wk. 3	105.3	45.2	1.5				
Net change (Wk. 3)	+23.4	+17.7	+0.6	+0.2	+3.4	+2.6	+2.7
Percent change	+28.6	+64.4	+66.7	+1.1	+28.6	+9.3	+34.6
Sig. level	NS	NS	0.05	NS	0.05	NS	0.05

[a] Variability reduced ($\alpha = 0.05$).
NS = not significant at $\alpha \leq 0.10$.

pavement allocated to each direction were used in a disorganized manner, generally by a truck movement occupying the curb lane and passenger cars and turning traffic sharing the adjacent lane. One of the questions of interest was whether the trucks would accept a narrower curb lane for their use. Observations of truck movement on McCarter Highway indicated that debris in the gutters was forcing trucks to use more of the pavement than would normally be required. Therefore, as part of this experiment, highway crews removed the debris and cleared the pavement for use. It was noted that, following implementation of the lane striping program, the trucks did move over and substantially used the curb lane without interference to movement in the second lane, which was used largely by passenger vehicles. It should be noted that there was little, if any, interference from parked vehicles to the use of the curb lane. In the location of McCarter Highway, there is little demand for parking, and parking has been restricted continuously for such a long period that violations are very infrequent.

Experiment B86, Revision of Lane Markings on Central Avenue, produced several interesting effects. During the midday period, when parking was permitted, there was a significant increase in speed in each direction. It is probable that the distinct delineation of three lanes in each direction in the six-lane section resulted in better organization of the parking, with a significant reduction of double parking and acceptance of two-lane operation in the remaining pavement, rather than a disorganized single-lane operation. Also, in the six-lane section a significant shift during afternoon peak periods was noted from use of the westbound curb lane to use of the adjacent lane. In the "before" condition, between 4:30 and 5:30 PM, an average of 420 vehicles used the curb lane and 654 used the adjacent lane, with vehicles being assigned to the lane occupied by the major portion of the vehicle, approximating the lane design of the "after" condition. During the "after" measurements, it was found that use of the curb lane had reduced to an average of 349 vehicles, whereas use of the adjacent lane had increased to 802 vehicles. The volume in the centermost lane remained nearly constant at 626 vehicles in the "before" condition and 644 in the "after" condition. This shift of use to the adjacent lane indicates a decided driver preference for avoiding the interference of bus stops, parking violations, and right-turning vehicles. It should also be noted that approximately 2 ft of pavement adjacent to the curbing was seldom used due to a large amount of debris in the gutters and a dip of approximately 3 in. at each catch basin, which was obviously avoided by drivers. No significant changes in volume were measured as a result of this experiment, and peak-hour speeds were not significantly altered.

Experiment A47, Lane Marking Revision at Central Avenue and West Market Street, was designed to increase the capacity at this critical intersection by offsetting the center line to provide an additional approach lane (from two to three lanes) on Central Avenue westbound at West Market Street. This is the transition section that joins the 50-ft and 60-ft sections of pavement (Fig. 9). The critical traffic condition at this intersection occurs during the afternoon peak period when a volume of 1,040 vehicles was measured for Central Avenue westbound and 1,200 vehicles was measured for West Market Street northbound.

Experiment A47 was the first major intersection experiment that involved use of delay time measurements; these are discussed in more detail in Chapter Three. This involves a detailed measurement on a cycle-by-cycle basis of vehicles stopped on red, vehicles through on green, queue lengths, and queues remaining at the end of the green interval. As a result of the changes instituted in this experiment, a substantial reduction of saturated cycles was measured. During the afternoon peak hour, saturated cycles were reduced from 97 percent to 4 percent on Central Avenue westbound and from 52 percent to 17 percent on West Market Street northbound. Figures 10 and 11 show, on a cycle-by-cycle basis, the queues that developed at each approach, displaying the difference between "before" and "after" conditions. The length of queue at the beginning of the green interval is indicated by the top of the shaded portion of each bar. The queue remaining at the end of the green interval is indicated by the black portion of each bar.

An item of extreme significance to successful implementation of a lane-marking program became apparent during this experiment. Two lanes were retained for eastbound movement on Central Avenue east of the West Market Street intersection. However, the pavement width allocated to the two lanes was narrowed by 5 ft, from one-half of the 50-ft pavement to two 10-ft lanes. Parking violations dur-

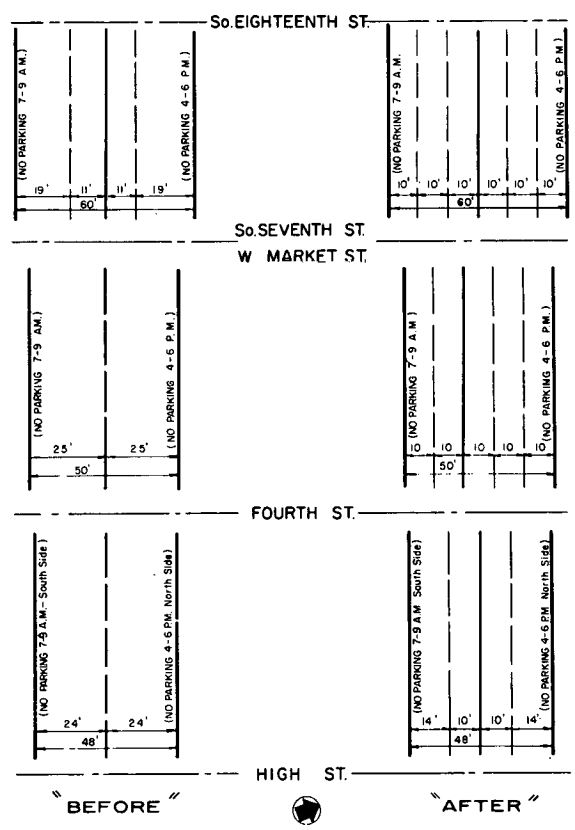


Figure 9. Experiments A47 and B86, lane markings.

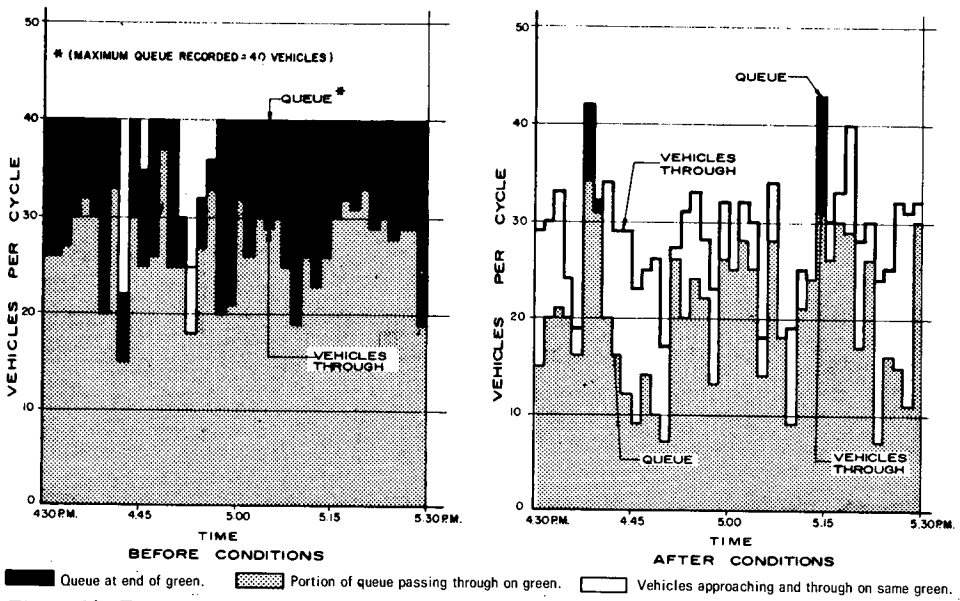


Figure 10. Experiment A47, Central Avenue westbound vehicles queued and through.

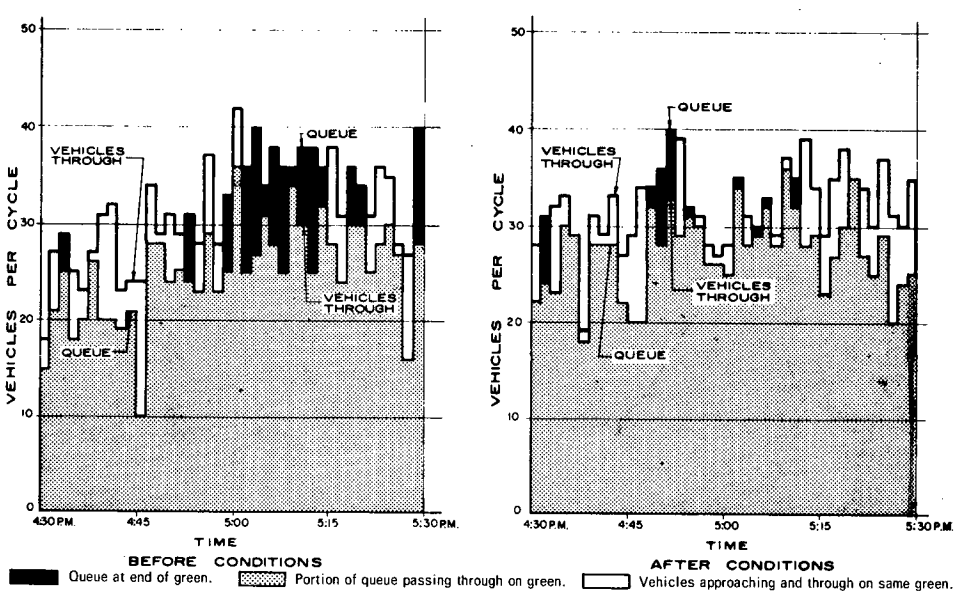


Figure 11. Experiment A47, West Market Street northbound vehicles queued and through.

ing the morning peak period in the block immediately east of the intersection narrowed the usable pavement to one 10-ft lane, creating serious congestion extending backward through the intersection. These violations were most harmful when they interfered with proper use of the bus stop located in this block. Under these conditions the bus was forced to stop in the remaining lane, totally blocking passage of vehicles in the narrowed section of pavement. In this case, as in several other experiments, it was noted that the optimum lane delineation, implemented for capacity consideration, was often more vulnerable to parking violations than the previous design, making strict enforcement a necessary component for successfully attaining the optimum condition. During infrequent periods when police enforcement maintained both lanes in operation, no significant detriment was measured for operation in the eastbound direction due to decreased width of this roadway.

Experiment D67 investigated the benefits obtained by adding lane control markings and curb-mounted signs to organize traffic at an exceptionally heavy left turn. Fig-

ure 12 shows conditions at the intersection of Main Street and Third Street in Louisville. Four sets of lane control arrows were installed on the Main Street approach; a sign, THIS LANE MUST TURN LEFT, was installed at the south curb. Measurements of vehicle volumes proceeding straight through the intersection or making the left turn were measured by cycle for the three lanes on the south side of Main Street. These lanes are designated Lanes A, B, and C (Fig. 12). It should be noted that no basic condition of lane use was changed in this experiment. The addition of arrows painted on the pavement and the curbside sign only supplemented the overhead signing located approximately 200 ft before the intersection. Analysis of the measurements indicated that the supplementary pavement arrows and a lane control sign did not produce a change in the proportion of vehicles turning left from each of the two left-turn lanes. However, a significant shift from the left-turn lanes to Lane C was noted for vehicles traveling straight through the intersection. Violation of the mandatory left-turn use of Lane A was significantly reduced from 0.8 percent to 0.4 percent of the straight-through movement. Only 14 percent of the left-turning vehicles used Lane B. Left-turning drivers probably avoided use of this lane because they might have been stopped during a red interval behind a straight-through vehicle. A sign, LEFT TURN ON RED AFTER STOP, permitted the left-turning vehicles in the curb lane to turn during the red interval.

Experiment A7, Raymond Boulevard and McCarter Highway Left-Turn and Pedestrian Control, included an initial phase that involved the lane marking of three blocks of Raymond Boulevard and the McCarter Highway approaches at the intersection of these two roadways. McCarter Highway (N.J. 21) is a major north-south arterial that forms a bypass of the Newark downtown area. Raymond Boulevard is a major east-west arterial conducting traffic to the heart of the downtown area. The Raymond Boulevard-McCarter Highway intersection is one of the most congested locations in the Newark study area, and has a very severe accident record. The intersection of these roadways is shown in Figure 13. Pavement markings developed for this experiment provided for opposing left-turn lanes for both Raymond Boulevard and McCarter Highway at their intersection and for Raymond Boulevard at its intersection with Mulberry Street.

Some of the experience gained through this experiment was totally unexpected, but very significant. Violations of traffic ordinances were so common that it was the exception, rather than the rule, to be able to make a survey for a significant period of time free of the influence of such violations. When enforcement was attempted to eliminate parking violations, the public reaction was vigorous. One of the city's employees remarked that the protest was the most drastic to occur during his years with the city. The validity of city ordinances permitting the city to post temporary parking restrictions was questioned in Council meeting and referred to the New Jersey Division of Motor Vehicles for a decision. Some of the haste in making the "after" measurements for this experiment can be explained by the need for police enforcement, which was being jeopardized by the public reaction. In addition to parking violations, misuse of left-turn lanes by drivers proceeding straight through the intersection, signal violations by both vehicles and pedestrians, and the extremely aggressive attitude exhibited by most of the drivers presented anything but the controlled laboratory environment that was desired. However, a significant betterment of traffic conditions was noted, as reflected by the data given in Table 4 and typically shown in Figure 14. Later phases of this experiment investigated the use of separate turn intervals and pedestrian controls at this intersection.

TABLE 4

SUMMARY OF MEASURED CHANGES,[a] RAYMOND BOULEVARD AND McCARTER HIGHWAY LANE MARKING REVISIONS

STREET AND DIRECTION	TIME PERIOD	CONDITION (DATE)	VOLUME (VPH)	VEHICLES STOPPED (%)	CYCLES SATURATED (%)
Raymond EB	AM	"Before" (Sept.)	1,044	56.7	0
	AM	"After" (Oct.)	1,172	61.8	4
	AM	Percent change	+12	+9	+
Raymond WB	PM	"Before" (Sept.)	1,325	78.8	18
	PM	"After" (Oct.)	1,290	53.2	2
	PM	Percent change	−3	−33	−89
McCarter SB	PM	"Before" (Oct.)	1,285	153.0	70
	PM	"After" (Oct.)	1,406	93.2	23
	PM	Percent change	+9	−39	−67
McCarter SB—left turn	PM	"Before" (Oct.)	287	—	—
	PM	"After" (Oct.)	424	—	—
	PM	Percent change	+48	—	—

[a] There were no measured differences during any time periods or at any approach directions other than those listed.

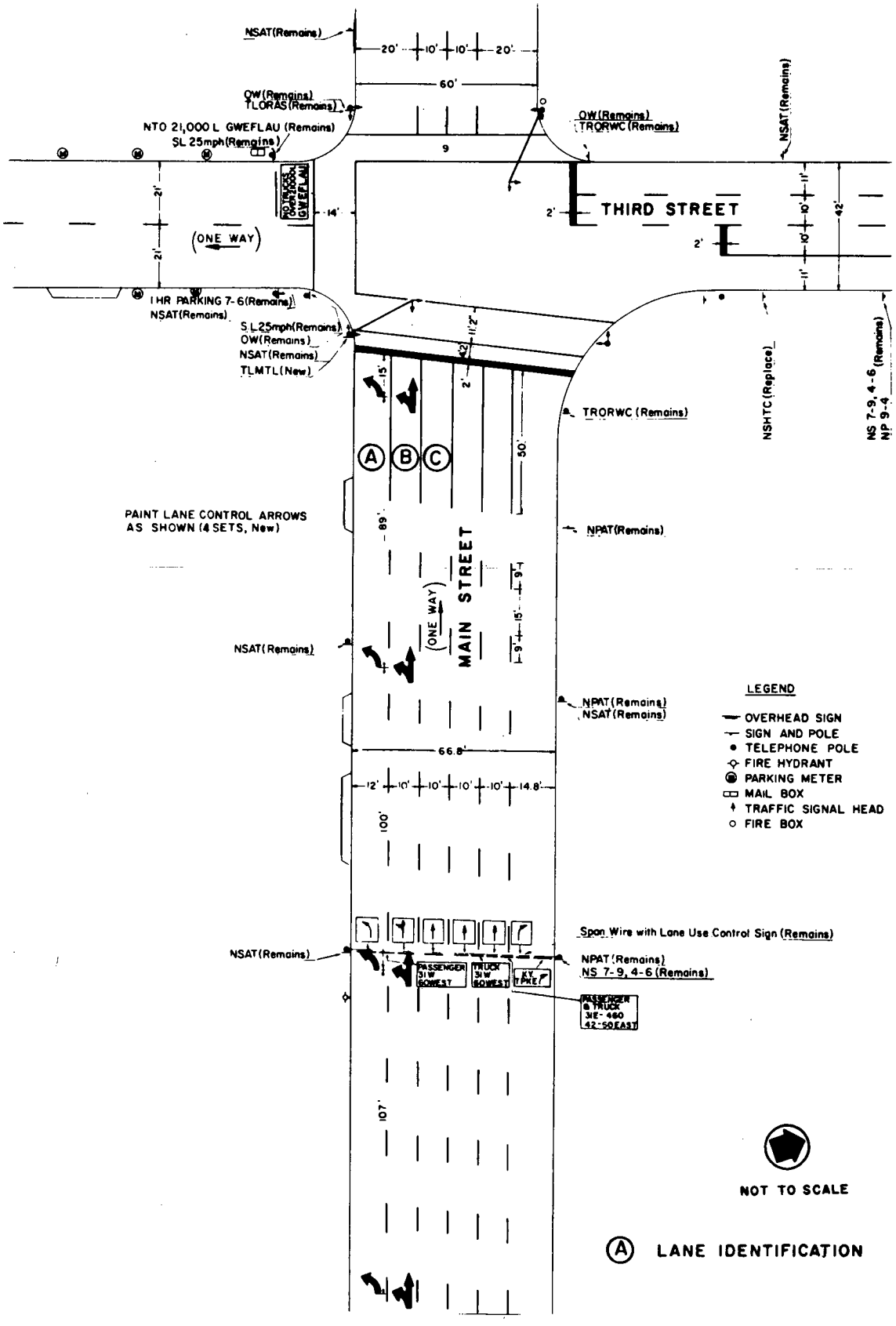


Figure 12. Experiment D67, design plan.

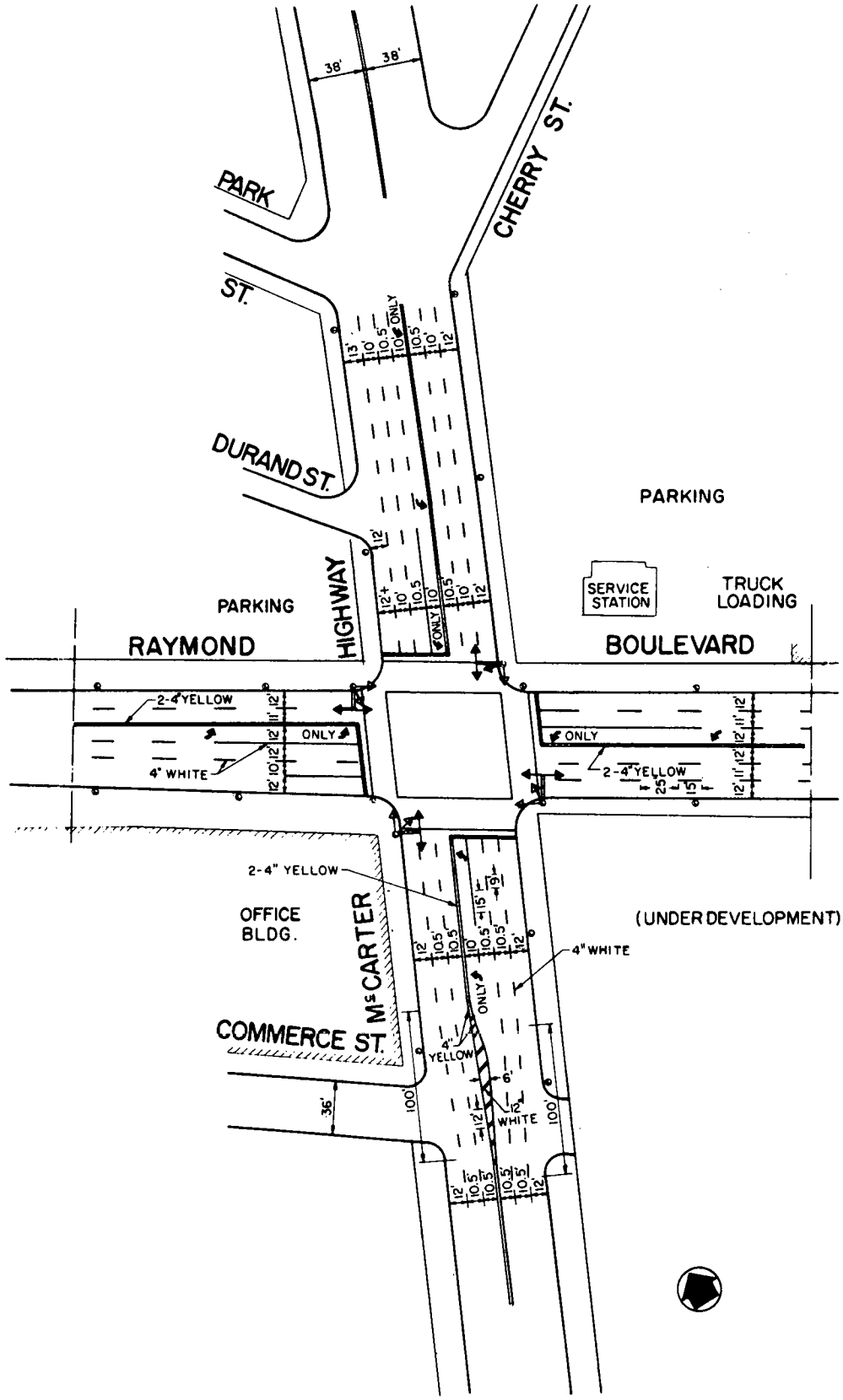


Figure 13. Experiment A7, vicinity map, "after."

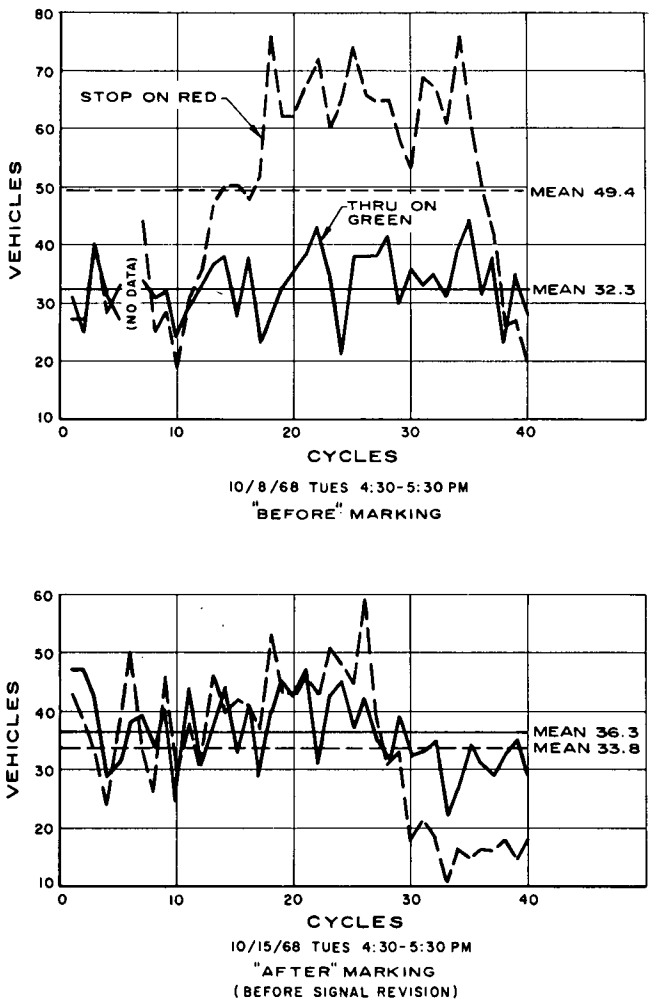


Figure 14. Experiment A7, McCarter Highway southbound vehicles stopped and through.

Curb-Lane Controls

In the study areas, as in many other urban areas, curbside parking was restricted during peak traffic periods on most arterial roadways. This, of course, limited the potential for experimental research. Five experiments were conducted to measure the effect of revisions to parking or truck loading restrictions. In Louisville, the following experiments were designed to measure the effect of additional lane capacity at intersection approaches gained through the parking restrictions: Experiments D2, Parking Restrictions on Oak Street at Sixth Street; D15, Parking Revisions at Oak Street and Shelby Street; and D66, Parking Restrictions on Seventh Street North of Oak Street. The locations of these experiments are shown in Figure 15.

Experiment D2 examines the effect of providing an additional travel lane by prohibiting parking on the northern side of the Oak Street approach to the intersection of Oak and Sixth Streets. The additional lane was primarily a left-turn lane for vehicles turning from Oak Street into Sixth Street, so that the flow of through traffic on Oak Street would not be delayed by turning vehicles. The city of Louisville, under strong pressure from merchants in the area, limited the "after" phase of the experiment to only three days. Therefore, the time given to drivers to adjust to the new situation may not have been adequate. Measurements of delay time made at the intersection indicate that no significant change occurred in the morning peak traffic period, when the approach volume was approximately 700 vehicles per hour (vph). However, a 20-percent reduction of delay time was experienced in the afternoon peak period, when the approach volume was approximately 1,250 vph. This finding agrees with those of several other experiments, which indicate that little or no benefits are experienced due to elimination of parking at intersection approaches during periods of light traffic volume.

Experiment D15 investigated the effect of banning parking on the eastbound and southbound approaches to the intersection of Oak Street and Shelby Street (Fig. 16). Vehicle volumes of 1,100 on the Oak Street approach and 950 on the Shelby Street approach showed no significant change resulting from the restrictions of parking. The measurements made at the Shelby Street approach during the critical evening peak hour (4:30 to 5:30 PM) showed no significant change in delay time. However, at the Oak Street approach, average delay per vehicle through was 2.51 sec before and 3.83 sec after the revision. This increased delay probably results from alteration of vehicle speed due to the availability of three lanes that changed their relationship to the signal progression on Oak Street.

Experiment D66 involved parking restrictions on the east side of Seventh Street, which were already enforced during the afternoon peak period but were revised, in this experiment, to restrict parking at all times. "Before" and "after" measurements, therefore, are for the morning peak period only, when approximately 800 vehicles used the Seventh Street approach to this intersection. These measurements confirm that no significant benefits to traffic flow result from restrictions of this type under low traffic volumes.

Experiment A68, Parking Revisions at Central Avenue and High Street, in Newark, offered an opportunity to test the effects of similar parking restrictions under heavier traffic use. In the "before" condition, parking was permitted on the 20-ft-wide High Street approach to this intersection, restricting traffic movement to one lane. During the "after" condition, parking was restricted for approximately 250 ft along the east side of High Street to provide two lanes for moving traffic. Traffic flows in the critical PM peak hour are shown in Figure 17. Capacity computations for this intersection indicated that 6 percent more green time could be allocated to Central Avenue under the revised conditions. Accordingly, the signals were reset to increase the Central Avenue green interval from 40 sec to 45 sec of a 90-sec cycle. As a result of these improvements, vehicles stopped on the critical High Street northbound approach during the PM peak hour were reduced 40 percent, from 28.4 to 17.0 per cycle. In the off-peak, southbound direction, the number of vehicles stopped increased 36 percent, from 5.9 to 8.0. Both Central Avenue approaches showed a reduction in the number of vehicles

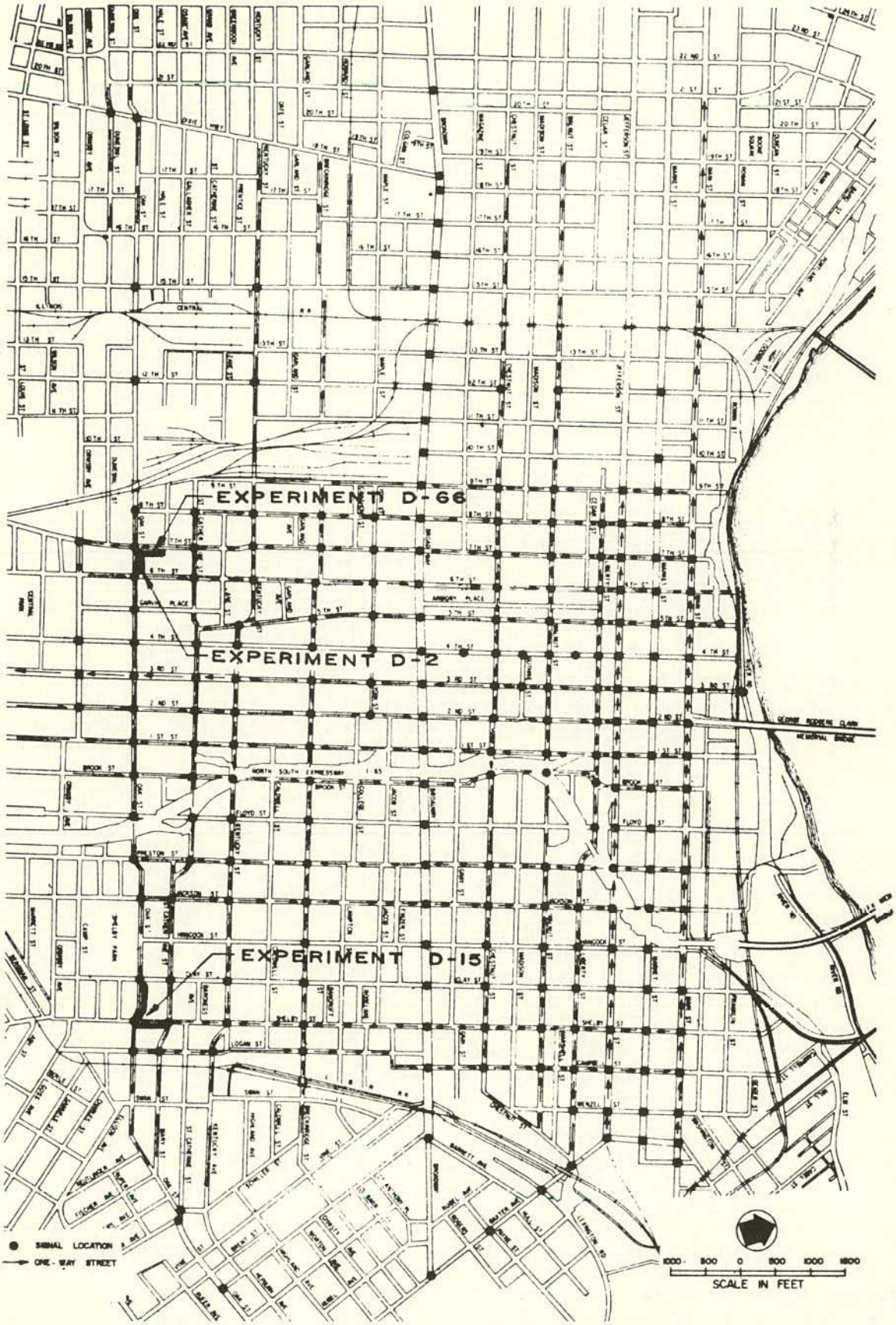


Figure 15. Experiments D2, D15, and D66, vicinity map.

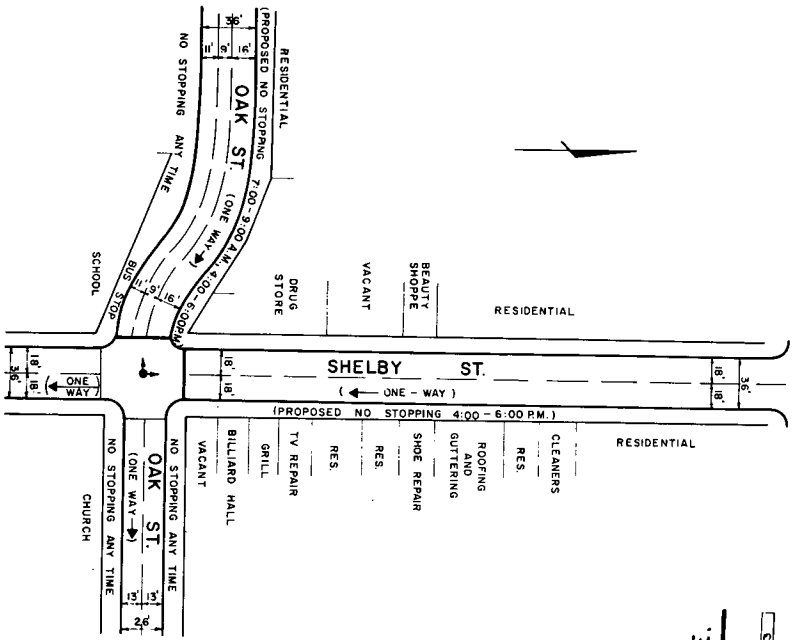


Figure 16. Experiment D15, vicinity map.

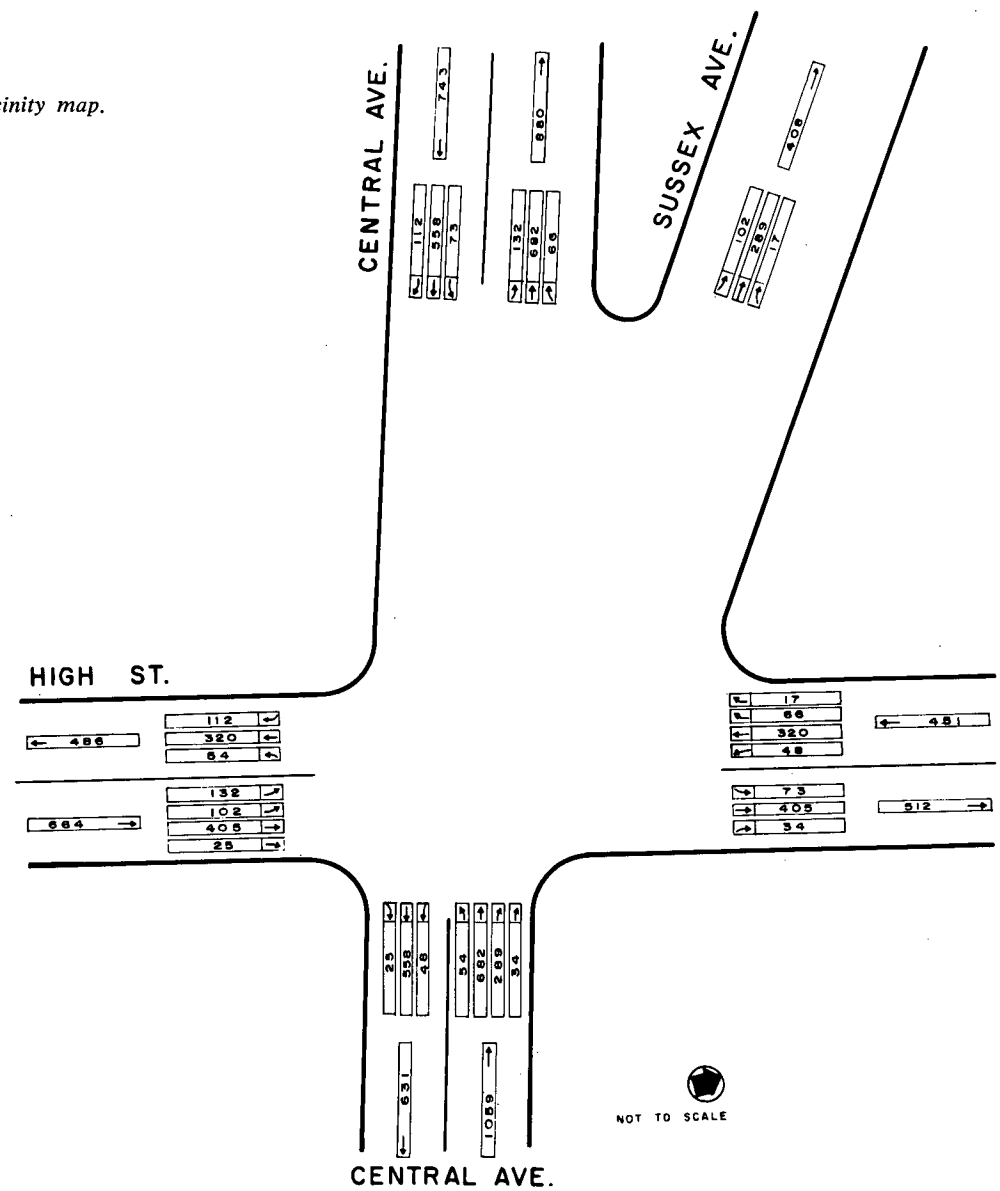


Figure 17. Experiment A68, PM *peak-hour traffic flow.*

stopped—eastbound, from 13.8 to 10.9 per cycle, a reduction of 21 percent; and westbound, the peak direction, from 24.8 to 18.3 vehicles per cycle, a reduction of 26 percent. The number of saturated cycles was also substantially reduced.

Experiment C123, Truck Loading Restrictions, University Avenue Between Orange Street and James Street, measured the change in travel time on University Avenue caused by elimination of truck loading along the west side of University Avenue between Orange Street and James Street. This restriction allowed two lanes of University Avenue to be used by moving traffic. The area of this experiment is shown in Figure 18. The most significant changes measured from this improvement occurred at the University Avenue-Orange Street intersection where the creation of a two-lane departure from the intersection for University Avenue traffic significantly expedited movement through the intersection. A summary of travel time measurements that depict the difference between "before" and "after" conditions is given in Table 5.

Channelization

In the Newark study area, the radial arterial street pattern provided many intersections where channelization could be used to organize conflicting traffic movements. However, the relatively high cost of construction for channelization of complex intersections caused the officials to decide against performing this work as part of the research project. The impending TOPICS program afforded an opportunity to obtain financial assistance for this work. Therefore, channelization was limited to use of paint markings and temporary barriers in the Newark study area.

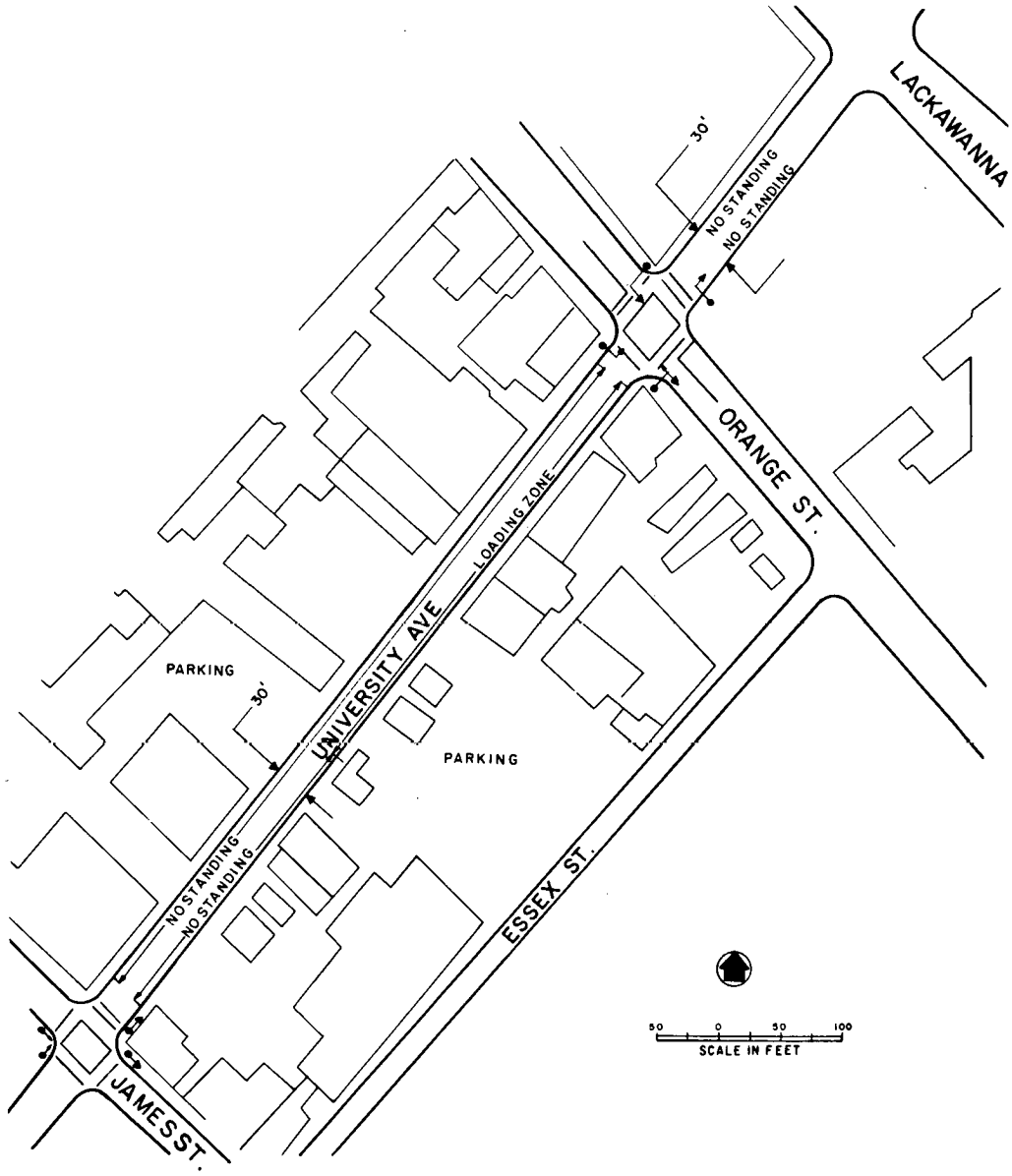


Figure 18. Experiment C123, vicinity map.

TABLE 5

EXPERIMENT C123, SUMMARY OF SPEED AND DELAY ANALYSIS

	SEGMENT 1—LACKAWANNA AVE. TO ORANGE ST.				OVER-ALL LENGTH—LACKAWANNA AVE. TO CENTRAL AVE.			
	MEAN VALUE				MEAN VALUE			
VARIABLE	"BEFORE"	"AFTER"	DIFFERENCE (%)	SIG. @ α LEVEL SPECIFIED	"BEFORE"	"AFTER"	DIFFERENCE (%)	SIG. @ α LEVEL SPECIFIED
Travel time (sec)	54.8	45.3	−17.3	0.01	154.7	141.8	−8.3	0.10
Delay time (sec)	37.0	28.4	−23.2	0.025	73.7	68.1	−7.6	NS[a]
No. of stops	1.1	0.9	−18.2	0.005	8.4	2.1	−12.5	0.05

[a] Difference not significant for $\alpha \leq 0.10$.

Experiment A33, Pedestrian-Vehicle Conflict Control, Market Street at Washington Street, used painted islands to relocate pedestrian crosswalks to eliminate conflicts between turning vehicles and pedestrians. Conditions before and after implementation of this experiment are shown in Figure 19. Relocation of the crosswalks, as shown, eliminated some conflicts between turning vehicles and pedestrians and, in addition, made the intersection more compact. Because of the change in clearance interval required for this intersection and to adjust to current traffic volumes, the signal timing was revised. With the exception of eastbound Market Street, significant decreases in delay time and frequency of stops were measured for all intersection approaches. The revised signal timing decreased the green interval for eastbound Market Street by 4.1 percent, causing an increase in vehicles stopped and delay time at that approach.

In Louisville, the basic grid pattern of roadways did not afford many opportunities for channelization. Four experiments were implemented using painted islands, curbing, and temporary barriers for channelization.

Experiment D8 involved use of channelization at the transition from two-way to one-way movement on Jefferson Street at Ninth Street. At this location eastbound traffic is diverted to Liberty Street. Between Ninth Street and Baxter Avenue, Jefferson Street and Liberty Street operate as a one-way pair, with Jefferson Street carrying westbound movements and Liberty Street accommodating eastbound traffic. A significant number of wrong-way movements at the Jefferson Street-Ninth Street intersection occurred, creating a hazardous condition. Experiment D8 proposed signing and channelization, as shown in Figure 20, to correct this condition. In addition, some signing and pavement markings were used to expedite turning movements at the intersection of Ninth and Liberty Streets. As a result of this improvement, a significant reduction in wrong-way movements was measured, as given in Table 6.

Experiment D10, Channelization at St. Catherine Street and Floyd Street, proposed a design to eliminate turns at the intersection of these streets which interfered with the movement of traffic from the I-65 off-ramp. This design is shown in Figure 21. Turns from St. Catherine Street westbound to Floyd Street southbound were prohibited, as were turns from the off-ramp into Floyd Street northbound. Curbing was extended westward, separating the ramp and St. Catherine Street to the Floyd Street intersection. Pavement markings and overhead signs were placed to indicate the proper lane use, and curb-mounted signs were placed to indicate the turn restrictions. A significant reduction in number of vehicles stopped and delay time for ramp vehicles was experienced as a result of these improvements. No significant change in these measurements was experienced for the St. Catherine Street traffic. The "before" and "after" measurements for the ramp approach to this intersection are given in Table 7.

Experiment D13 investigated traffic congestion, often causing queues extending backward onto I-65 at the off-ramp to Brook Street. This ramp joins Brook Street at its intersection with Jacob Street one block south of Broadway. At the intersection of Brook Street and Broadway, an extremely heavy left-turning movement was found to be the cause of this congestion. A plan was proposed that would afford two lanes for left-turning traffic—a mandatory left lane and an optional left or through lane. Jacob Street traffic was diverted from the intersection, and Brook Street traffic was confined to the two right lanes by use of a channelizing island. This design is shown in Figure 22 and Figure 23. Lane striping, lane-use pavement markings, and overhead signs were placed, and the stop line for eastbound Broadway traffic was set back to provide room for a large turn radius for left-turning traffic. As a result of these improvements, travel time for autos from the ramp making the left turn at Broadway was reduced from an average of 73.4 sec in the "before" measurements to 38.5 sec in the "after" measurements during the critical morning peak hour. Also, the number of cycles (90 sec) when vehicles were stored on the ramp at the end of the Brook Street red interval at Broadway was reduced from an average of 13 before the improvements were made to none after the improvements were in effect.

Experiment D68 used channelization to assist traffic flow at another I-65 off-ramp, located at Brook and Jefferson Streets. Violation of turn regulations, as well as ramp queues, had become a significant problem at this location.

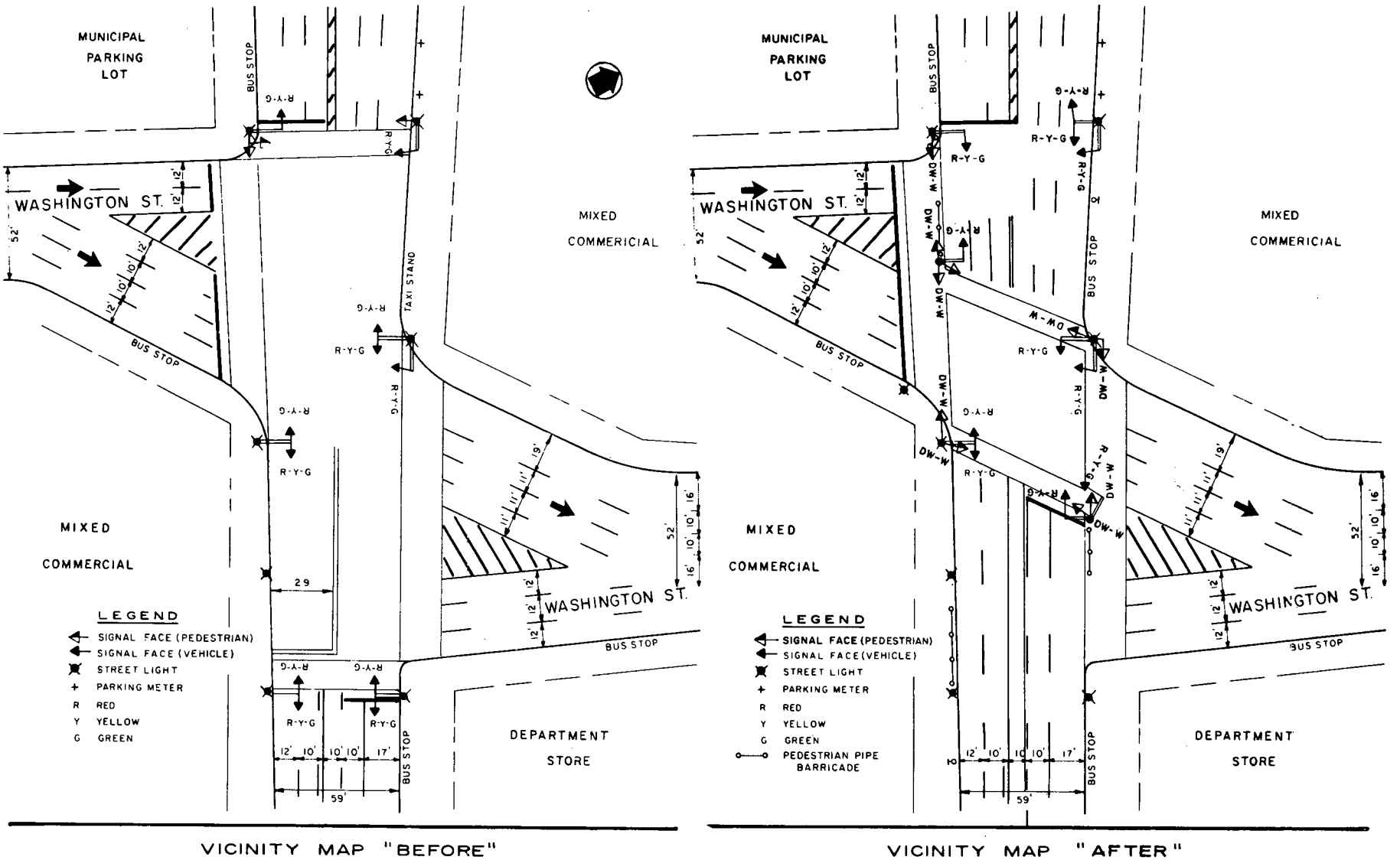


Figure 19. Experiment A33, pedestrian-vehicle conflicts.

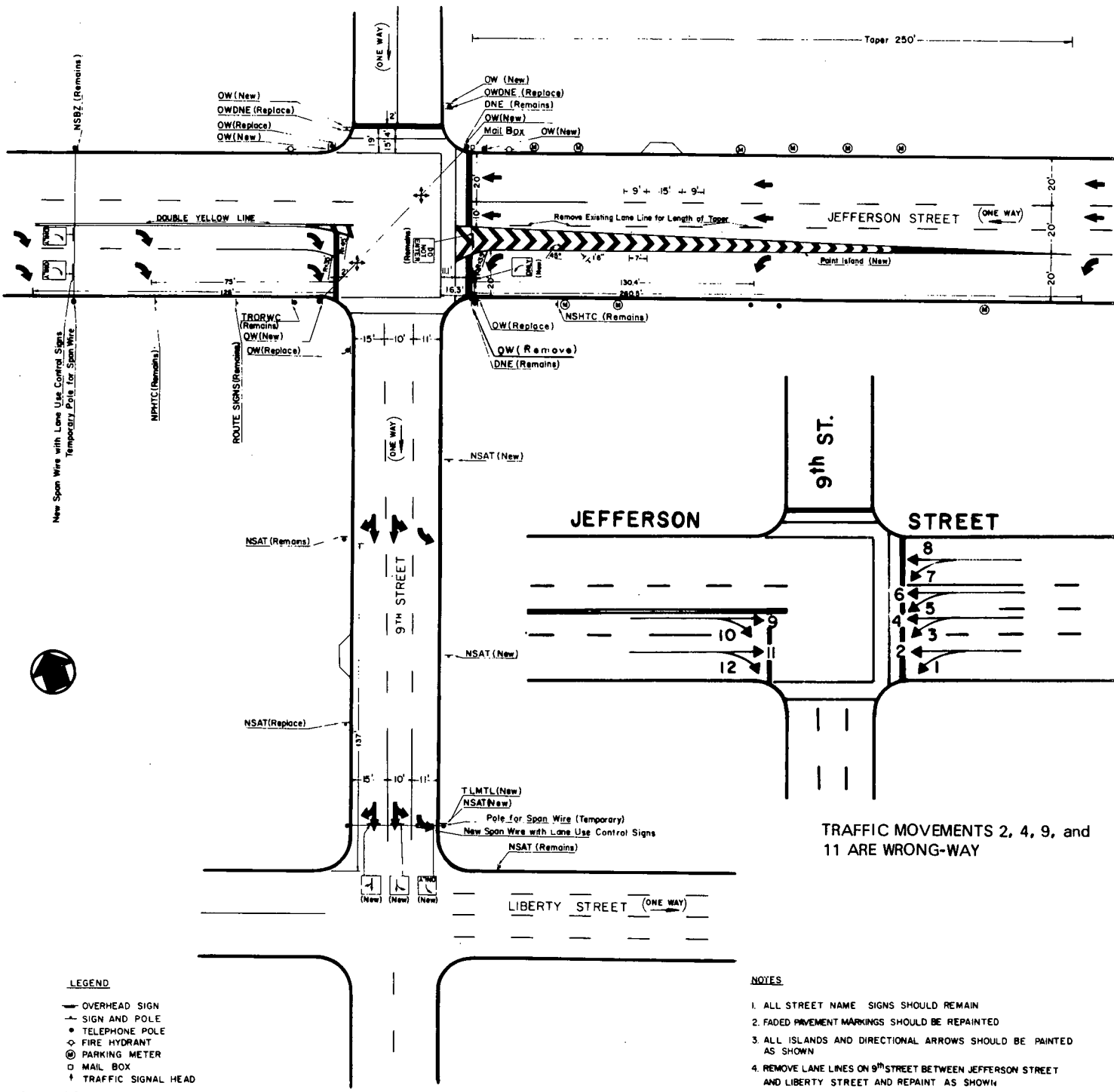
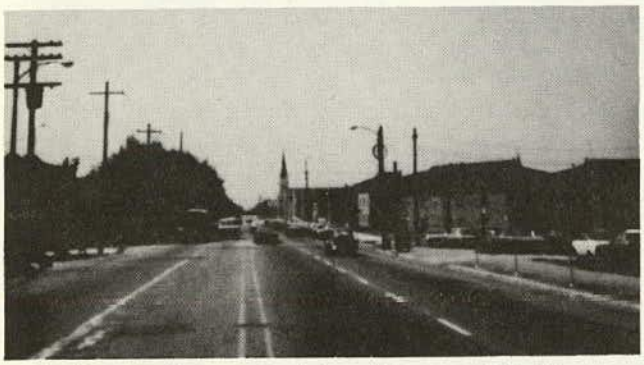


Figure 20. Experiment D8, design plan.

This ramp is a major point of access to the downtown Louisville area. To avoid turning movements at the intersection of Brook and Jefferson Streets, the original design divided the ramp to afford separate approaches to Brook Street northbound and Jefferson Street westbound. Traffic was required to use the appropriate approach to avoid turning at the intersection. During the morning peak period, long queues often formed on the ramp, and drivers would use the approach that afforded the least congestion, regardless of the turn restrictions. To circumvent these violations and to afford more capacity for ramp traffic at the intersection, a channelization was proposed that forced Jefferson Street traffic to use the three right lanes, reserving the two left lanes for ramp traffic. At Brook Street, a similar construction was used to reserve use of the right lane for ramp traffic. Barricades were placed to interfere

Jefferson Street looking East from 9th Street



Jefferson Street looking West at 9th Street

with turning movements from the ramp approaches, overhead signs were reworded to include the word ONLY in directing traffic to the Brook Street or Jefferson Street alternates, and lanes were striped in accordance with the plan shown in Figure 24. In addition, overhead signs and lane-use markings were placed to control traffic in the weave section on Jefferson Street between Brook and First Streets. This weave involved ramp traffic moving straight through the First Street intersection conflicting with Jefferson Street traffic making a left turn at First Street. Measurements made to compare "before" and "after" traffic conditions show that ramp volumes for the AM peak hour increased 8.3 percent from 1,541 to 1,669 vehicles. A shift of traffic from the Brook Street approach to the Jefferson Street approach was also measured, with a decrease of 124 vehicles using the ramp to Brook Street and 252 more vehicles using the ramp to Jefferson Street. The number of violations of turn regulations was greatly reduced. Vehicles stopped on red at the Brook Street approach decreased from 32 percent to 24 percent of a total volume of approximately 1,150 vehicles. At the Jefferson Street approach to its intersection with First Street, vehicles stopped on red showed a similar reduction from 33 percent to 24 percent of an average volume of 2,670 vehicles.

TABLE 6

EXPERIMENT D8, WRONG-WAY MOVEMENTS ON JEFFERSON STREET APPROACH

DATA GROUP	VEHICLES, BY MOVEMENT			
	(*a*) Westbound			
	WRONG WAY		LEFT TURN	
	2	4	1 & 3	ALL
1	2	14	1,162	1,178
2	0	0	1,052	1,052
3	1	0	1,126	1,127
	(*b*) Eastbound			
	WRONG WAY		RIGHT TURN	
	9	11	10 & 12	ALL
1	5	4	4,607	4,616
2	0	1	4,662	4,663
3	1	1	4,648	4,650

Signal Controls

The subject of signal controls includes study of both the individual intersection and the offset relationships between intersections.

Intersection Signal Controls

Included within the subject of intersection signal controls are cycle length changes, allocation of green time, turn controls, and pedestrian controls. With nearly 200 signalized intersections in each of the study areas, the potential for experimentation was almost unlimited.

Experiment A31, Pedestrian Control at Halsey Street, Academy Street, and Bank Street, examines the effect on pedestrian and vehicular traffic of double cycling to operate traffic signals on a cycle length of one-half the system cycle length at the intersections of Bank and Academy Streets with Halsey Street. These intersections are located in the heart of the shopping district of Newark (Fig. 25). The purpose of double cycling is to shorten the waiting time for pedestrians. A large number of pedestrian violations of signal indications was observed in this area. It was anticipated that a shorter interval would reduce violations. Comparison of "before" and "after" measurements for the period of 4:30 to 5:30 PM indicated a significant reduction in violations. Of a total of approximately 7,800 pedestrians, 26.3 percent violated the signal indications during the "before" period and 17.4 percent violated them during the "after" period at the intersection of Academy Street and Halsey Street. Although the number of vehicles stopped on red increased significantly, the duration of stops was reduced, and the balance between these measurements seems to be favorable. This indicates a reduction of pedestrian interference to movement of vehicles. In areas where large volumes of pedestrians are conflicting with circulating traf-

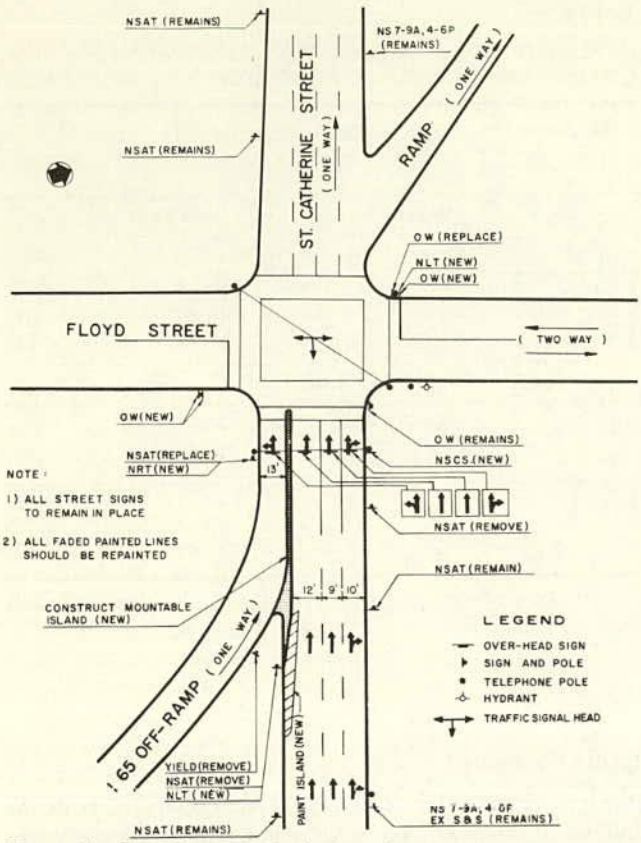


Figure 21. Experiment D10, design plan.

St. Catherine Street looking East at Floyd Street and the I-65 Ramps

St. Catherine Street looking West at I-65 Ramp

fic on minor streets, this expedient may be used to enhance pedestrians' convenience and safety.

Experiment A53, Signal Retiming at First Street and Central Avenue, involved a minor change of the allocation

TABLE 7

EXPERIMENT D10, SUMMARY OF RESULTS, RAMP APPROACH, ST. CATHERINE STREET AND FLOYD STREET

VARIABLE	TIME PERIOD	UNIT	AVERAGE "BEFORE"	AVERAGE "AFTER"	DIFFERENCE	CHANGE (%)	SIG. @ $\alpha=0.05$
Vehicles stopped on red	AM	Veh	3.2 [a]	2.6 [a]	−0.6	−18.8	Yes
	PM	Veh	2.0 [b]	1.0 [b]	−1.0	−50.0	Yes
Vehicles through	AM	Veh	6.9 [a]	7.1 [a]	+0.2	+2.9	No
	PM	Veh	4.8 [a]	4.4 [a]	−0.4	−8.3	No
Delay time on red	AM	Sec	35.8 [a]	24.3 [a]	−11.5	−32.2	Yes
	PM	Sec	29.0 [b]	11.0 [b]	−14.0	−48.3	Yes
Delay time on red/vehicle stopped on red	AM	Sec/veh	11.5 to 13.0 [b]	9.0 to 11.5 [b]	−1.5 to −2.5	−11.5 to −21.8	Yes
	PM	Sec/veh	13.8 [b]	9.0 [b]	−4.8	−34.8	Yes
Delay time on red/vehicle through	AM	Sec/veh	5.0 [a]	3.5 [a]	−1.5	−30.0	Yes
	PM	Sec/veh	6.0 [b]	2.7 [b]	−3.3	−55.0	Yes

[a] Mean values.
[b] Median values.

of green time at the subject intersection. Central Avenue is a major east-west arterial. First Street is a collector roadway that carries a substantially smaller traffic volume than Central Avenue. However, in the "before" situation the allocation of green time had disproportionately favored Central Avenue. The vehicular volumes and "before" and "after" signal data are shown in Figure 26. Long queues of traffic on First Street had been observed in the initial investigations, indicating a large amount of delay time for the First Street approaches. The revised signal timing allocated 8 percent more green time to First Street in an effort to minimize delay time for the intersection. The experiment resulted in a 36-percent decrease in vehicle delay time at all approaches during the critical evening peak period, from 13.2 to 8.5 veh-hr. This represents a 7-sec saving for each vehicle through the intersection. A 72-percent decrease was realized on the First Street northbound approach, from 7.5 to 2.1 veh-hr, the difference of 0.7 veh-hr being an increase in delay time for the Central Avenue approaches. The very striking reduction in delay time that resulted from a relatively minor adjustment to signal timing illustrates the importance of periodic checking to determine that signal settings accurately reflect changes in traffic flow. This experiment produced a very noticeable local effect; however, no significant changes in volume or other network effects were measured. This experiment also illustrates the need for practical survey methods and analysis procedures leading to the optimization of signal timing through minimizing delay time at an intersection.

Experiment A69, Revision of Signal Timing at Central Avenue and West Market Street, investigates the engineer's ability to control a driver's choice of route by allocating more signal time to the preferred route. During the morning peak hour, 7:30 to 8:30 AM, an exceptionally heavy left-turn movement from West Market Street southbound to Central Avenue eastbound interfered with establishing a signal progression for eight signals on Central Avenue. Because this intersection is critical in establishing a green band for Central Avenue, it would be desirable to have drivers use Central Avenue for their inbound trips, thereby becoming part of the platoons passing through this intersection. This diversion of trips is possible because the available arterial streets for east-west travel in this corridor are nearly parallel. West Market Street lies diagonally across and interconnects several of the available east-west arterials of this corridor, as shown in Figure 27. This experiment preceded the work of establishing the signal progression alleviating the situation at the critical bottleneck at Central Avenue and West Market Street.

Traffic signal timing for the "before" and "after" conditions of this experiment are shown in Figure 28. As a result of the preferential treatment that allocated 70 percent of the green time to Central Avenue, the number of vehicles stopped per cycle on Central Avenue eastbound decreased 51.4 percent, whereas the number stopped on the southbound West Market Street approach increased 160.5 percent. Although this measurement indicates a worsened condition for the intersection, the purpose of the experiment was accomplished. In the peak hour, approximately 160 drivers who had used the West Market Street route during the "before" period apparently chose to use the Central Avenue route during the "after" period. Of these 160 vehicles, about 42 were eliminated from the left turn from southbound West Market Street to Central Avenue.

Experiment D20 compared the efficiency of fully actuated signal controls at a critical intersection in an otherwise interconnected fixed-time control system. This experiment was conducted at the intersection of Main and Second Streets in Louisville, at the approach to the Clark Bridge over the Ohio River. Prior to this experimentation, a fully actuated controller had been used to monitor traffic flows at this location. Time-space diagrams for Main Street and Second Street indicated the feasibility of substituting a fixed-time interconnected controller at this location. "Before" measurements indicate the efficiency of the fully actuated controller; "after" measurements depict traffic conditions under fixed-time interconnected control. Measurements taken for the purpose of comparing these two situations indicate a substantial betterment of conditions in the "after" period (Table 8). No significant change in traffic volumes was encountered during the "before" and "after" periods that would affect these measurements.

Experiment A40, Signal Rephasing at Central Avenue and High Street, investigates the benefits of providing a separate turn phase for an intersection approach having a large number of left-turning vehicles. This experiment was performed prior to Experiment A68, which provided a second lane for moving traffic at this approach through the elimination of parking. Under single-lane operation, the large number of left-turning vehicles from High Street northbound to Central Avenue westbound during the afternoon peak period was impeding traffic movement on High Street. The revised signal timing is shown in Figure 29. The introduction of the leading green interval did produce significant improvements. There was a substantial decrease in the mean number of vehicles queued per cycle and the number of saturated cycles on the northbound High Street approach. No adverse effects were measured on any of the

TABLE 8

EXPERIMENT D20, TRAVEL CHARACTERISTICS, SPEED AND DELAY DATA

		"BEFORE"		"AFTER"	
VARIABLE	TIME PERIOD	MEAN	NO. OF OBS.	MEAN	NO. OF OBS.
Travel time (sec)	AM	300.2	30	268.8	9
	Midday	287.3	23	298.5	8
	PM	297.5	24	316.8	6
No. of stops	AM	1.6	31	0.7	9
	Midday	1.2	23	0.9	8
	PM	1.6	24	1.8	6
Delay per stop (total run) (sec)	AM	21.1	26	16.0	5
	Midday	20.4	17	21.8	6
	PM	20.8	22	17.3	5
Delay per stop (at Main St.) (sec)	AM	14.8	9	0	0
	Midday	22.4	5	35	1
	PM	19.4	8	0	0

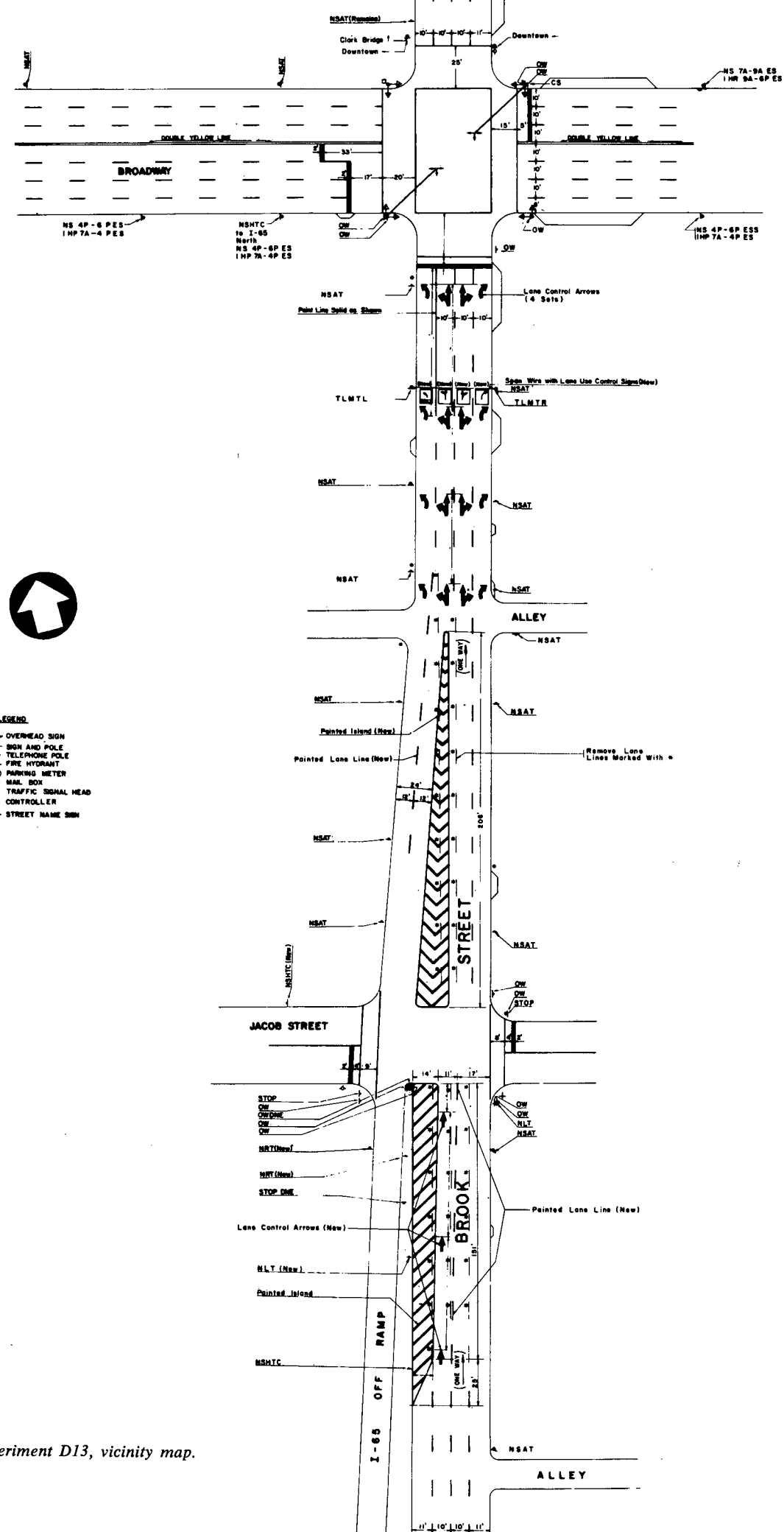


Figure 22. Experiment D13, vicinity map.

Brook Street Looking South at Jacob Street

I-65 Off-Ramp to Brook Street looking South at Jacob Street

other approaches. Figure 30 shows the effect of this change on vehicles through and queue lengths at the critical High Street northbound approach.

Signal Coordination

The coordination of signals at adjacent intersections serves to reduce the number of stops and delay time for all vehicles moving through an area. In an urban downtown signal system with closely spaced signalized intersections, numerous mid-block frictions, high volumes of turning vehicles, substantial percentages of trucks and buses, and large fluctuations of traffic volume, the problem of signal coordination becomes very complex. To assist in analysis of interacting influences of adjacent signals, the Analog Traffic Signal Model was developed for use in both study areas. This development is described in Appendix D. In addition, the SIGOP * computer program was used in analysis of offset plans for signals of the Louisville study area.

Four experiments conducted in the Newark study area established signal progressions for major arterials of the downtown network. These were Experiment B88 on Springfield Avenue, B90 on Central Avenue, B93 on McCarter Highway, and B100 on Broad Street. Experiment E40 investigated the effect of a revised offset plan for Oak Street in Louisville, and Experiment E35 developed an offset plan for a network of more than 200 intersections in the downtown Louisville area.

Experiment B88, Springfield Avenue Signal Progression, was conducted in two stages, which compared 25-mph and 30-mph speeds of progression for identical offset plans. The offset plan in use prior to this experimentation provided for practically simultaneous operation of the signals. Narrow time bands of 14 percent for outbound and 3 percent for inbound movement could be constructed for this plan. Offset plans for average, inbound preferential, and outbound preferential conditions were developed using a 90-sec cycle. The average offset plan provided for a green band of 24 percent inbound and 22 percent outbound, whereas the preferential outbound green band was 50 percent of the cycle. Figure 31 shows the time-space diagram for the inbound preferential offset plan. Traffic volumes, total vehicle-hours, and average vehicle speeds are summarized for "before" and "after" conditions for morning and afternoon peak periods in Table 9. Analysis of AM bus trip time on Springfield Avenue indicated an average saving of 28.7 percent, from 6.76 to 4.82 min for buses inbound during the morning peak period.

The data given in Table 9 indicate that the peak directions, AM eastbound and PM westbound, showed no significant improvement in Stage 2, when the speed of progression was increased to 30 mph. Observation of conditions on Springfield Avenue indicate that poor pavement conditions and frequent parking violations probably prevent platoons of traffic from maintaining the increased speed of progression.

Experiment B90 conducted on Central Avenue in Newark was similar to Experiment B88, but produced significantly different results. The effects produced by the implementation of separate progression plans to favor the predominant flows of morning and evening peak traffic were studied in this experiment. Between High Street and the East Orange-Newark city line, a distance of approximately 7,900 ft, there are eight signalized intersections. From High Street to West Market Street, Central Avenue is 48 ft wide, divided into four lanes. From West Market Street to the city line, Central Avenue is 60 ft wide, divided into six lanes. Because parking is prohibited only for peak-hour, peak-direction traffic, the number of lanes available for moving traffic is reduced. Traffic volumes and the number

* Traffic Signal Optimization Program.

"THE COURIER JOURNAL", November 21, 1968

Brook Section Traffic Flow Being Changed

The Louisville-Jefferson County Traffic Engineering Department is changing the traffic pattern on Brook between Jacob and Broadway to speed up movement of vehicles, particularly those coming off the North-South Expressway.

Department crews yesterday began painting signs on the pavement for the new traffic-control system. Overhead signs will be hung later.

Under the new system, northbound traffic coming off the expressway onto Brook at Jacob will funnel into two lanes, either of which could be used to turn left onto Broadway. Traffic in the curb lane will have to turn at Broadway, while that in the adjoining lane can either turn left or continue northward on Brook.

In the past, vehicles turning onto Broadway used the curb lane only

Adjoining Lane Has Option

Traffic from the south on Brook will be funneled into two lanes on the east side of the street just south of Jacob. Vehicles in the right curb lane will have to turn right at Broadway, while those in the adjoining lane can turn right or continue on Brook.

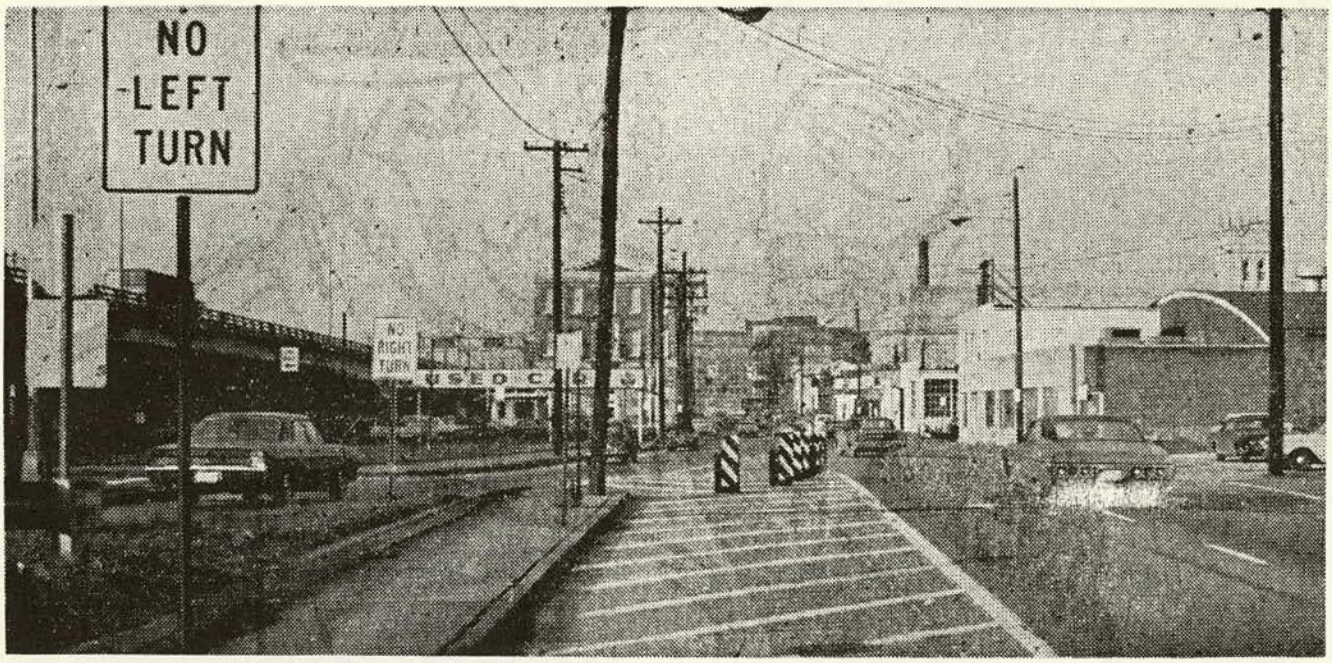
Vehicles coming from the south on Brook and wanting to turn left on Broadway will have to work their way into the center lane on the west side of the street. Those coming off the expressway and wanting to turn right onto Broadway will have to get into the center lane on the east side of Brook.

Parking meters are being removed and parking will be banned on both sides of Brook from a point just south of Jacob to Broadway.

Traffic Engineer Arthur R. Daniel Jr., said relatively little revenue will be lost by removal of the meters. They have been used less than 40 per cent of the time, he explained.

Daniel said that Jacob will be made one way west between Brook and Second under the new system.

"THE LOUISVILLE TIMES," February 6, 1969



Staff Photo

A row of barricades erected on Brook at Jacob is expected to reduce the traffic hazard at the intersection. The obstruction is located where northbound traffic exits from the North-South Expressway, and it will prevent drivers from cutting sharply east across lanes carrying northbound traffic on Brook. Jacob is a block south of Broadway.

Restrainer

Figure 23. Experiment D13, newspaper publicity.

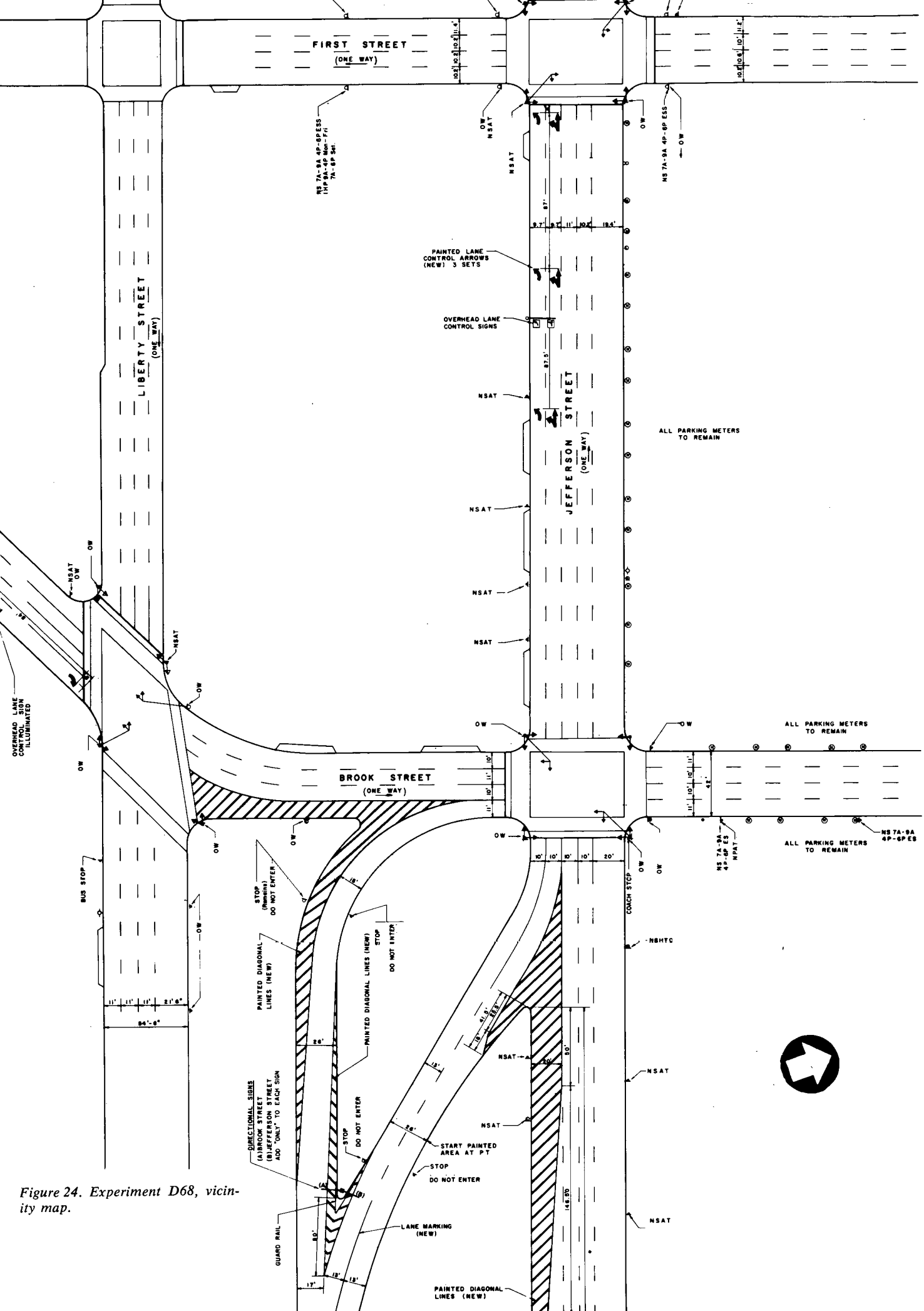
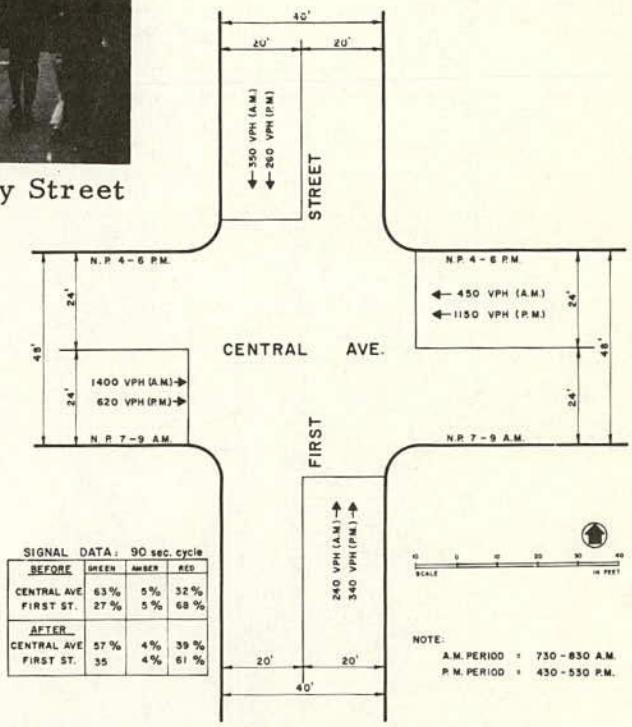


Figure 24. Experiment D68, vicinity map.

Halsey Street looking South from Bank Street

Halsey Street looking North from Academy Street

Figure 25. Experiment A31, Halsey Street.



SIGNAL DATA: 90 sec. cycle

BEFORE	GREEN	AMBER	RED
CENTRAL AVE.	63%	5%	32%
FIRST ST.	27%	5%	68%
AFTER			
CENTRAL AVE	57%	4%	39%
FIRST ST.	35	4%	61%

Figure 26. Experiment A53, vicinity map.

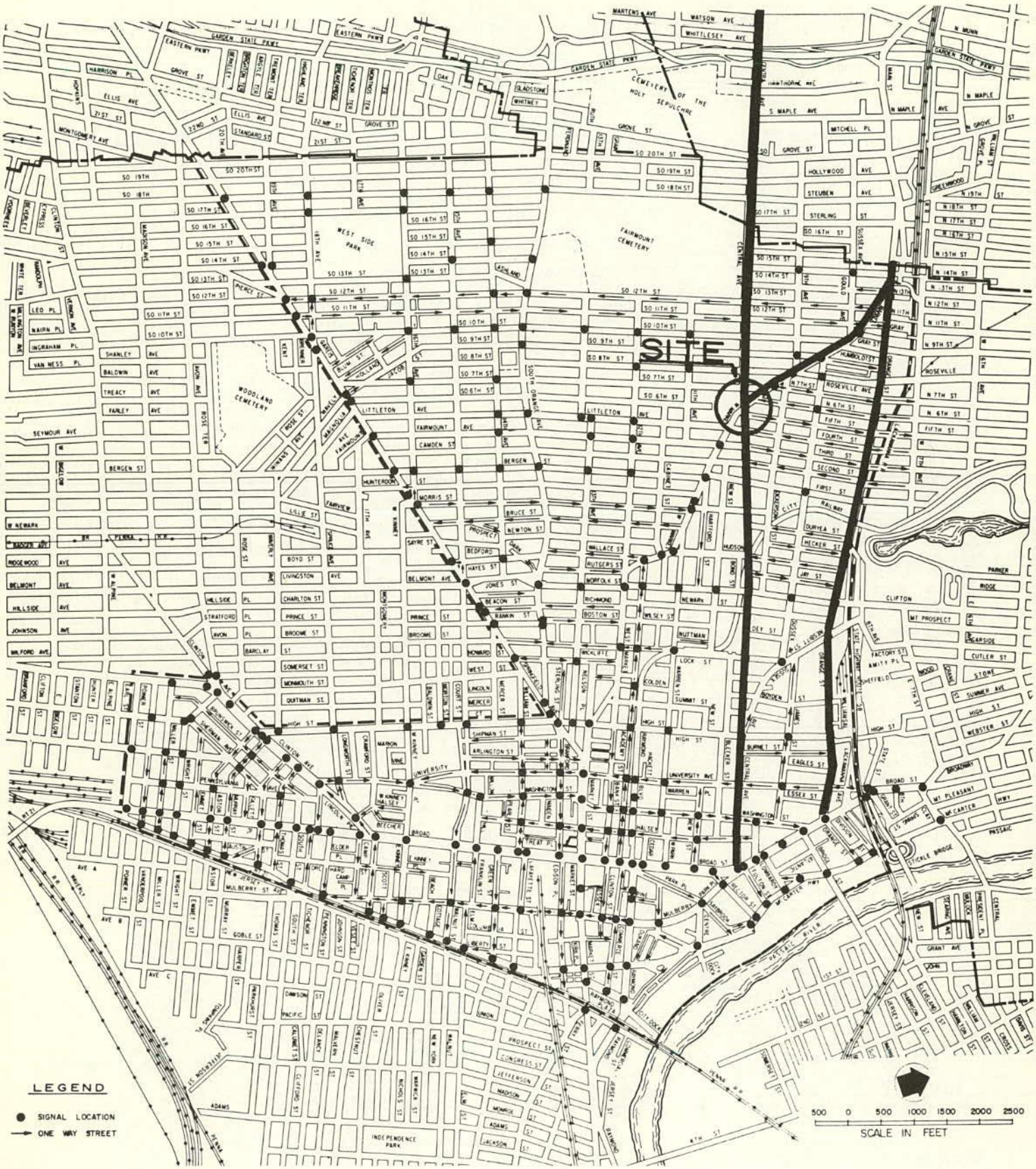


Figure 27. Experiment A69, location map.

TRAFFIC SIGNAL TIMING

CENTRAL AVENUE	G	A	R	R	R
WEST MARKET ST	R	R	R	G	A
TIME-"BEFORE"(%)	44	4	4	44	4
TIME-"AFTER" (%)	62	4	4	26	4

Cycle = 90 Seconds

TRAFFIC SIGNAL OPERATION

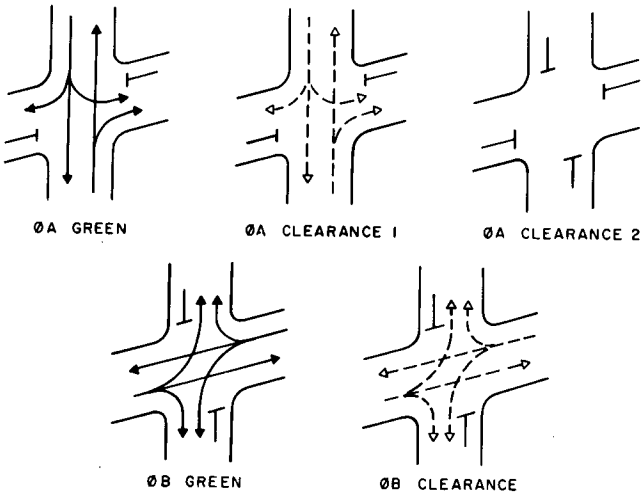
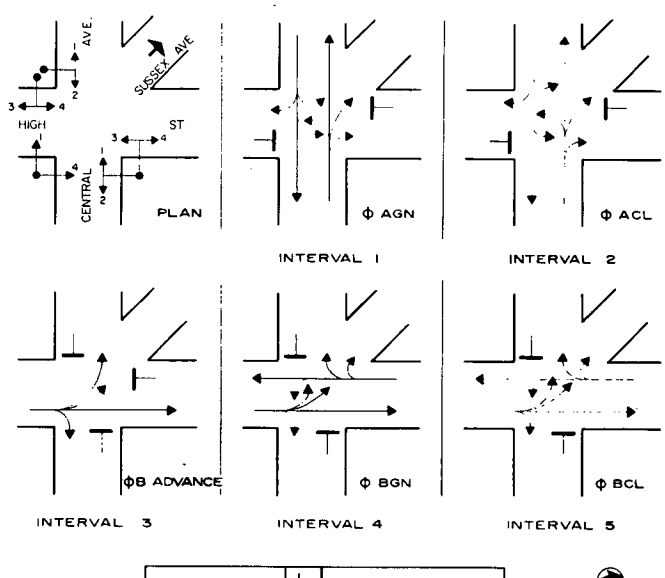


Figure 28. Experiment A69, signal timing.



STREET	SIGNAL NO.	INTERVAL 1	2	3	4	5
CENTRAL AVE EB	1	G	A	R	R	R
CENTRAL AVE WB	2	G	A	R	R	R
HIGH ST. NB	3	R	R	G	G	A
HIGH ST. SB	4	R	R	R	G	A
"BEFORE"- TIME IN %		44	5	2	44	5
"AFTER"- TIME IN %		44	5	12	34	5

CYCLE = 90 SECONDS

Figure 29. Experiment A40, signal timing.

of lanes available for each direction of traffic movement during each time period are shown in Figure 32. Traffic signal controllers are either one- or two-dial fixed-time, without interconnection. Because of the limitations of this equipment, it was necessary to set the controllers manually for each time period of each day during the experiment.

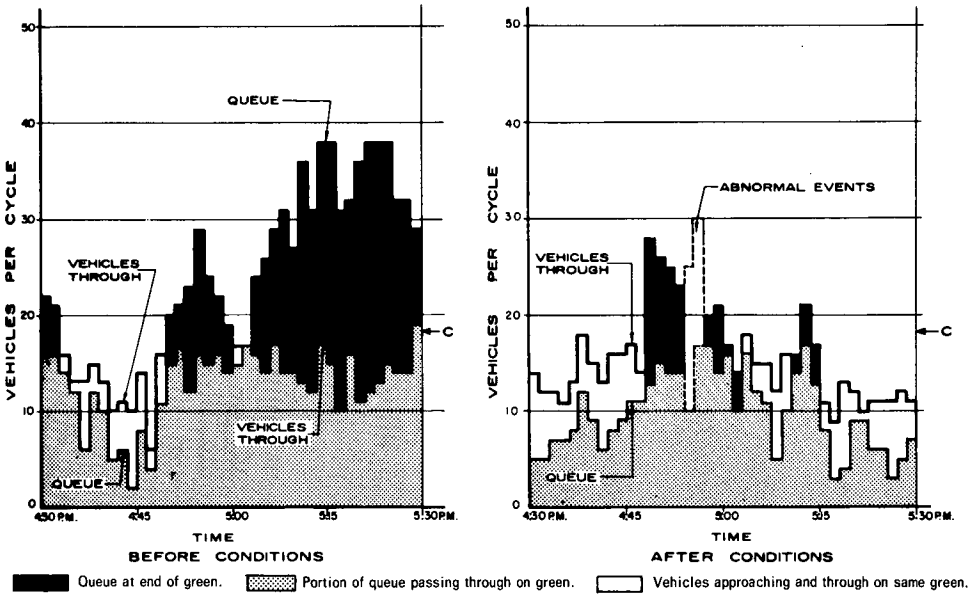


Figure 30. Experiment A40, High Street northbound vehicles queued and through.

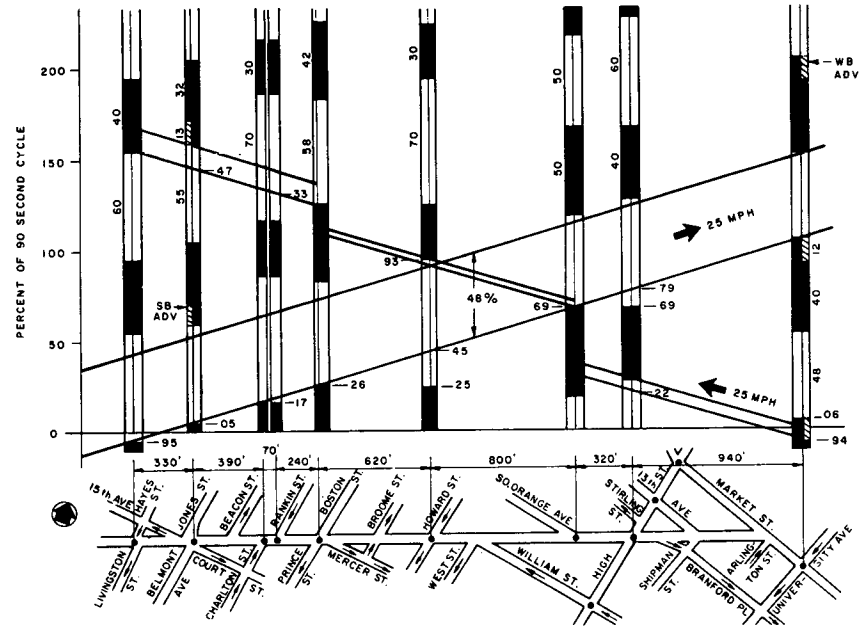


Figure 31. Experiment B88, inbound offset plan.

This, of course, limited the time during which the experiment could be conducted.

In the "before" conditions, signals were set in a basically simultaneous offset pattern, which was retained for midday use during the experiment. Preferential eastbound, morning, and westbound, evening, progressions were developed for use during the peak periods. Using data from the travel time surveys, speeds of progression of 35 mph in the six-lane section and 30 mph in the four-lane section were selected for use. The inbound progression provides a green band of 55 percent, and the outbound progression, 44 percent of a 90-sec cycle. Analysis of travel time indicated that no significant improvement was realized as a result of these progressions. While the number of delays at signalized intersections was reduced, the number of delays at unsignalized intersections increased and total delay time was not significantly changed. Observation of conditions on Central Avenue during the experiment indicated several reasons for this lack of improvement. Poor pavement conditions in the curb lane caused drivers to avoid use of this lane. Numerous violations of parking prohibitions interfered with use of curb lanes. Guards at school crossings often interfered with passage of a traffic platoon rather than taking advantage of the gaps between platoons. Cross-street traffic at unsignalized intersections frequently interfered with Central Avenue traffic. The composite effect of all of these interferences was to prevent effective platooning

TABLE 9

EXPERIMENT B88, COMPARISON OF VEHICLE-HOURS OF TRAVEL

			"BEFORE"		"AFTER" STAGE 1 [b]		"AFTER" STAGE 2 [b]	
TIME PERIOD	DIRECTION	PEAK-HOUR VOLUME [a]	SPEED (MPH)	VEH-HR	SPEED (MPH)	VEH-HR	SPEED (MPH)	VEH-HR
AM	EB	1,057	11.1	67.97	19.1	40.69	16.6	46.83
	WB	405	16.4	18.06	12.1	23.77	15.3	19.28
	ALL	1,462		86.03		64.46		66.11
PM	EB	401	11.2	25.94	12.3	23.94	10.4	27.63
	WB	926	9.8	72.32	12.7	53.15	13.0	54.08
	ALL	1,327		98.26		77.09		81.71

[a] AM peak hour (7:30-8:30); PM peak hour (4:30-5:30)
[b] Stage 1 speed of progression, 25 mph. Stage 2 speed of progression, 30 mph.

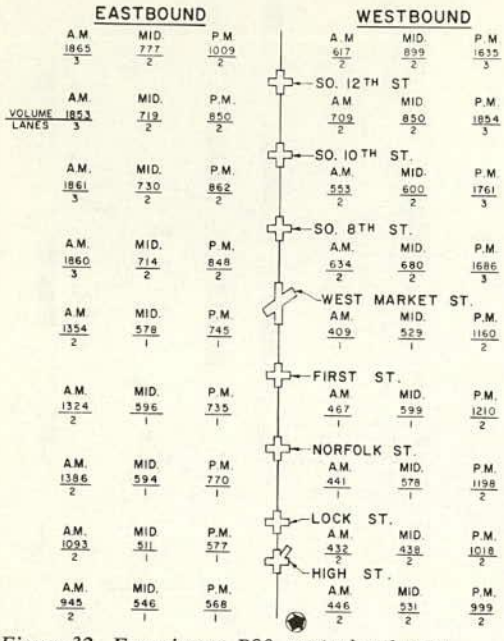


Figure 32. Experiment B90, typical volumes.

Looking West From Broad Street

Looking West From Washington Street

of traffic and to hinder the drivers' ability to maintain the speed of progression.

Experience gained during this experiment forcefully illustrates the need for strict enforcement of parking regulations, control of unsignalized intersections, and elimination of other interferences to traffic flow for successful implementation of signal progressions. The ability to successfully develop and maintain platoons traveling at the designed speed of progression is dependent on rigid enforcement as well as traffic engineering.

Experiment B93 developed signal progressions for a 2.58-mile section of McCarter Highway. This section of highway has 29 signalized intersections. The pavement varies from 50 to 80 ft in width and was marked with a center line only. Generally, the pavement is used as four lanes, two in each direction, with short sections of roadway having six lanes. Average weekday traffic volume varies from 33,000 to 38,000 vehicles along this section of roadway. There are approximately 1,400 vehicles during the peak hour in the peak direction. McCarter Highway is the major north-south truck route through the area, and the aforementioned traffic volume includes 20 to 25 percent heavy trucks.

In the "before" condition, signals were set in a basically simultaneous pattern, using the 90-sec cycle that is common to all signals of the downtown area. In the first phase of this experiment, an average offset pattern was developed, using a speed of progression of approximately 26 mph and

maintaining green bands averaging about 28 percent of the cycle. In the second phase of the experiment, preferential progressions were developed, with the highway divided into two sections to separate areas having opposing directions of peak flow.

Market Street, which is approximately opposite the center of downtown Newark, was used as the dividing line between these sections. In the north section, the design speed for northbound traffic was approximately 32 mph, with a green band of between 50 and 55 percent of the cycle. In the south section, the speed of progression was also approximately 32 mph, with band widths varying from 33 to 55 percent of the cycle. Time-space diagrams for the second stage of this experiment are shown in Figures 33 and 34. The only substantial volume change noted over the period of the experiment occurred in the north section during the second phase where afternoon northbound volumes increased 32 percent while southbound volumes decreased 11 percent. Under the average offset condition, morning peak-hour average vehicle speed increased from 12.7 to 14.5 mph in the northbound direction and from 13.1 to 15.3 mph for the southbound direction over the entire length of roadway. In the north section, afternoon peak-hour average vehicle speed for northbound traffic increased from 7.4 mph before the experiment to 15.2 mph after Stage 1, and 16.4 mph after Stage 2. For these same conditions, southbound traffic averaged 12.6 mph, 18.3 mph, and 14.2 mph, respectively. In the south section, outbound traffic (southbound) traveled at an average speed of 12.8 mph before the experiment, 11.5 mph after Stage 1, and 16.7 mph after Stage 2; the off-peak direction (northbound) traveled at 15.7 mph before the experiment, 15.3 mph after Stage 1, and 13.6 mph after Stage 2. The effect of the signal progression on trip time, delay time, and number of stops is given in Tables 10 and 11.

The improvements to traffic flow experienced on McCarter Highway contrast sharply with the lack of improvement experienced on Central Avenue during Experiment B90. This may be attributed to several significant differences. First, on McCarter Highway parking violations are almost nonexistent, whereas on Central Avenue these violations are frequent; second, there are many more unsignalized intersections on Central Avenue than on McCarter Highway. At these unsignalized intersections, crossing and turning traffic from side streets can severely interfere with the organization of platoons and the ability of drivers to maintain the speed of progression.

Experiment B100 investigated the effect of establishing signal progressions for Broad Street, Newark, in the same area where Experiment B78 had investigated the effect of reversible lane operations. The pattern of roadway use resulting from Experiment B78 has been accepted and continued in use by the city. Therefore, Experiment B100 was conducted with the reversible lanes in operation. The composite effect of Experiments B78 and B100 was a striking improvement in traffic flow on this portion of one of Newark's busiest arterial roadways. However, practically no benefits were realized until the signals were adjusted. Before Experiment B100 was implemented, signals in this area operated simultaneously using a 90-sec cycle. Inbound preferential, outbound preferential, and average offset operations were designed and implemented in two stages. Stage 1 tested the effect of using longer cycles during peak periods and a shorter cycle during midday. In this stage, the inbound preferential progression in use between 7 and 9 AM operated on a 100-sec cycle, with a design speed of progression of 25 mph and a green band of 56 percent of the cycle. The outbound preferential offset plan was placed in operation between 4 to 6 PM, using a 110-sec cycle with a 48-percent green band also operating at a design speed of 25 mph. During midday, the average offset plan used a 30-mph design speed, a 70-sec cycle, and attained green bands of 26 percent northbound and 22 percent southbound. Stage 2 of this experiment reverted to a common

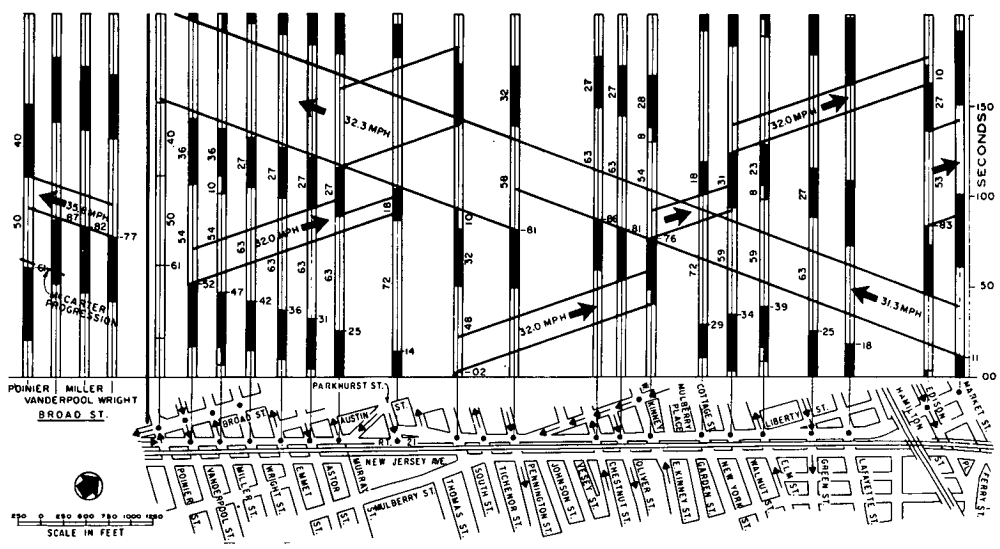
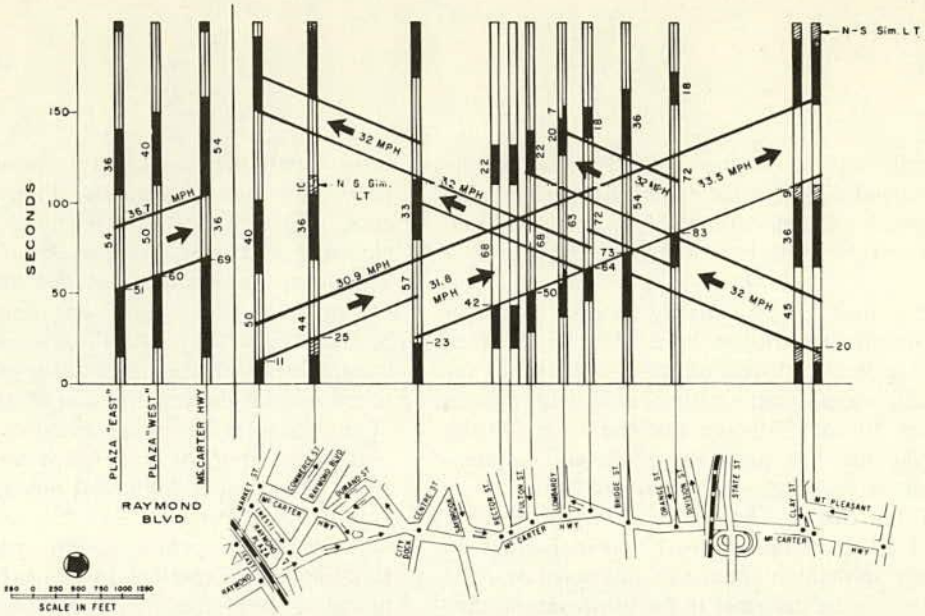


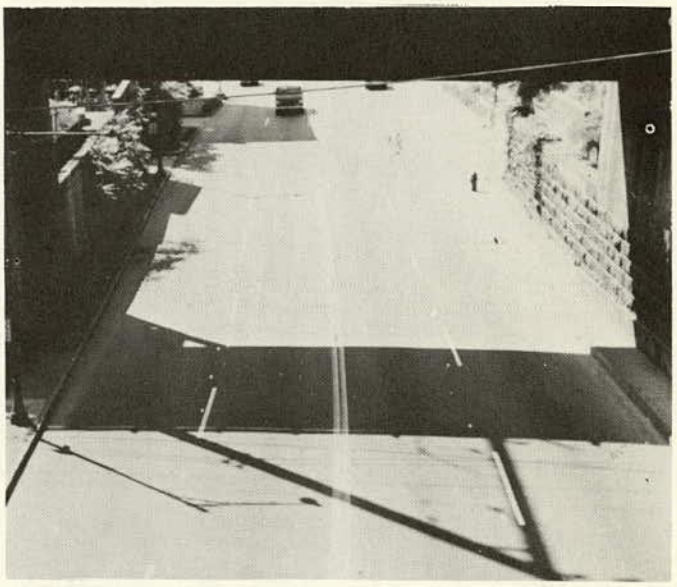
Figure 33. Experiment B93, outbound offset, Segment A.



Looking North From Stickle Bridge

Looking North at Bridge Street

Looking South at Lombardy Place



Looking South—under Erie-Lackawanna Bridge

Figure 34. Experiment B93, outbound offset, Segment B.

TABLE 10

EXPERIMENT B93, SUMMARY OF ANALYSIS, NORTHBOUND, PM TIME PERIOD

DATA GROUP	NORTH SECTION NO. OF RUNS	TRAVEL TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	SOUTH SECTION NO. OF RUNS	TRAVEL TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS
"Before"	22	310.0	120.5	3.7	22	456.9	237.5	7.1
"After" average	11	318.7	84.2	4.2	9	223.1 [a]	63.2 [a]	2.8 [a]
Net change		+8.7	−36.3	+0.5		−233.8	−174.3	−4.3
Percent change		+2.8	−30.1	+13.5		−51.2	−73.4	−60.6
Significant (α=0.05)		No	Yes	No		Yes	Yes	Yes
"After" outbound	16	358.3	131.3	5.6	19	206.8	48.9	1.9
Net change (from average)		+39.6	+47.1	+1.4		−16.3	−14.3	−0.9
Percent change		+12.4	+55.9	+33.3		−7.3	−22.6	−32.1
Significant (α=0.05)		Yes	Yes	Yes		No	No	No
Net change (from "before")		+48.3	+10.8	+1.9		−250.1	−188.6	−5.2
Percent change		+15.6	+9.0	+51.4		−54.7	−79.4	−73.2
Significant (α=0.05)		Yes	No	Yes		Yes	Yes	Yes

[a] Variability reduced.

TABLE 11

EXPERIMENT B93, SUMMARY OF ANALYSIS, SOUTHBOUND, PM TIME PERIOD

DATA GROUP	SOUTH SECTION NO. OF RUNS	TRAVEL TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	NORTH SECTION NO. OF RUNS	TRAVEL TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS
"Before"	22	395.5	153.0	4.9	24	269.2	99.5	3.3
"After" average	18	441.7	163.3	6.2	18	185.2	41.1	2.0
Net change		+46.2	+10.3	+1.3		−84.0	−58.4	−1.3
Percent change		+11.7	+6.7	+26.5		−31.2	−58.7	−39.4
Significant (α=0.05)		No	No	Yes		Yes	Yes	Yes
"After" outbound	17	303.2	66.8	2.5	22	238.9	91.0	3.4
Net change (from average)		−138.5	−96.5	−3.7		+53.7	+49.9	+1.4
Percent change		−31.4	−59.0	−59.7		+29.0	+121.4	+70.0
Significant (α=0.05)		Yes	Yes	Yes		Yes	Yes	Yes
Net change (from "before")		−92.3	−86.2	−2.4		−30.3	−8.5	+0.1
Percent change		−23.3	−56.3	−49.0		−11.3	−8.5	+3.0
Significant (α=0.05)		Yes	Yes	Yes		Yes	No	No

90-sec cycle with inbound preferential, outbound preferential, and average design speeds of progression of 29.2, 30.5 and 30 mph, respectively. Green bands were 56 percent during morning peak periods, 48 percent during afternoon peak periods, 26 percent northbound, and 22 percent southbound during midday. Changes in trip time, delay time, and number of stops, as well as vehicles through (total) and the number of vehicles stopped at various checkpoint intersections, are given in Tables 12 and 13. Average trip time was reduced approximately 21 percent and delay time was reduced approximately 56 percent for the morning peak hour. During the afternoon peak hour, northbound traffic experienced even greater improvement, with trip time being reduced by approximately 38 percent and delay time being reduced by 69 percent.

Experiment E40 investigated the effects of adjustments to signal progressions for Oak Street in Louisville. Travel time surveys had revealed that morning and midday travel speeds were fairly consistent at approximately 20 mph. However, during the PM peak period, travel speed reduced to approximately 16 mph, with delay time averaging approximately 32 percent. The primary purpose of this experiment was to improve conditions during this critical period. Oak Street is a one-way eastbound roadway be-

TABLE 12

EXPERIMENT B100, SUMMARY OF ANALYSIS

CONDITION	NORTHBOUND TRAFFIC: SPEED AND DELAY: TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	VEHICLES PER HOUR: ORANGE STREET: THROUGH	ORANGE STREET: STOP	PARK PLACE: THROUGH	PARK PLACE: STOP	CENTRAL AVENUE: THROUGH	CENTRAL AVENUE: STOP
(*a*) AM time period									
"Before"	184.0	70.3	2.3	1026.7	480.7	224.7	171.0	634.0	179.3
"After" 25 mph	205.0	91.7	3.0	1074.7	324.3	238.8	87.0	692.3	202.0
Net change	+21.0	+21.4	+0.7	+48.0	−156.4	+13.6	−84.0	+58.3	+22.7
Percent change	+11.4	+30.4	+30.4	+4.7	−32.5	+6.1	−49.1	+9.2	+12.7
Sig. level	NS	0.05	0.05	NS	0.005	NS	0.005	NS	NS
"After" 30 mph	189.2	73.9	2.8						
Net change (25 to 30)	−15.8	−17.8	−0.2						
Percent change	−7.7	−19.4	−6.7						
Sig. level	NS	0.05	NS						
Net change ("before" to 30)	+5.2	+3.6	+0.5						
Percent change	+2.8	+5.1	+21.7						
Sig. level	NS	NS	0.05						
(*b*) Midday time period									
"Before"	196.0	68.8	2.7						
"After" 30 mph (70-sec cycle)	178.7	62.8	2.7						
Net change	−17.3	−6.0	0.0						
Percent change	−8.8	−8.7	0.0						
(*c*) PM time period									
"Before"	118.5	53.4	1.9	2753.0	1453.5	838.3		1244.3	287.3
"After" 25 mph	74.0	16.7	0.8	2911.0	782.5	888.3		1224.0	376.0
Net change	−44.5	−36.7	−1.1	+158.0	−671.0	+50.0		−20.3	+88.7
Percent change	−37.6	−68.7	−57.9	+5.7	−46.2	+6.0		−1.6	+30.9
Sig. level	0.0005	0.0005	0.005	0.05	0.005	NS		NS	0.005

NS = not significant at $\alpha = 0.10$.

tween Eighth Street and Logan Street, the area in which this experiment was conducted. The street width varies between 36 and 42 ft from Eighth Street to Shelby Street and is 26 ft in width from Shelby Street to Logan Street.

There are three travel lanes between Eighth Street and Shelby Street, with a fourth lane between Third Street and Second Street. The 26-ft-wide section from Shelby Street to Logan Street is divided for two travel lanes. From 4 to 6 PM, because parking is permitted, the block between Eighth and Seventh Streets is restricted to only two travel lanes. There are 15 signals on this section of roadway, which is approximately 7,300 ft long. Traffic volumes during the AM peak hour average 900 vehicles west of the I-65 on-ramp and 350 vehicles east of the ramp. During the PM peak hour there were 1,300 vehicles on Oak Street west of the I-65 on-ramp and 900 vehicles east of this ramp. Signals on Oak Street are part of a two-dial Trafflex system that is in operation south of Broadway. The signals operated on a 60-sec cycle both in the "before" and "after" periods of the experiment. Dial 1 controlled the signals during the entire day, with the exception of the period from 2:30 to 6 PM, which was controlled by Dial 2. Figures 35 and 36 show the time-space diagrams for "before" conditions. The work of establishing the revised signal settings was implemented in two phases. Phase 1 was the direct implementation of the time-space diagrams developed from survey information, whereas Phase 2 resulted from field adjustment of the time-space diagrams developed for Phase 1. Figures 37 and 38 show the ultimate development of signal settings for Phase 2.

Traffic counts conducted during the experiment indicated no significant change of volumes during any time period. The relationship of "before" and "after" average vehicle speeds, number of stops, total delay time, and delay time per stop is compared in Figure 39. Delay time experienced between checkpoints on Oak Street is given in Table 14. No significant changes occurred during AM or midday periods. During the PM peak period, a slight increase in average vehicle speed, reduction in delay time, decrease in number of stops, and increase in delay per stop were experienced, although these were not statistically significant at the $\alpha = 0.05$ level.

Although the Oak Street experiment has produced little improvement in traffic flows, it has illustrated the diminish-

TABLE 13

EXPERIMENT B100, SUMMARY OF ANALYSIS

CONDITION	SOUTHBOUND TRAFFIC: SPEED AND DELAY: TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	VEHICLES PER HOUR: ORANGE STREET: THROUGH	ORANGE STREET: STOP	CENTRAL AVENUE: THROUGH	CENTRAL AVENUE: STOP
(*a*) AM time period							
"Before"	158.6	53.9	1.8	1946.7	1007.3	1777.7	478.0
"After"25 mph	132.4	26.2	1.2	2051.0	909.7	2221.7	169.3
Net change	−26.2	−27.7	−0.6	+104.3	−97.6	+444.0	−308.7
Percent change	−16.5	−51.4	−33.3	+5.4	−9.7	+25.0	−64.6
Sig. level	0.05	0.05	0.05	0.10		0.005	0.005
"After" 30 mph	125.4	23.5	0.9				
Net change (25 to 30)	−7.0	−2.7	−0.3				
Percent change	−5.3	−10.3	−25.0				
Sig. level	NS	NS	NS				
Net change ("before" to 30)	−33.2	−30.4	−0.9				
Percent change	−20.9	−56.4	−50.0				
Sig. level	0.05	0.05	0.05				
(*b*) Midday time period							
"Before"	176.6	48.0	2.1				
"After" 30 mph (70-sec cycle)	163.6	29.8	1.8				
Net change	−13.0	−18.2	−0.3				
Percent change	−7.4	−37.9	−14.3				
(*c*) PM time period							
"Before"	106.9	46.3	1.4	755.0	662.5	1229.5	443.0
"After" 25 mph	125.9	62.2	1.7	681.0	439.5	1119.0	449.0
Net change	+19.0	+15.9	+0.3	−74.0	−223.0	−110.5	+6.0
Percent change	+17.8	+34.3	+21.4	−9.8	−33.7	−9.0	+1.4
Sig. level	0.05	0.025	0.10	0.10	0.005	0.025	NS

NS = not significant at $\alpha = 0.10$.

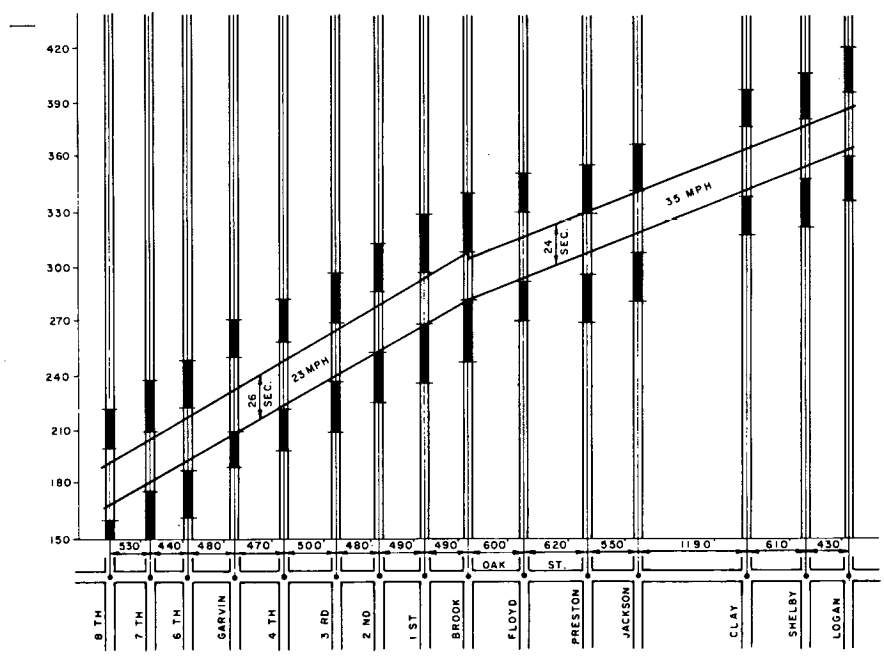


Figure 35. Experiment E40, original settings, Dial 1.

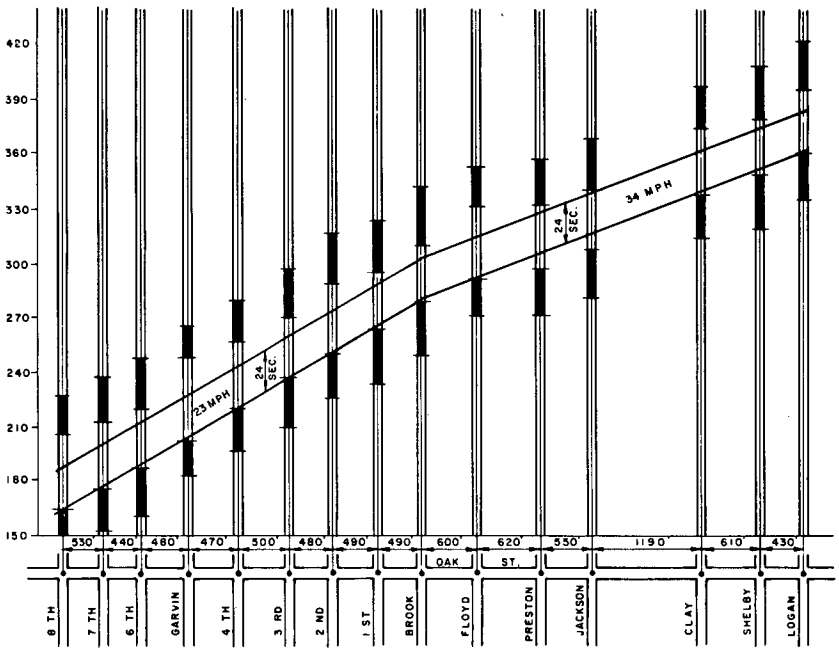


Figure 36. Experiment E40, original settings, Dial 2.

ing returns resulting from fine adjustment to an already well-developed system. The "before" traffic speeds of 16 mph indicated a fairly satisfactory level of operation as compared with results of other experiments such as those conducted on McCarter Highway, Springfield Avenue, and Broad Street in Newark. In these experiments, an average vehicle speed of approximately 15 mph was attained due to the improvements. The composite result of these experiments indicates that an average vehicle speed of 15 mph during peak periods is a reasonable level for congested downtown arterial streets.

Experiment E35 (Figs. 40 and 41), conducted in downtown Louisville, investigated the results of coordinated signal timing of 217 intersections of the downtown area. These intersections are part of four systems that were in operation prior to this experiment. Each of these systems operated independently, with the signals on Broadway using a 90-sec cycle; all other signals used a 60-sec cycle. As part of the preparatory work for this experiment, means were developed to interconnect and coordinate these four systems. Because of equipment limitations, only two dials were available for use. Dial 1 was used to control signals from 6 PM to 2:30 PM; Dial 2 was in use from 2:30 PM to 6 PM, the most critical time period. Travel time information was developed and used to formulate time-space diagrams for the roadways involved in this experiment. Time-space dia-

TABLE 14

EXPERIMENT E40, AVERAGE DELAY TIME PER VEHICLE BETWEEN CHECKPOINTS

		AVERAGE DELAY TIME (SEC), BY CROSS STREET														
TIME PERIOD	DATA GROUP	SEVENTH	SIXTH	GARVIN	FOURTH	THIRD	SECOND	FIRST	BROOK	FLOYD	PRESTON	JACKSON	CLAY	SHELBY	LOGAN	TOTAL DELAY TIME (SEC)
AM	1					0.7				7.9	9.2	5.5				23.3
	2	0.3			1.6	0.2		6.8	1.6	4.7	3.8		1.5	1.0		21.5
	3	3.3			1.2	0.2	1.0	3.0		3.3	9.3		2.3	1.3		24.9
Midday	1		0.4					4.5		8.9	4.6	2.7	1.3			22.4
	2			1.3	1.7		0.3	1.5		6.7	1.5		3.6			16.6
	3				1.8	1.5	1.5	4.1	2.4	4.3	6.0		2.7			24.3
PM	1		0.3	3.0	15.0	5.8	12.5	8.3	2.3	14.7	5.3		2.8			70.0
	2	6.5	3.9	19.4	9.8	1.4	5.8	10.2		9.1	3.4	1.2	1.5			72.2
	3		2.5	6.7	8.6	5.4	6.4	8.8		4.2	8.8	1.4	3.2			56.0

grams were designed in three steps. Step one involved the use of the SIGOP program to produce initial settings for this system. Step two involved displaying the SIGOP output on the Analog Traffic Signal Model and making manual adjustments. Step three consisted of plotting the information from the model on graph paper and fine-tuning the system. Cycle lengths used in developing the signal settings recognize the controls imposed by traffic volumes and pedestrian crossing time of Broadway. The long cycle required for Broadway was considered unacceptable for general use in the downtown area; thus, it was decided to use a common cycle length for the areas north and south

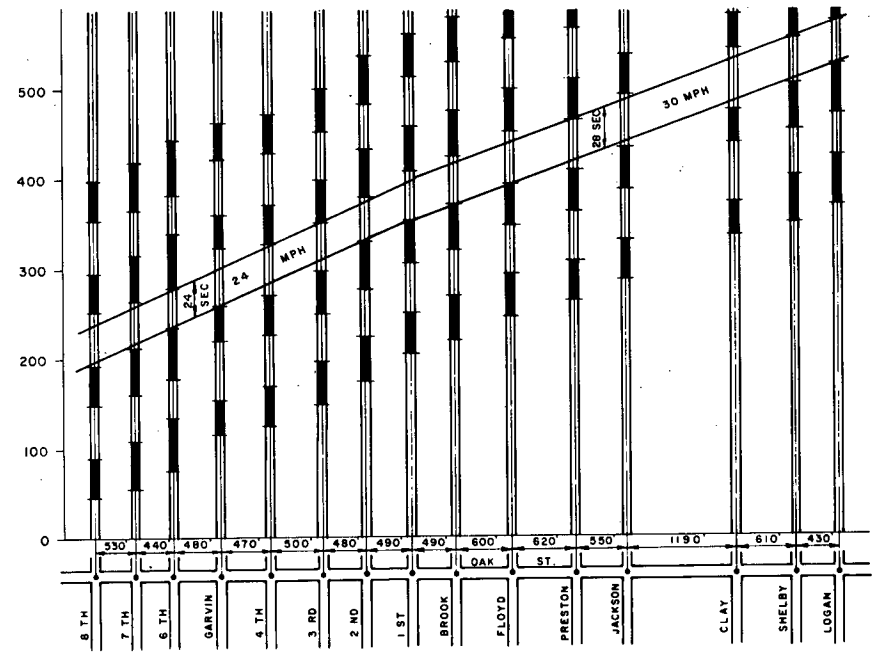


Figure 37. Experiment E40, Phase II settings, Dial 1.

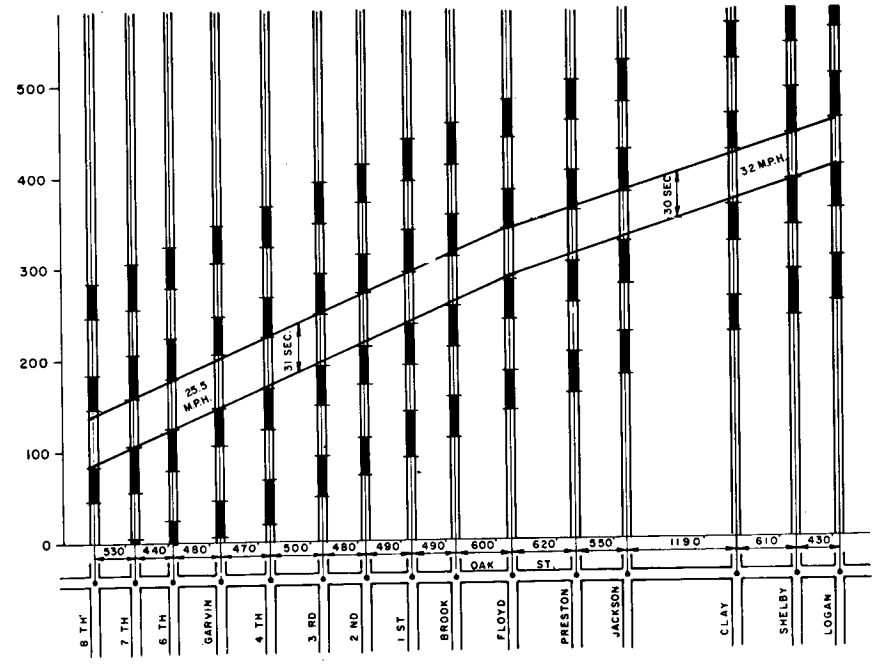


Figure 38. Experiment E40, Phase II settings, Dial 2.

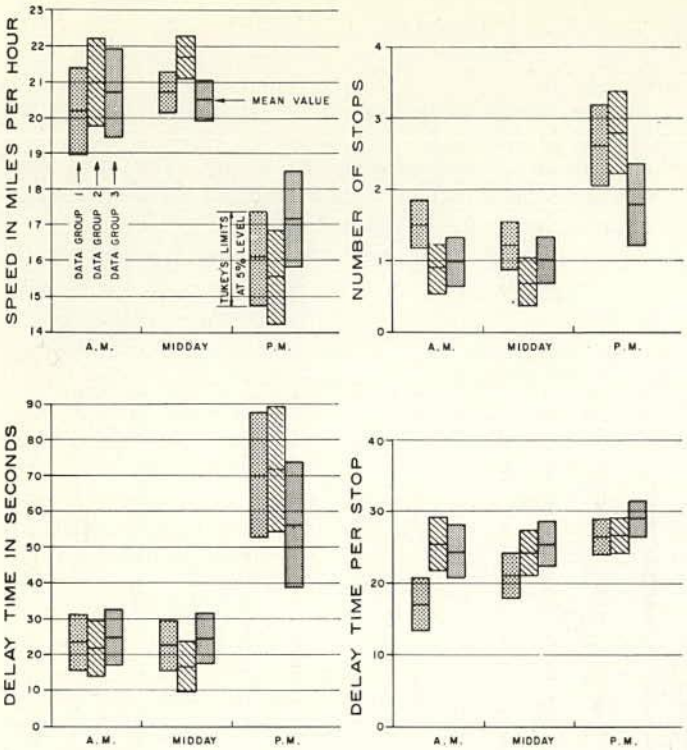


Figure 39. Experiment E40, analysis of speed and delay data.

of Broadway and to use double the cycle length for Broadway. Cycle lengths of 50 and 100 sec were used for average conditions; cycle lengths of 60 and 120 sec were used for the afternoon peak period. The new phasings and offsets were installed and adjusted during the period between May 24 and July 11, 1969.

Based on the analysis of data from the travel time survey of all arterials, it can be concluded that:

1. During the AM time period, average speed decreased by 0.5 mph (2.6 percent); delay time decreased by 3.4 sec per vehicle (6.7 percent); number of stops increased by 0.2 per vehicle (9.5 percent); delay time per stop decreased by 4.4 sec per vehicle (17.9 percent); and the number of stops per mile increased by 0.2 (14.3 percent). In terms of

Broadway Looking East From 9th Street

Broadway Looking West From 9th Street

Figure 40. Experiment E35, Broadway.

4th Street Looking South From Walnut Street

4th Street Looking South From Liberty Street

Figure 41. Experiment E35, Fourth Street.

total travel during the AM peak hour, delay time decreased by 30.7 veh-hr; vehicle stops increased by 2,200; and vehicle stops per mile increased by 715.

2. During the midday time period, the number of stops increased by 0.2 per vehicle (9.5 percent); delay time per stop decreased by 3.1 sec per vehicle (13 percent); and stops per mile increased by 0.1 per vehicle (6.7 percent).

3. During the PM time period, speed was reduced by 0.6 mph (3.6 percent); number of stops increased by 0.4 per vehicle (12.9 percent); delay time per stop decreased by 1.4 sec per vehicle (5.3 percent); and stops per mile increased by 0.3 per vehicle (14.3 percent). In terms of total travel during the PM peak hour, delay time increased by 7.8 veh-hr; vehicle stops increased by 4,915; and vehicle stops per mile increased by 1,415.

4. The variability of the "before" and "after" measurements for delay time per stop decreased significantly for all arterials during the AM time period.

The increase in number of stops and decrease in delay per stop are natural results of the decreased cycle length for all streets except Broadway, exclusive of the PM time period. Conversely, analysis of the increase in cycle length on Broadway during the PM time period indicated that the number of stops was decreased and delay time per stop was increased.

Inclement Weather Effects

Two experiments were conducted, one in each study area, to assess the effect of inclement weather on travel time, delay time, and number of stops for trips on two roadways —Liberty and Walnut Streets in Louisville and McCarter Highway in Newark.

Experiment E69 studied these conditions on the sections of Liberty and Walnut Streets between Ninth and Chestnut Streets (Fig. 42). Average weekday volumes on Walnut Street were approximately 6,000 vehicles between Chestnut and Floyd Streets and 8,000 to 10,000 vehicles between Floyd and Ninth Streets. On Liberty Street, average weekday volumes were approximately 12,000 vehicles between Ninth Street and the I-65 on-ramp east of Floyd Street. From the I-65 on-ramp to Chestnut Street, average weekday traffic volumes are about 5,000 vehicles. Parking regulations, signal settings, and other traffic regulations remained constant during the period of this study. Comparison of clear weather and inclement weather conditions was made through travel time studies and intersection counts conducted at the Walnut Street approach to Fourth Street and the Liberty Street approach to Third Street. Travel time studies made by a test car injected into the traffic stream indicate the experience of an average vehicle traversing the entire roadway; the intersection surveys indicate the experience of all vehicles at a given location. The experience of the travel time survey is given in Table 15. There were no significant differences in travel time, delay time, and number of stops for trips made during clear and inclement weather as determined from speed and delay data. The number of vehicles through per cycle determined from the intersection survey did not change at the two approaches studied. Mean values and variances for vehicles stopped were greater for inclement weather at Walnut and Fourth Streets during the AM and midday periods, and at Liberty

TABLE 15

EXPERIMENT E69, SUMMARY OF SPEED AND DELAY ANALYSIS

STREET	TIME PERIOD	VARIABLE	MEAN VALUE		VARIANCE		NO. OF OBS.	
			CLEAR	RAIN	CLEAR	RAIN	CLEAR	RAIN
Walnut	AM	Travel time (sec)	416.7	436.3	4955.7	4538.4	27	20
		Delay time (sec)	84.4	87.1	2130.2	1707.7		
		No. of stops	2.8	3.2	1.7	2.6		
	Midday	Travel time (sec)	403.6	408.2	2093.1	2848.4	17	17
		Delay time (sec)	69.4	65.1	883.3	1100.0		
		No. of stops	2.5	2.4	1.0	1.1		
	PM	Travel time (sec)	443.9	430.2	8633.6	21700.6	29	6
		Delay time (sec)	93.7	81.8	4256.2	7425.4		
		No. of stops	3.4	3.2	4.8	10.6		
Liberty	AM	Travel time (sec)	326.9	339.6	1902.6	1954.6	63	19
		Delay time (sec)	47.2	48.4	1181.4	786.9		
		No. of stops	1.5	1.4	0.8	0.6		
	Midday	Travel time (sec)	352.8	351.0	4435.8	4270.0	64	12
		Delay time (sec)	61.0	55.2	2185.0	1961.4		
		No. of stops	2.0	1.7	2.1	1.5		
	PM	Travel time (sec)	398.6	375.5	5156.0	7028.3	78	6
		Delay time (sec)	85.2	64.0	2354.5	1943.2		
		No. of stops	3.2	2.5	2.8	5.1		

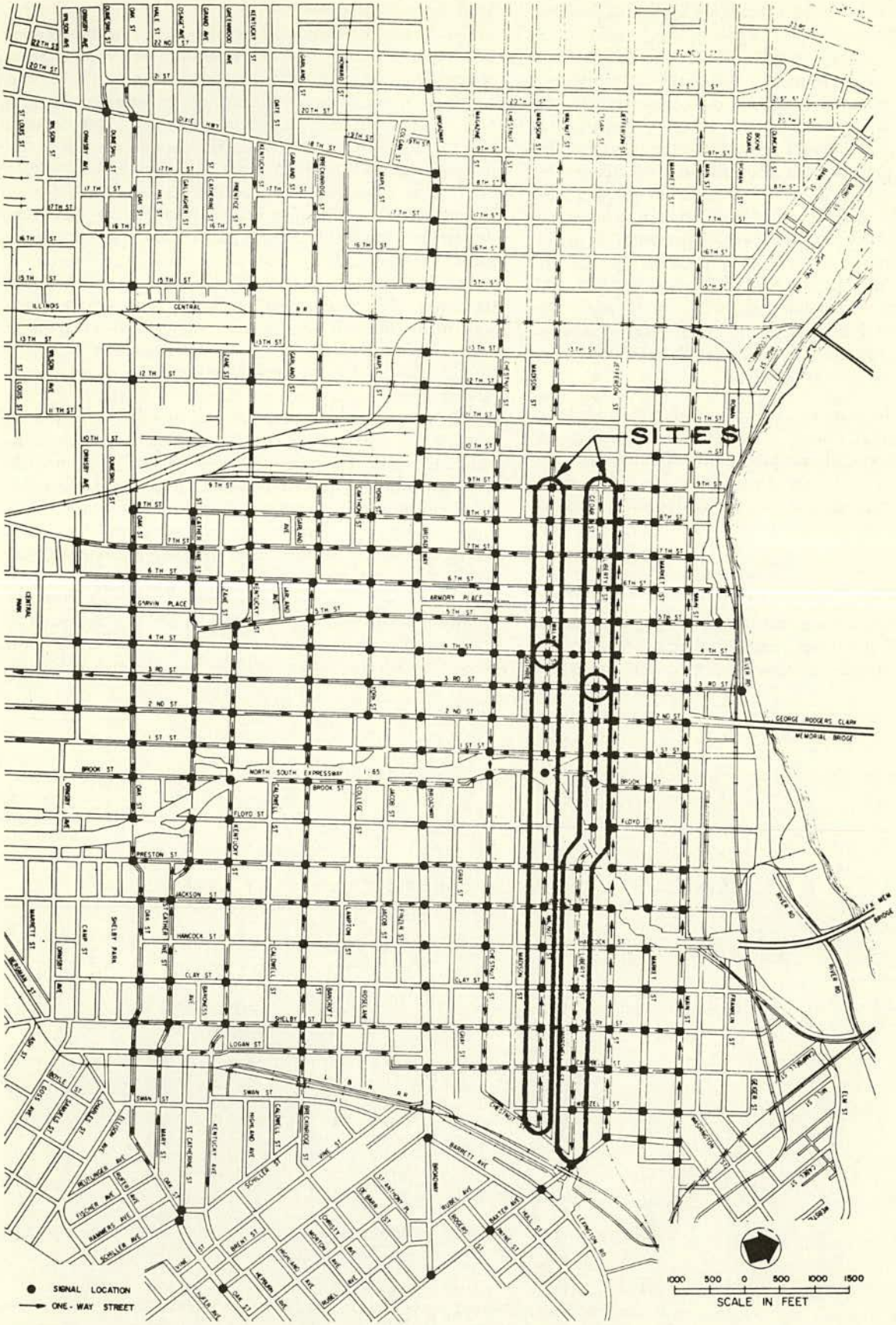


Figure 42. Experiment E69, location map.

and Third Streets during the midday and PM periods. It should be noted that the AM period on Walnut Street and PM period on Liberty Street are the intervals of peak traffic flow. The experience of this survey indicates that the effect of the inclement weather is to interfere with the cohesive and compact platooning of traffic. Indications are that a larger number of drivers than usual tend to drive slowly, not maintaining the designed speed of progression, and are, therefore, cut off at the end of the green band, increasing the incidence of stops.

Experiment B99 conducted on McCarter Highway in Newark developed further information on the effects of inclement weather. Figure 43 shows the location of this section of McCarter Highway. Volumes of traffic were monitored by use of automatic traffic-recording equipment and showed no significant change during the period of this experiment. General traffic conditions, traffic regulations, and signal timing remained constant during the experiment. Comparisons were made for the morning peak period only, because data describing inclement weather conditions for other periods were insufficient for analysis. These comparisons were made through analysis of speed and delay data obtained by a test vehicle traveling in the stream of traffic. A summary of the measurements developed in the travel time survey is given in Table 16, where a significant increase in all measurements can be seen to occur during inclement weather.

The density of traffic on McCarter Highway is much greater than that experienced on Walnut and Liberty Streets in Louisville. Drivers experience a distinct lack of freedom of movement and are controlled by the slower vehicles of the traffic stream. Under these conditions, it can be seen that the average vehicle experiences a considerable increase in travel time, delay time, and number of stops.

Bus Operation

Five experiments were conducted to develop data for the optimization of bus transit use in the downtown area. The total travel time used by a bus in performing its service is divided into three elements: bus stop operations, passenger service operations, and route time between bus stops.

Bus stop operations include all time required for maneuvers into and out of the bus stop, including signal delay that may be encountered during this interval. It does not include time required for boarding and alighting of passengers. This element is called passenger service operations. Route time between bus stops includes all elements of travel between the limits established for measurement of bus stop operations. Bus stop operations and passenger service operations were investigated in a series of experiments conducted in Louisville.

Experiment F63, the investigation of passenger service operations, established the relationship between time required for passenger service and number of passengers boarding and alighting. In the initial experiment a fare of $0.30, without zone limitation and with free transfer privileges, was charged. This was a "cash and change" system, with payment on entering. Passengers must board at the front of the bus but could alight using either front or rear door. During a later period of the project, when the Louisville Transit Company had changed to an "exact fare" system, measurements were repeated to evaluate this system and to make possible comparison of the two fare collection systems. Analysis of these data resulted in the development of regression equations for prediction of passenger service time under various conditions. For the "cash and change" system, if Y equals expected passenger service time (seconds), X_1 equals the number of passengers alighting, and X_2 equals the number of passengers boarding, then for conditions where passengers are:

alighting only,

$$Y = 1.8437 + 1.1122X_1 \quad (1)$$

boarding only,

$$Y = -0.0855 + 2.5855X_2 \quad (2)$$

both boarding and alighting,

$$Y = 1.7701 + 0.9727X_1 + 2.2756X_2 - 0.02338X_1X_2 \quad (3)$$

Graphs of these equations appear in Chapter Four. Similar equations were developed for the exact fare system (*2*); the comparison of these systems is shown in Figure 44.

Inspection of these equations shows that the intercept values, which can be considered additional time required for beginning and ending passenger service, average about

TABLE 16

EXPERIMENT B99, TRAVEL CHARACTERISTICS, SPEED AND DELAY DATA

		AM PEAK PERIOD					
		DRY		WET			SIG. @ α LEVEL INDICATED
DIRECTION	VARIABLE	MEAN	OBS.	MEAN	OBS.	DIFFERENCE	
NB	Travel time (sec)	615.8	18	687.7	10	+71.9 (11.7%)	0.10
	Delay time (sec)	222.5	17	249.8	10	+27.3 (12.3%)	0.15
	No. of stops	7.2	17	8.3	10	+1.1 (15.3%)	0.10
SB	Travel time (sec)	595.1	16	706.7	10	+111.6 (18.8%)	0.025
	Delay time (sec)	212.8	16	263.8	10	+51.0 (24.0%)	0.025
	No. of stops	6.8	16	8.6	10	+1.8 (26.5%)	0.025

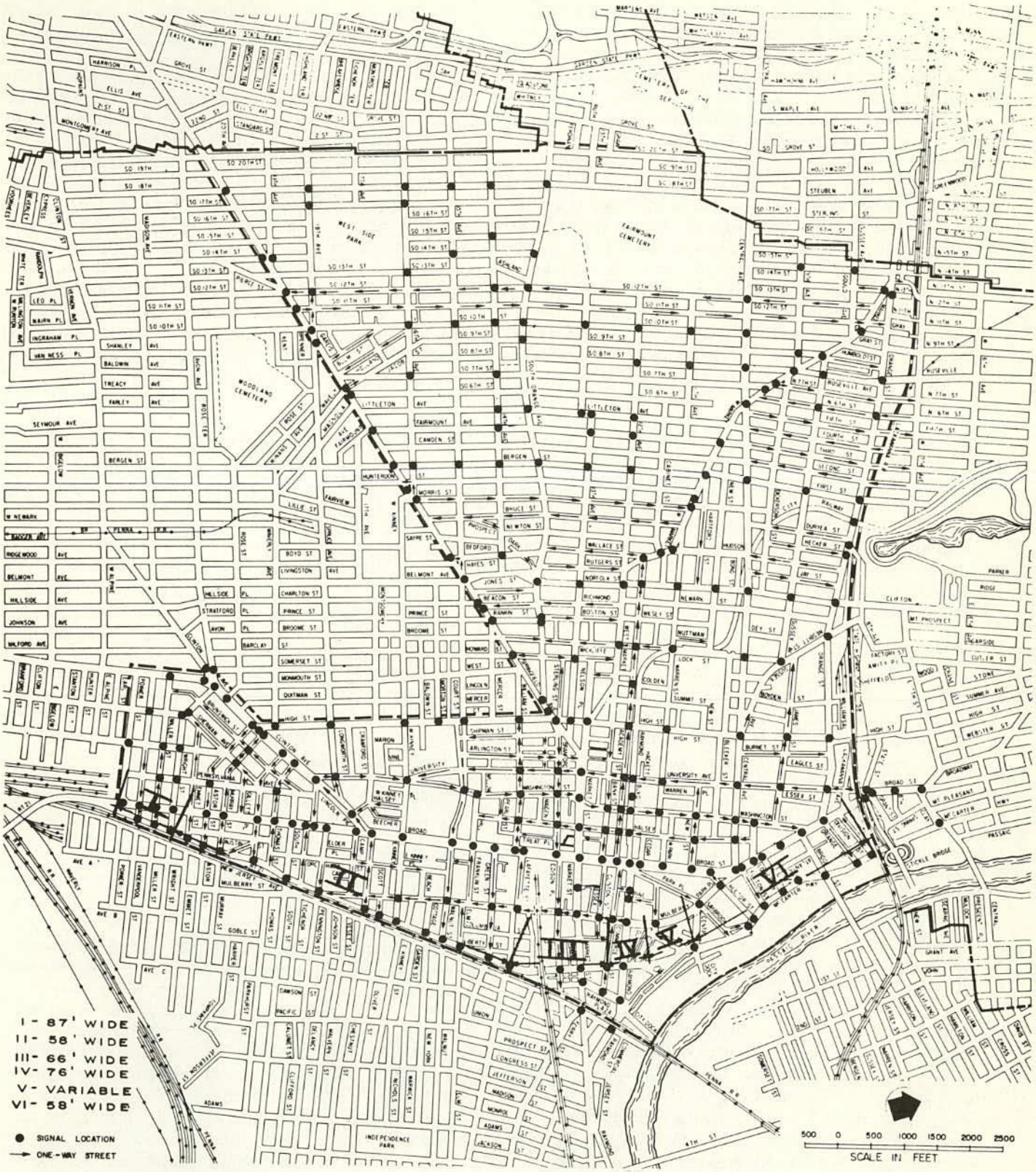


Figure 43. Experiment B99, vicinity map.

1.3 sec. This interval depends on conditions established during the surveys and may vary somewhat due to minute differences in the judgment of survey personnel. In addition, alighting requires 1.1 sec per passenger; and "cash and change" boarding, 2.6 sec per passenger. The "exact fare" system saves approximately 0.6 second for each boarding passenger. For actual scheduling purposes, it may be desirable to use some value between the mean and upper limit shown on the graphs rather than the means that have been used in developing these approximations.

Experiment F64 investigated the relative efficiency of near-side and far-side bus stops, considering parking conditions adjacent to the bus stop and direction of cross-street traffic flow. Eight survey locations in Louisville were selected, each one of which represented one of the possible combinations of these variables. Surveys measured time required for bus stop operations within common limits established as 155 ft before the near crosswalk and 120 ft from the far crosswalk. Analysis of the results of 168 individual observations of bus stop operations indicates that bus stop operation time is always greater at near-side stops than at far-side stops, although the difference was found in some cases not to be statistically significant. Observations of signal delay indicated more frequent occurrence of such delay at near-side stop locations. Therefore, separate analyses were made of those instances when signal delay did and did not occur. It was found that when signal delay occurred, total delay experienced (including passenger service) was greater at far-side stop locations. This is logical, because at near-side locations a portion of this delay time could be used for passenger service. When no signal delay occurred, total delay experienced was greater at near-side stop locations when cross-street traffic flow was from left to right. This reflects the interference of right-turning vehicles with bus movement. When parking is permitted, the detrimental influence of right-turning vehicles is greater than when parking is prohibited. Bus stop operation time for near-side stops was 24.1 percent greater than for far-side stops with parking permitted, and only 10.1 percent greater with parking prohibited.

Conferences with public officials and the bus operators revealed two areas at which the study of bus stop locations could be continued through actual relocation of the stop from a near-side to far-side position.

Experiments F51 and F53, conducted at the intersections of Broadway and Campbell Streets and Fourth and Liberty Streets, respectively, in Louisville, were the two sites. Both of the streets on which the bus stops were located, Broadway and Fourth Streets, are two-way streets, and both of the cross streets, Campbell and Liberty Streets, are one-way with traffic moving from left to right.

In implementing these experiments it was found that the change at Broadway and Campbell Street (Experiment F51) made no significant difference in bus stop operations. This was probably due to the very small volume of right-turning traffic at this location. At Fourth and Liberty Streets (Experiment F53), the findings of Experiment F64 relative to these conditions were substantially confirmed. Bus stop operation time was found to be significantly less when signal delay was not encountered at the far side and

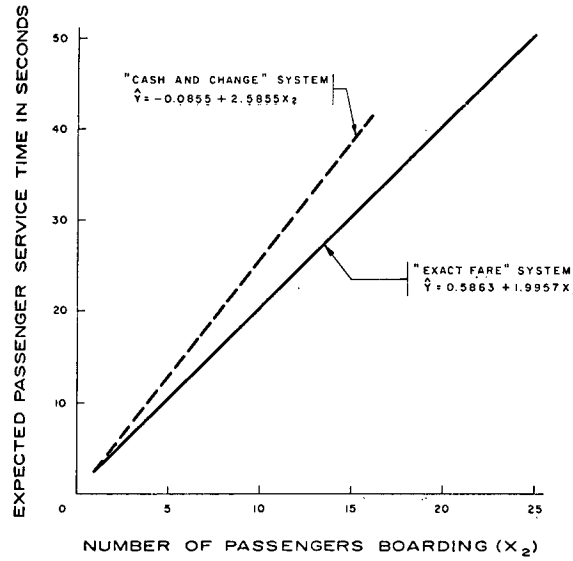


Figure 44. Experiment F63, regression plots.

slightly less at the near side when signal delay was encountered. At this location the effect of right-turning vehicles was apparent, the benefits of far-side operations being less during time periods when right-turning volumes were lower.

Experiment C110, Market Street Bus Operation, investigated organization of bus stops in the heart of the downtown area during the PM peak period—a time of intense bus activity. Since 1956 the city of Newark had permitted buses to load from two lanes on Market Street westbound at the intersections of Broad Street, Halsey Street, and Washington Street. This required passengers waiting to board buses in the second lane to stand in the street, separated from moving traffic only by movable barriers. The 19-ft width of the curb lane was sufficient so that a portion of this width could be allocated to storage of bus patrons. Experiment C110 eliminated the use of the second lane for bus loading. Initially the curb lane and second lane were reserved for bus use. However, after the experiment had been in operation for several weeks, the city traffic engineers decided that exclusive bus use of the second lane was not warranted. Except for small areas at crosswalks, newsstands, and alleys, the entire curb lane of this 3½-block section of Market Street was used for bus loading. Surveys were conducted to count the number of buses during the peak afternoon period to determine the time each bus spent in the bus stop, the number of passengers boarding and alighting, and the time used in passage through this 3½-block area. Studies were made to determine the "community of interest" of each bus route so that routes serving the same area could be assigned to the same stop locations. Each bus was allocated at least two stops within the 3½-block section. Advance publicity was given to this change; bus drivers were informed by their supervisors; signs were erected listing the bus routes using each bus stop; and stop bars were painted on the pavement to indicate the proper position of the front of the bus to the operator. Prior to

implementation of the experiment, tests had been conducted by Public Service Coordinated Transport to determine the minimum area required for bus movement in and out of the bus locations. It was decided that each bus stop should be long enough so that two buses could load simultaneously, with sufficient room between the rear bus of one stop location and the first bus of the adjacent stop location so that this latter bus could maneuver into the second lane when departing. Provision for two buses loading simultaneously in each stop was necessary because of the large number of buses loading in this area. Approximately 200 buses move westbound from the center of Newark from these bus stops during the 2-hr afternoon peak period of 4 to 6 PM. As a result of these modifications it was found that, on the average, bus trip time was reduced by approximately 0.77 min from 5.14 min in the "before" condition to 4.37 in the "after" condition. This includes all time required to traverse the 3½-block study area, including time in the bus stops. Westbound passenger-car trip time during the afternoon peak hour was reduced 20.4 percent (25.5 sec) in passage through this area. In addition, safety was enhanced by the removal of the "in-street" bus loading areas.

SECOND LEVEL ANALYSIS—MEASURING THE EFFECTS OF SYSTEM CHANGES

The general investigations and statistical evaluation of flow data collected during the experimental phase of this project have shown that variations of flow measurements can be defined and that some interrelationships of traffic flow do exist. Specifically, variations of volume and travel time by season of the year and by day of the week can be controlled through proper design of the survey, whereas both volume and time measurements must be stratified by time of day and direction of travel for analysis.

Traffic Characteristics

The standard deviation of directional hourly volume counts for major arterials ranges from 70 to 150 vph, after removing the effects of day of week and time of day. A standard deviation from 2 mph for mean travel speeds of 10 mph to 3 mph for mean travel speeds of 20 mph can be expected for directional test-car travel runs, after eliminating the effect of day of week and time of day.

At signalized intersections within downtown roadway networks, the number of vehicles clearing an approach, cycle by cycle, within an hour period was found to be representable by a Poisson distribution. The cycle-by-cycle variation of the number of vehicles stopped at an approach to a signalized intersection was found to be proportional to the mean number of vehicles stopped.

No direct relationship between speed and volume could be developed for downtown roadways, although a general trend showing a decrease of running speed with an increase of a 15-min rate of flow per lane was observed, after eliminating the effect of signal delay and time of day. A definite relationship between delay time per mile and overall travel speed for major downtown streets and urban arterials with closely spaced traffic signals was established.

The importance of signalized intersections on traffic flow in a downtown network is evidenced by the proportional relationship found to exist between the mean velocity gradient and the travel time per mile for test-car travel runs on several roadways. In fact, a review of the recorded data from test-car runs indicates that although they are numerous in occurrence, delays resulting from mid-block interferences are seldom of sufficient duration to appreciably affect the over-all travel speed along a roadway. A mid-block delay, however, often causes a vehicle to reduce its running speed from that of the speed of signal progression, resulting in an additional intersection delay. This effect is illustrated by the relationship between the number of stops and delay time, both mid-block and at intersections, found to exist from test-car runs. The average duration of stops for 47 test-car observations on several roadways in Louisville was 23.9 sec. A similar comparison for 89 observations in Newark resulted in an average of 29.7 sec per stop. The difference in the delay per stop between the two cities is probably due to the differences of signal cycle lengths. The majority of signals in Louisville have a 60-sec cycle length, whereas all of the Newark signals have a 90-sec cycle length.

No direct relationship between volume and travel speed could be established. Measurements of the average directional volume on each section of roadway, together with the distance and average travel time, make possible reliable estimation of the total vehicle-miles and vehicle-hours of travel, and the average travel speed within the system for network analysis.

A relationship between travel speed, delay time, and number of stops has been developed for roadways within each of the downtown systems studied. In addition, a common relationship between travel speed and delay time exists among roadways of both cities studied (Fig. 45). Travel speed is a measure of the quality of traffic flow that is meaningful to the driving public and can be reliably sampled through test-car travel time studies injected into the stream of traffic. The relationships that have been developed between average travel speed and delay time can be used to develop an estimation of running time, delay time, and average running speed.

Level-of-Service Definition

To understand the basis for the travel time relationships that have been developed, it is necessary to relate them to the well-known speed-flow relationships: "As traffic flow increases, the space mean speed of traffic decreases" (*1*, p. 60). This relationship applies to conditions of uninterrupted flow. Under uninterrupted flow the element that predominates in interfering with a driver's ability to maintain maximum speed is the internal friction of the traffic stream itself. Consideration should be given to what this implies as the environment is changed from the freeway to the downtown street. On the downtown street the internal frictions no longer predominate in influencing the speed that a driver may maintain. The predominating influences have become external, such as traffic signals, pedestrians, and parking maneuvers. This has been demonstrated by the relationships that are developed in Chapters Three and

Five. These investigations show that delay time is a reliable measure of total interference and that percent delay, the delay ratio, is therefore a figure of merit by which the quality characteristic of flow may be described. Use of the delay ratio extends the basic concept of the speed-flow relationship to include total interference to flow, whether internal or external. In using this ratio it is recognized that those minor influences that interfere with flow but do not produce a complete stop are ignored. Since these influences have been found to be of minor importance, this does not significantly affect the ability of the ratio to describe the quality of traffic flow.

A level-of-service definition based on speed-volume/capacity relationships as used in the analysis of highways and rural roads, therefore, is not applicable to roadways of the downtown area. In the discussion of downtown streets in the *Highway Capacity Manual* (*1*, p. 332) it is stated that:

> At present, with the current limited knowledge of the complex relationships that govern downtown traffic flow, it is not possible to develop even typical speed-v/c [volume-capacity] ratio relationships. The capacities of apparently similar downtown streets can vary widely due to differing environmental conditions.

Investigations of this project have revealed that speed-v/c relationships are comparatively meaningless in the analysis of downtown streets. This subject is discussed more fully in Chapter Five. However, a very significant relationship, when considering the efficiency of travel in the downtown area, may be based on delay time experienced. This relationship, based on 990 observations made within the two study areas, is between the average travel speed and delay time. Therefore, if the level-of-service definition is based on a relationship between average speed and the major influence affecting speed, it can be seen that the level-of-service definition developed for downtown roads through consideration of delay time is not drastically different from that which has previously been used on highways and rural roads, using the speed-v/c relationship. Therefore, a travel time survey that includes complete description of the duration and reasons for all delays becomes an essential component of the analysis of downtown streets. The resulting levels of service computed for networks, arterials, or particular locations can set a priority of needs for these areas. Direct reference to the travel time survey will describe the location and cause of repeated delays, which then become focal points for attention in the street optimization program.

In the analysis of the level of service of an intersection, a measurement has been developed that has proved to be extremely useful. This is the number of saturated cycles at a given approach to an intersection. A saturation factor is then defined as the number of saturated cycles divided by the total number of cycles during a specified time period. This factor is similar to the load factor defined in the *Highway Capacity Manual* (*1*, p. 17) as "A ratio of the total number of green signal intervals that are fully utilized by traffic during the peak hour to the total number of green intervals for that approach during the same period." The saturation factor is always equal to or greater than the load factor and may be similarly used. Its advantage is in the ease with which it may be surveyed. The saturation factor is developed through a count of vehicles stopped on red and vehicles through, while the load factor requires considerable judgment on the part of the surveyor to determine use of the approach by traffic.

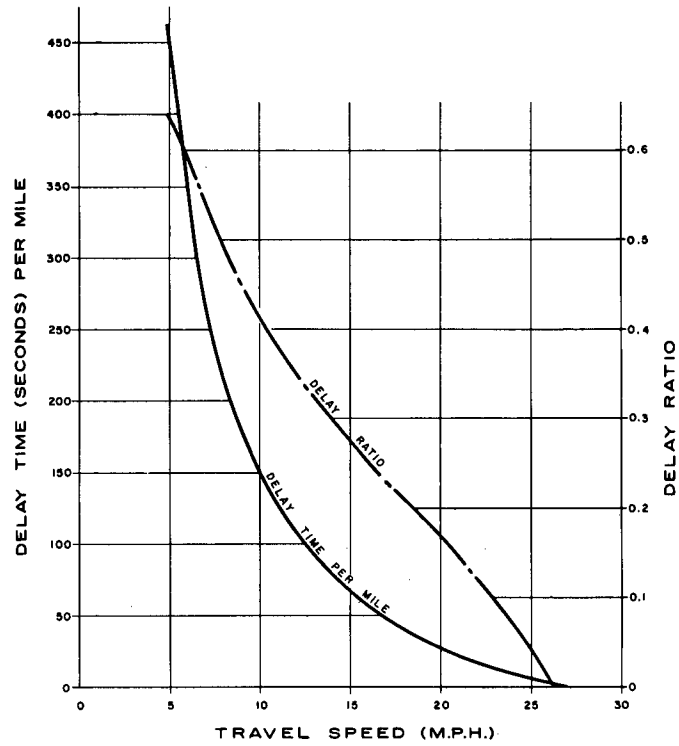


Figure 45. Delay time and delay ratio versus travel speed.

INVESTIGATIONS OF MODELS

As a result of the investigations of various models, it has been found that several existing models are particularly applicable to the problem of analysis of downtown traffic flow. These are Newell's Intersection Delay Model, a Network Assignment Model using the Bureau of Public Roads' traffic assignment system, and the Signal Analog Model for the study of offset relationships between adjacent signals of a network.

Newell's Intersection Delay Model

Newell's Intersection Delay Model (see Appendix B) is used to estimate the delay occurring at an intersection approach, using the surveyed measurements of vehicles stopped on red and vehicles through. The development of delay time at each approach to an intersection makes possible direct evaluation of the allocation of green time to each approach. Also, it may be used for predictive purposes to estimate the change in traffic flows resulting from intersection modifications.

The Network Assignment Model

The Network Assignment Model (see Appendix C) has been found a useful tool for evaluating the larger functional changes that may be made to the street system. It may also be used to evaluate those operational elements affecting arterial flow over extended sections of roadway. It is particularly useful for sensing diversions of traffic that may occur from major changes. In two experiments conducted on this project (Experiment B78, Reversible Lane Operation on Broad Street, Newark; and Experiment B93, Signal Progression on McCarter Highway) these diversions were discernible through field measurement. These two experiments were used in validation tests for the model. In local experiments, minor diversions may have occurred, but their magnitude was not significant compared to other fluctuations in traffic flow. The model offers a stable base not subject to such fluctuations for analysis of these diversions. It also offers the best means of establishing network measurements of total vehicle mileage and total vehicle-hours under conditions in which such diversions are likely to occur.

An additional benefit accruing from development of a Network Assignment Model is the complete organization of related physical and traffic characteristics for the downtown area. This complete inventory is usable in many ways to gain insight into the problems that affect traffic flow. The link data cards become a data bank that may be sorted and investigated for many purposes by electronic data-processing techniques.

The Analog Traffic Signal Model

The Analog Traffic Signal Model is a useful tool to assist the engineer in three-dimensional time-space diagramming in the densely signalized environment of the downtown area. Its prime purpose is to make the entire pattern of offset relationships visible for study and adjustment. Approximate signal offset patterns developed through use of the model must, of course, be reduced to exact data through computations before implementation in the signal system. In the Newark study area, the Analog Traffic Signal Model was used to display existing signal settings developing improved patterns from this base. In Louisville, the model was used to display the output of the SIGOP program (*3*). Adjustments were made, through use of the model, to the program output before time-space diagrams were drawn and exact data were computed.

PUBLIC INTEREST AND REACTIONS

As a result of implementing many improvements on the city streets of the two study areas, many reactions both for and against the work being done were received. In general, it can be stated that the elements of traffic engineering that are well received by the public are those that guide and direct but do not interfere with freedom of use of travel facilities. Such items as traffic signal installation or adjustment, lane striping, installation of lane control signs and markings, and guide and directional signing are all well received and apparently are appreciated by the traveling public. However, regulations such as prohibition of parking or control of loading operations are almost sure to bring a negative public reaction. It is also in these regulatory areas that the problem of enforcement is extremely difficult. Numerous violations of parking regulations and truck loading and pedestrian controls were noted, especially in the Newark study area. The experience of this project indicates that the need for regulation of parking and truck loading should be clearly established and, if they are determined to be necessary, regulations must be strictly enforced. To do less is to promote contempt for traffic engineering management and police enforcement on the part of a large segment of the public.

A very pertinent finding of this program was the need for adequate enforcement to back up the traffic engineering efforts that were often nullified by violation of traffic regulations. The beneficial effects of strict enforcement are described in Experiment A7, where an attempt was made to educate pedestrians to proper recognition of traffic signals at the intersection of Raymond Boulevard and McCarter Highway. Although the effect of violations could not be directly quantified, it obviously had a major effect on traffic flows in many of the experimental areas.

CHAPTER THREE

FINDINGS—TRAVEL TIME STUDIES

Investigations of data collected in both study areas of this research project have developed measurements of traffic flow within urban downtown roadway network systems that are meaningful to the driving public and are practical to survey and analyze. These measurements of relative congestion or quality of flow, discussed in Chapter Five, are expressed in terms of travel time and delay time.

This chapter describes the survey techniques and methods

of analysis used to measure traffic flow within urban network systems and to evaluate the degree of change in network flow resulting from specific traffic operational changes.

TRAVEL TIME SURVEYS

The roadway system to be measured was selected prior to designing travel time surveys. This included defining the general boundaries of the study area and selecting the specific roadways to be incorporated into the network system. In addition, the time periods for measurements were defined. The differences in travel patterns, traffic regulations, signal controls, and driver characteristics that exist during various times of a day, in effect, develop more than one network system within any particular urban area. Three separate systems may be defined by these changes, representing the morning period of heavy inbound traffic, the evening period of heavy outbound traffic, and all other periods of relatively light traffic volumes. One hour was selected as the minimum duration for surveys during each time period to avoid the influence of wide variations of traffic flow found to occur during shorter time periods.

Roadway Volume Counts

Although traffic volume is not a particularly meaningful measurement as far as driver reaction is concerned, volume counts are necessary to quantify traffic flow. Unlike construction of new facilities, traffic operational improvements in urban areas are not normally expected to alter the total volume of traffic within a network system by inducing new trips to or through the area. The trip origins and destinations are also assumed to remain unaffected by traffic operational improvements.

The primary use of volume counts, therefore, was to establish the quantity of flow within the system and to monitor the stability of network flow during experimentation. Volume counts were also used to measure the direct effect of an operational change, such as diverting traffic from one roadway to another.

Counting stations were established at key locations (Figs. 46 and 47) within each study area to provide periodic traffic volume data. Traffic volumes were collected at each key counting station for one full week every month, using pneumatic detectors (road tubes) and 15-min printed-tape automatic traffic recorders. A series of 48-hr road-tube counts and short-term manual counts along roadway sections and at intersection approaches were obtained to develop the average volumes throughout each study area network system. In addition, numerous counts were made before and after experiments for the determination of local effects. Volume flow maps, representing the directional volume passing each section of roadway during an average weekday for the Louisville and the Newark study areas, are shown in Figures 48 and 49. Similar maps for the morning and evening peak hours for the Newark study area are shown in Figures 50 and 51.

Test-Vehicle Surveys

Average travel time, over-all travel speed, total delay time, and total number of stops were selected as the important measures of relative congestion to be used to evaluate network traffic flow. These network measurements were sampled using the test-vehicle method of survey; that is, a test vehicle was driven over selected routes according to the driver's judgment of the average speed of all other vehicles in the traffic stream.

It was found to be impractical to design travel paths that would represent typical trip patterns within the network. The possible travel paths were too numerous and varied to permit selection of several patterns that reliably could represent the entire system. Therefore, "straight" runs along each roadway within the system were used to sample the network travel times. The entire length of roadway within the study area, generally between 0.75 to 2.5 miles, was traveled on each trip to avoid the larger variations of trip time resulting from shorter runs where delay at a single signalized intersection might be a dominant factor.

The date, weather condition, and all unusual events (such as signal malfunctions, roadway repairs, and accidents) were recorded for each test-car run. The time of starting; time of crossing predesignated checkpoints; duration, cause, and location of each delay encountered; and time of finish were also recorded. On the majority of runs, the test car was equipped with the Marbelite Traffic Data Compiler, Model TD-1, which produced a graph having vehicle speed as the ordinate and travel time as the abscissa (Fig. 52). When the Data Compiler was not available, stopwatch readings were recorded manually.

The test-car runs, representing "straight" traffic with no turns, were used for individual roadway "before" and "after" studies. When the total network travel time was being developed, the test-car results were supplemented with intersection measurements to include the delay experienced by turning vehicles. Time-lapse photography proved useful to record the number of vehicles and total travel time for each movement through complex intersections.

Although the test-car travel time surveys proved to be a reliable method of sampling travel time, running time, delay time, and number of stops over a reasonable length of roadway, this technique was found to be impractical to develop data for specific locations or relatively short sections of roadway. Manual observations, using stopwatches or time-lapse photography, were used to obtain a reliable sample size to measure vehicular travel time over relatively short sections of roadway for "before" and "after" studies.

Intersection Counts

The importance of signalized intersections to traffic flow within the study area roadway systems is apparent from a review of the travel time surveys. Depending on the roadway, direction of travel, and time of day, delay time at signalized intersections usually represents between 20 and 50 percent of the total travel time of a trip. The number of stops at signalized intersections often exceeds five per mile—more than for all other causes combined. It is apparent that intersections within an urban street network are most often the locations of critical capacity and therefore the focus of most attention when the problems of these

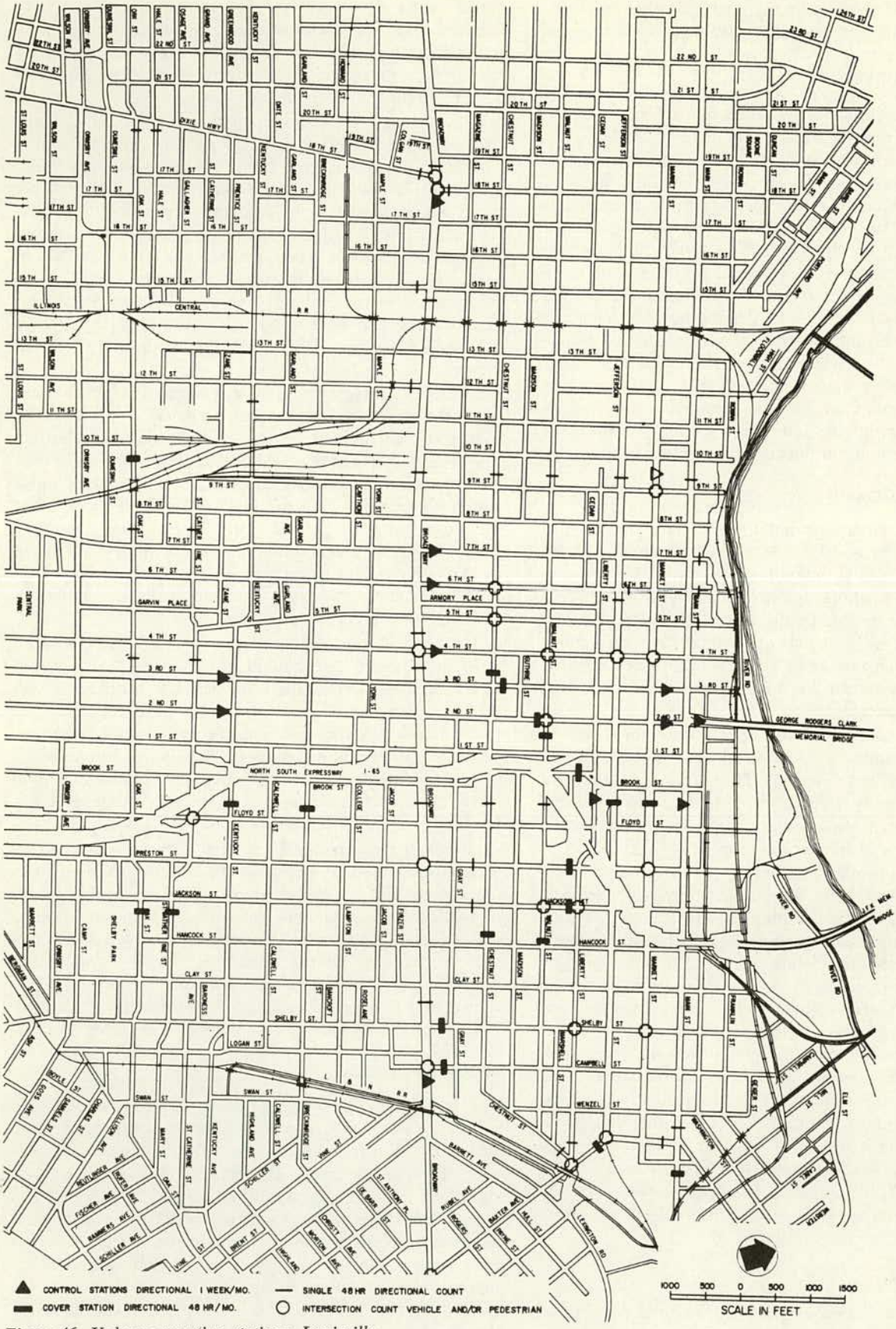


Figure 46. Volume counting stations, Louisville.

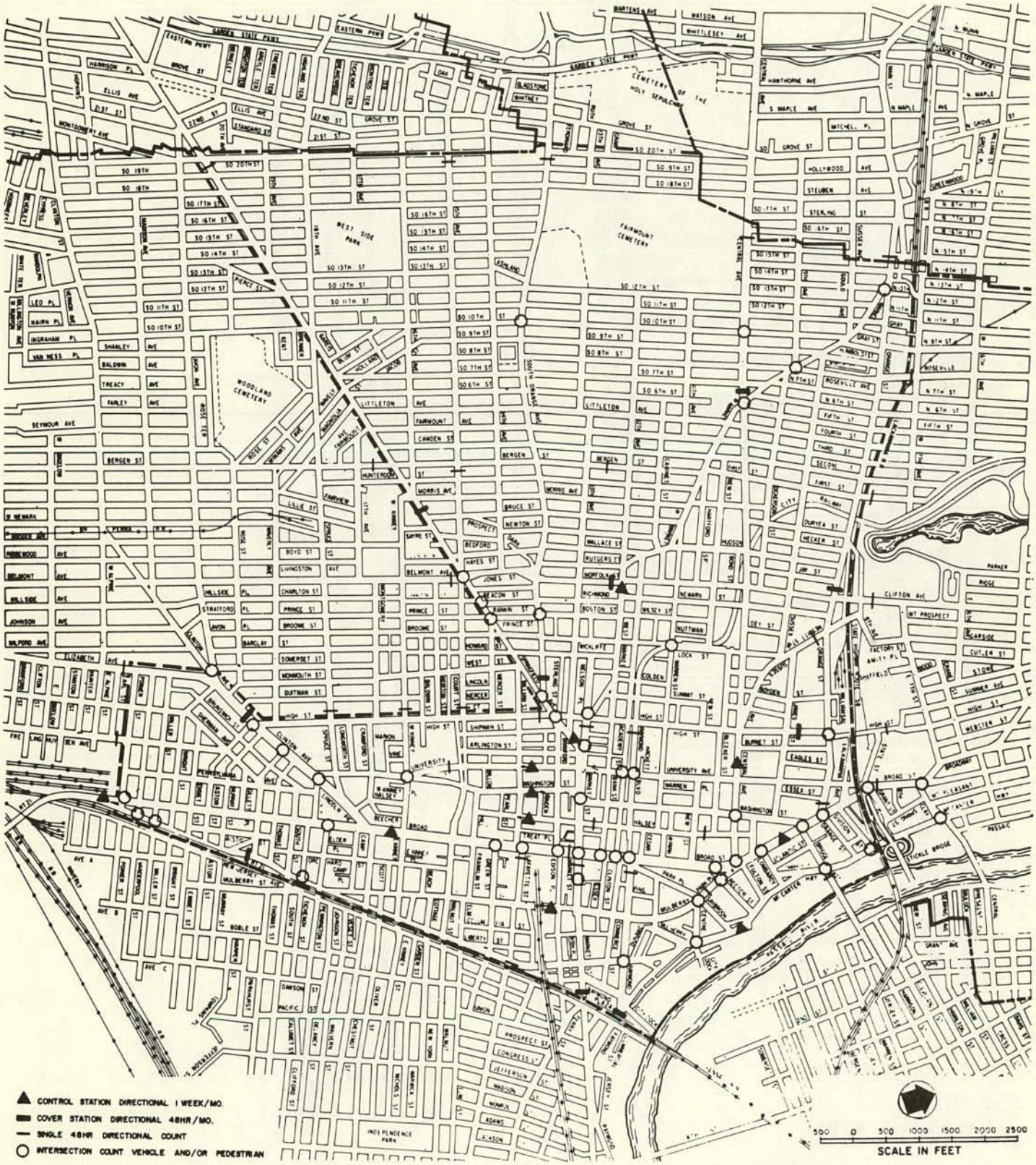


Figure 47. Volume counting stations, Newark.

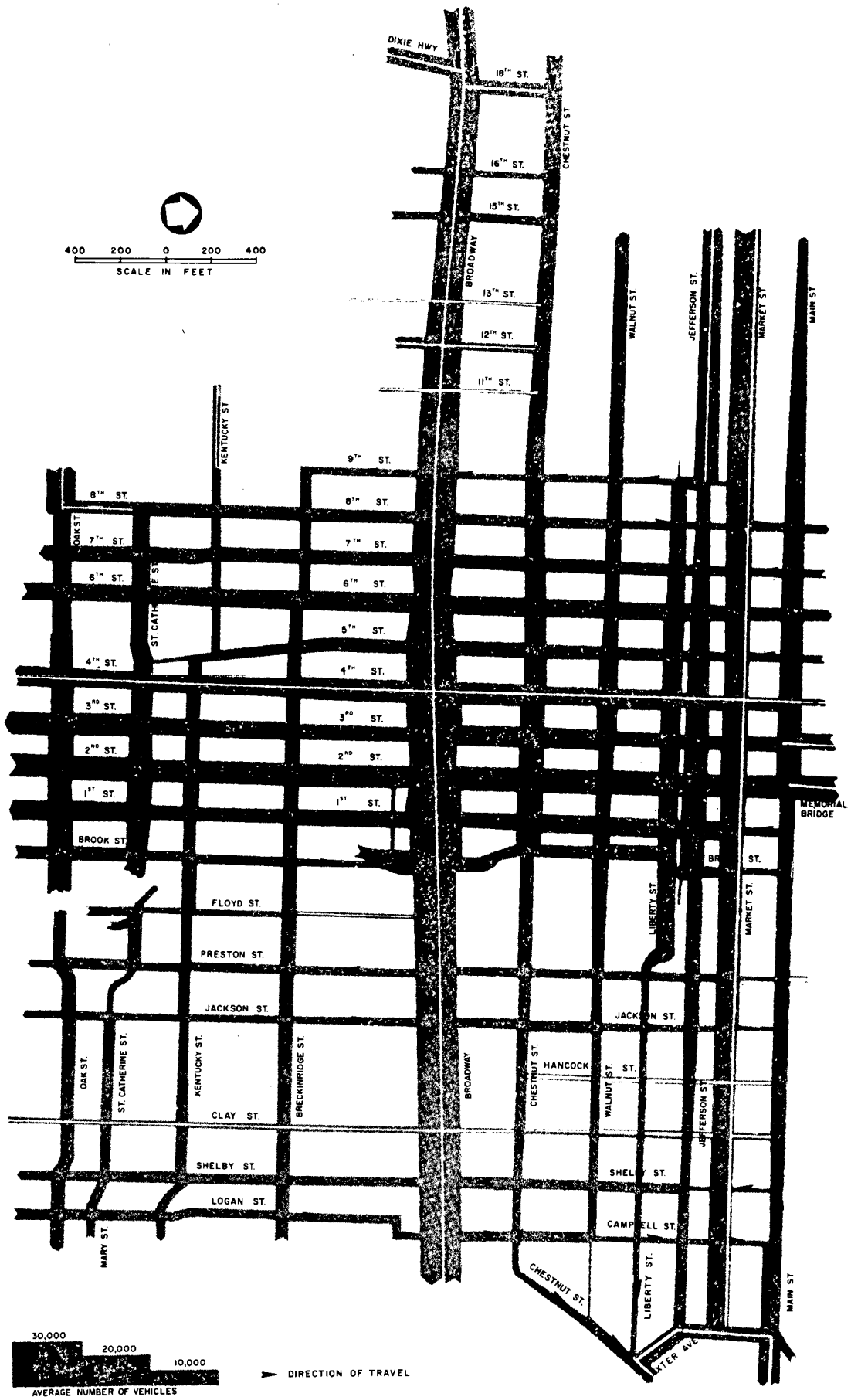


Figure 48. 1968 average weekday traffic, Louisville.

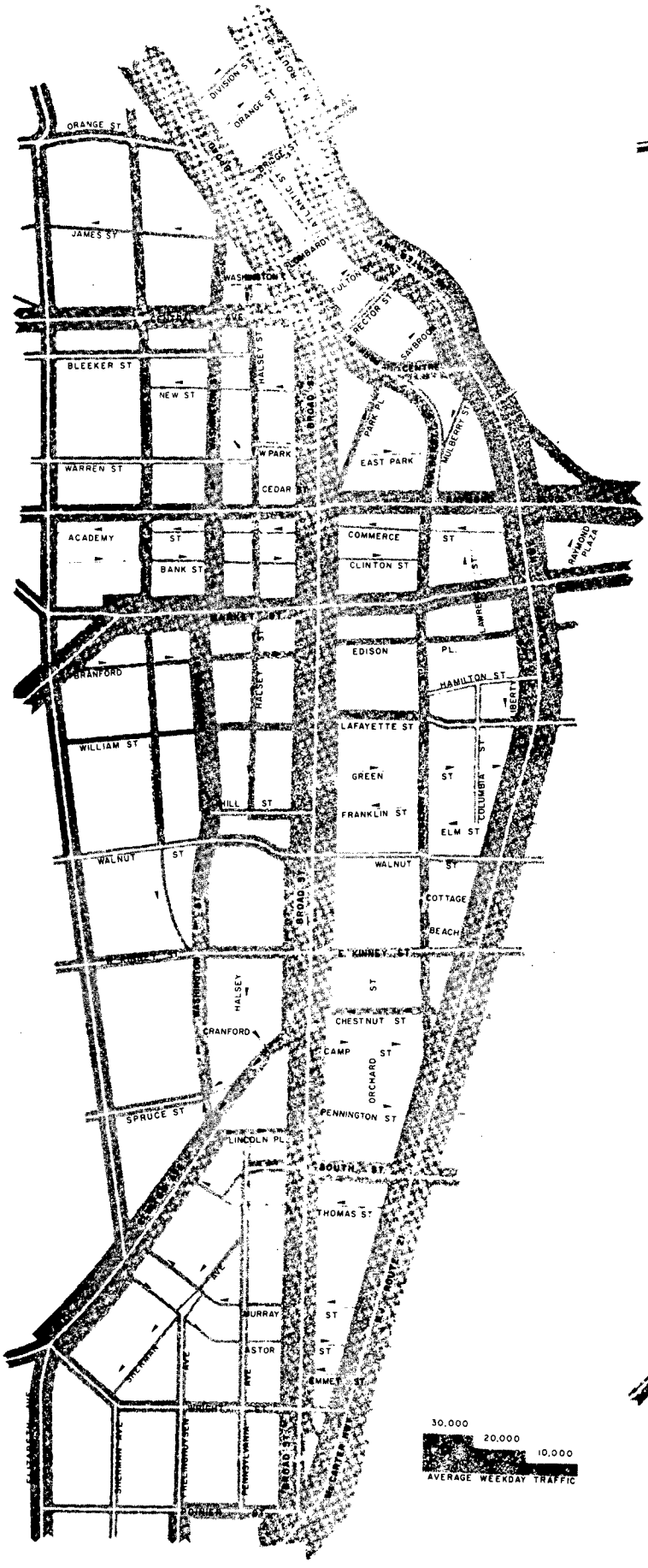


Figure 49. 1968 average weekday traffic, Newark.

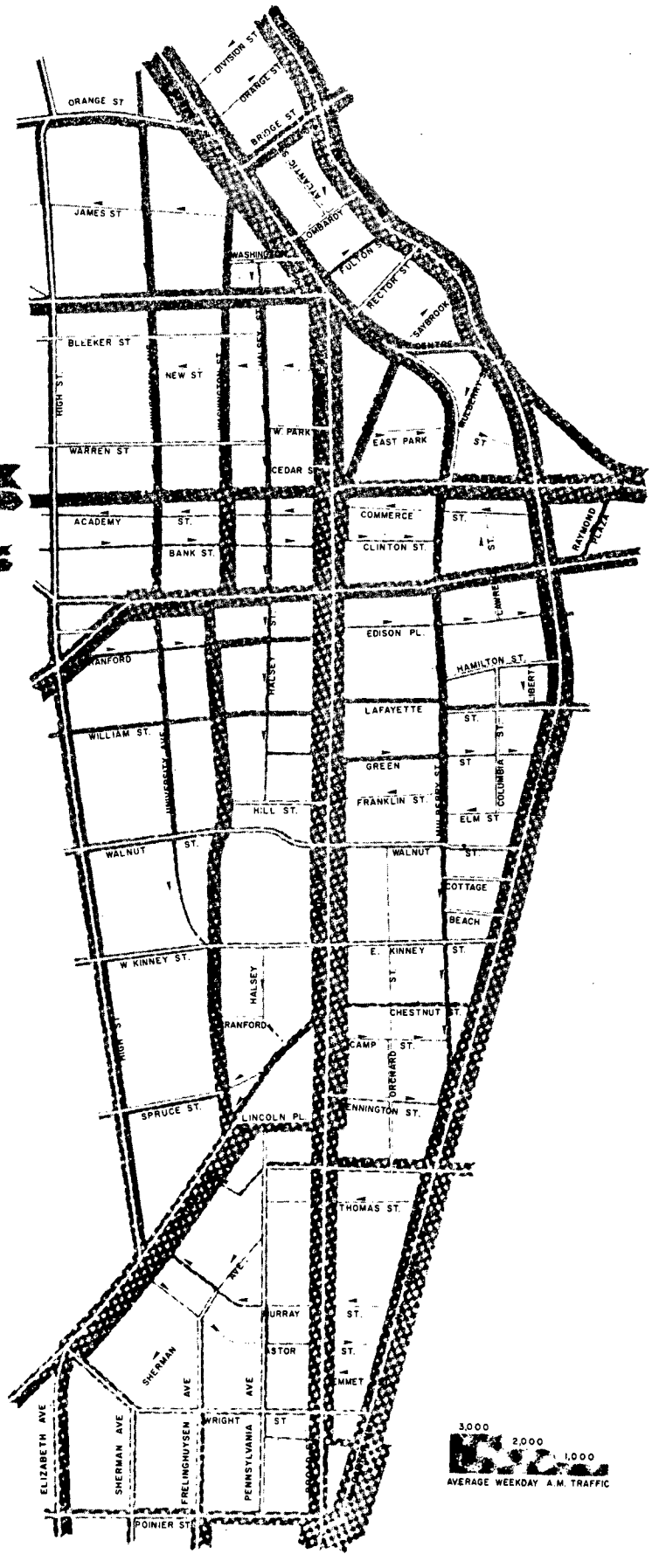


Figure 50. AWDT AM peak-hour flow map, Newark.

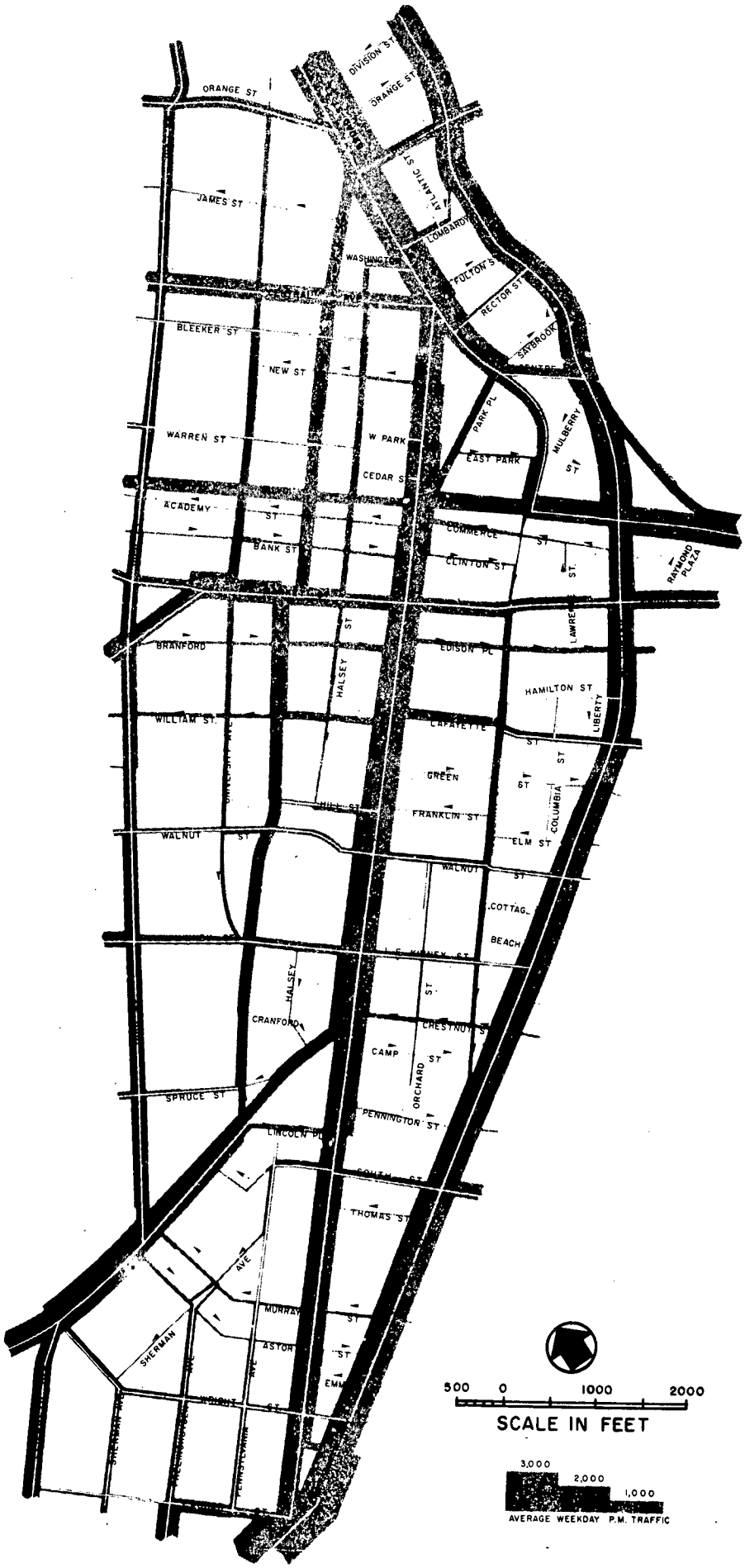


Figure 51. AWDT PM *peak-hour flow map, Newark.*

areas are being studied. A distinct change of viewpoint becomes necessary when one moves from consideration of the problems of expressways and rural roads to study of urban traffic. In the rural environment a network is viewed as a system of intersecting roadways. In an urban area one must begin to think in terms of a system of connected intersections. The road sections often serve as little more than approaches to and departures from intersections. Because of the dominant interest in and importance of the intersection, it becomes necessary to develop an expression of the action of traffic at an intersection in terms of travel time.

At fixed-time signal-controlled intersections, the passage of time is directly measured by the signal control equipment. Traffic at these locations is supervised in its movement by this timing mechanism. In a densely signalized downtown area the control equipment is usually "fixed time," and very often such equipment at all intersections is set to a common cycle length. Therefore, the cycle becomes the logical unit for measurement of time. Variations of flow within the cycle have been studied but are found to be of little significance when compared to the large variations that are common between cycles. In Experiments A53 and D15, surveys were organized to develop data describing the arrival rate of vehicles for very small time intervals (3 sec). Such surveys proved to be impractical, requiring considerable and often rather subjective judgment on the part of the field observer.

The problem of measuring delay time at an intersection has been recognized for some time and has led to the development of a series of intersection delay models. Most of these models are based on a queuing theory developed from estimations of arrival and departure rates. Intersection surveys were designed to provide input data for a delay model as well as to conform to the needs of direct intersection analysis.

The specific counts designed for intersection evaluation were the number of vehicles stopped in all lanes at the end of each red interval (vehicles stopped on red) and the number of vehicles through the intersection during each green interval (vehicles through) for each approach to the intersection. Turn lanes were usually counted as a separate approach. Counts were recorded by cycle for a 1-hr period, usually 7:30 to 8:30 AM for morning peak-hour counts and 4:30 to 5:30 PM for evening peak-hour counts. One man, using tally-board counters, was required to record the vehicles stopped and vehicles through at each approach leg. When a suitable vantage point was available, time-lapse photography was used to record data at complex intersections.

The number of vehicles stopped was recorded by lane, and the number of vehicles through was recorded by turning movement and vehicle type (truck, bus, or car) for each approach leg. During periods of extreme congestion, the number of vehicles stopped (or queue length) at some approach legs occasionally extended beyond the next upstream signalized intersection. When the upstream intersection was less than 500 ft from the study intersection, the queue count was extended beyond the upstream intersection. If the upstream intersection was more than 500 ft

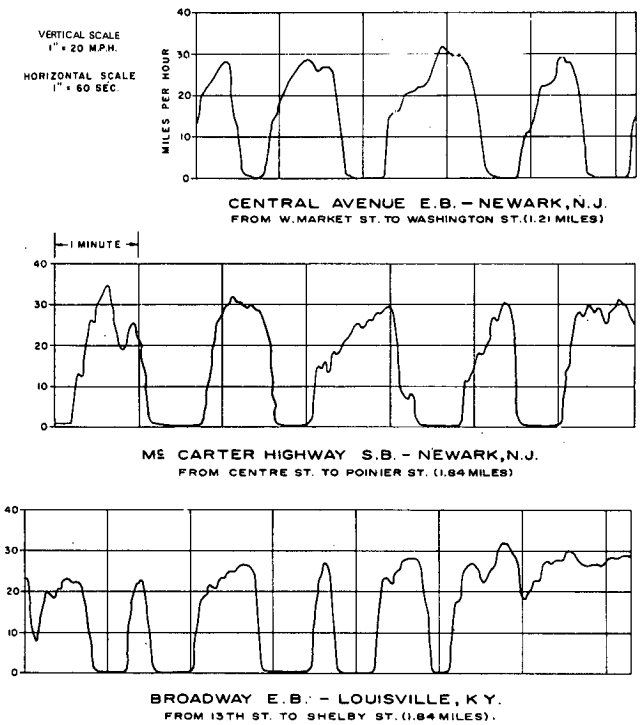


Figure 52. Typical speed profiles.

from the study intersection, the queue count was limited to the number of vehicles stored between intersections. No attempt was made to include vehicles eventually turning into the approach from an upstream intersection in the queue count. Each cycle, when the queue length extended beyond the upstream intersection, was identified.

Unusual events (such as stalled vehicles, illegal parking, signal malfunctions, and minor accidents) that may have had an effect on any count, were noted; and the time of their occurrence and their duration were recorded. A general reconnaissance of the area surrounding the study intersection was maintained prior to and during the hour of counting to observe any abnormal event that could affect traffic flow through the intersection.

TRAVEL TIME ANALYSIS

Field data collected during the travel time and intersection-count surveys were transferred to data-processing cards for computer summarization and analysis. All data were plotted to detect any counts that may have been recorded erroneously or may have been influenced by an unusual occurrence. These counts, termed "outliers," were eliminated prior to analysis.

The analysis of surveys to determine operational improvements was performed differently for measurements involving roadway sections than for measurements of individual intersections. However, the results of these analyses were compatible for both types (roadway or intersection) of operational improvement; that is, both were expressed in terms of travel time and delay time.

Roadway Analysis

An operational change extending over a considerable length of roadway was surveyed through test-car travel time studies performed before and after the change was implemented. Volume counts also were obtained at selected locations along the roadway prior to and after implementation. Time between implementation and "after" surveys was usually sufficient to allow traffic to adjust to the operational changes. In several instances external circumstances required "after" measurements to be made prior to complete adjustment of traffic to the change.

Variations of the volume counts and travel time measurements determined the sample size required to distinguish meaningful changes in traffic flows. A statistical analysis of the "before" and "after" data was performed to measure the significance of the changes in traffic volume or travel time resulting from the operational change. Specifically, the direct effect of any roadway operational change was expressed in terms of changes in the hourly volume, 15-min rate of flow, travel speed, travel time, delay time, and number of stops along the section of roadway.

The results of a travel time survey refer to an average of several sample trips in one direction over the same section of roadway during a specific time period (hour). It is also necessary to develop a common basis for comparison between different sections of roadway or between different roadways that will compensate for the various lengths of roadway being surveyed. This may be done by expressing the various categories of delay time as a percentage of total travel time or by relating the measurements to a unit mile of roadway.

A comparison of measured results of the implementation of various signal progression systems on different segments of roadway in Newark is given in Table 17. All of the roadways listed are two-way, with a predominant directional flow during the morning and evening peak hours. The greatest measured change was for the northbound, northern section of McCarter Highway (B93) during the PM peak hour. The travel speed increased from 7.4 to 16.4 mph on this section as a result of the signal progression, and the delay time was reduced by 200 sec per mile of travel. The listing indicates a signal progression favoring the predominant direction of flow can result in average travel speeds close to 15 mph, in delay time between 50 and 70 sec per mile of travel, and a delay ratio between 0.22 and 0.30 for traffic in the heavy direction of a downtown roadway, with no adverse effect on the flow in the minor direction.

Intersection Analysis

An operational change at a specific intersection, not expected to influence flow beyond the immediate area of the intersection, was surveyed by counting the number of vehicles stopped at the end of each red interval (vehicles

TABLE 17

COMPARISON OF SIGNAL PROGRESSION EXPERIMENTS

EXPERIMENT—ROADWAY	DIRECTION	TIME PERIOD	CONDITION	DISTANCE (MILES)	TRAVEL SPEED (MPH)	TRAVEL TIME PER MILE (SEC)	DELAY TIME PER MILE (SEC)	STOPS PER MILE	DELAY RATIO
B100—Broad St. northern section	NB [a]	PM	"Before"	0.294	8.9	404	182	7.1	0.45
	NB [a]	PM	"After"	0.294	14.3	251	57	3.4	0.23
	SB [b]	PM	"Before"	0.294	9.9	364	157	4.8	0.43
	SB [b]	PM	"After"	0.294	8.4	428	211	5.8	0.49
B93—McCarter Hwy.	NB	AM	"Before"	2.298	12.7	284	111	3.5	0.39
	NB	AM	"After"	2.298	14.5	248	71	2.8	0.29
	SB	AM	"Before"	2.348	13.1	274	101	3.3	0.37
	SB	AM	"After"	2.348	15.3	236	70	2.9	0.30
B93—McCarter Hwy. northern section	NB [a]	PM	"Before"	0.941	7.4	485	252	7.5	0.52
	NB [a]	PM	"After"	0.941	16.4	220	52	2.0	0.24
	SB [b]	PM	"Before"	0.941	12.6	286	106	3.5	0.37
	SB [b]	PM	"After"	0.941	14.2	254	97	3.6	0.38
B93—McCarter Hwy. southern section	NB [b]	PM	"Before"	1.357	15.7	229	89	2.7	0.39
	NB [b]	PM	"After"	1.357	13.6	264	97	4.1	0.37
	SB [a]	PM	"Before"	1.407	12.8	281	109	3.5	0.39
	SB [b]	PM	"After"	1.407	16.7	216	48	1.8	0.22
B88—Springfield Ave.	EB [a]	AM	"Before"	0.704	11.0	328	124	4.1	0.38
	EB [a]	AM	"After"	0.704	15.9	226	54	2.3	0.24
	WB [b]	AM	"Before"	0.704	15.8	228	56	2.3	0.25
	WB [b]	AM	"After"	0.704	14.8	244	71	3.4	0.29
	EB [b]	PM	"Before"	0.704	10.9	331	118	3.8	0.36
	EB [b]	PM	"After"	0.704	10.7	338	141	5.5	0.42
	WB [a]	PM	"Before"	0.704	9.0	400	157	5.8	0.39
	WB [a]	PM	"After"	0.704	12.0	299	85	3.3	0.28

[a] Predominant direction of flow.
[b] Minor direction of flow.

stopped on red) and the number of vehicles clearing each green interval (vehicles through) at each approach. For some intersection experiments (A7, A33, A40, D13, and D20), where the area of influence was in doubt prior to implementation, test-car travel surveys were performed to supplement the intersection counts.

A statistical analysis was performed to measure the significance of the changes in the number of vehicles clearing each cycle, the number of vehicles stopped each cycle, and the ratio of the number of saturated cycles to the total number of cycles at each approach to the intersection as a result of the operational change. Generally, three days of counts before and after implementation of the operational change were analyzed for each hour period measured.

As stated previously, direct field measurements of delay time at intersections proved to be impractical. Although extreme accuracy of delay time is not a necessary requirement because of the large fluctuations and variations of flow in urban networks, an elaborate technique would still be required to sample vehicular delay directly. However, it is possible to measure approach and departure rates at each approach to compute an approximate delay time. An estimation of the change in delay time resulting from an operational improvement at an intersection is required so that this evaluation will be compatible with that of roadway improvements.

Several intersection delay models based on the queuing theory have been developed over the past two decades. The model developed by Newell (*6*) has proven to be promising for estimating the average delay at an intersection, using field counts of vehicles stopped and vehicles through per cycle. The derivation and use of the Newell delay model is discussed in Appendix B. A comparison of estimated values (using Newell's equation) with measured values for several approach legs to intersections is given in Table 18.

The difference between "before" and "after" delay per vehicle at the Washington Street northbound approach to Market Street was measured by test-car travel runs and compared with the estimated difference using the Newell model. The comparison is surprisingly close. However, it should be remembered that the test-car "measured" value of delay per vehicle is, in itself, only an approximation.

The equation used to estimate "before" and "after" delay time per vehicle for an approach:

$$\overline{w} = \frac{R^2}{2(R+G)(1-q/s)} + \overline{Q}_R - qR \qquad (4)$$

in which

$\overline{w}$ = average delay per vehicle (minutes);
R = red interval (minutes);
G = green interval (minutes);
$\overline{Q}_R$ = average number of vehicles stopped at the end of a red interval;
q = arrival rate (vehicle/minute); and
s = service rate (vehicles/minute green);

contains all measured values except the arrival rate (q) and service rate (s). The arrival rate is merely the average number of vehicles through per cycle divided by the cycle length in minutes. The service rate (s) is the discharge rate under full demand and can be obtained from cycle counts where the measurement "vehicles stopped on red" is greater than "vehicles through." The computation of s is the average number of vehicles through during these saturated cycles divided by the green time in minutes. The s value

TABLE 18

SUMMARY OF NEWELL'S INTERSECTION MODEL TESTS

INTERSECTION APPROACH	ITEM COMPARED	MEASURED VALUE	NEWELL INTERSECTION MODEL: ITEMS INPUT	NEWELL INTERSECTION MODEL: EST. VALUE
Washington St. NB at West Market St.	Delay	−0.13 min/veh	$R, G, q, s, \overline{Q}_R$ ("before and "after")	−0.13 min/veh
Central Ave. WB at High St.	$\overline{Q}_R$ "before"	24.7 veh/cycle	R, G, q, s, I ("before")	21.4 veh/cycle
	$\overline{Q}_R$ "after"	18.3 veh/cycle	R, G, q, s, I ("after")	18.4 veh/cycle
	$\overline{Q}_R$ "after"	18.3 veh/cycle	R, G ("after"); q, s, I ("before")	15.6 veh/cycle
Central Ave. EB at West Market St.	$\overline{Q}_R$ "before"	34.6 veh/cycle	R, G, q, s, I ("before")	34.2 veh/cycle
	$\overline{Q}_R$ "after"	16.7 veh/cycle	R, G, q, s, I ("after")	19.1 veh/cycle
	$\overline{Q}_R$ "after"	16.7 veh/cycle	R, G ("after); q, s, I ("before")	16.7 veh/cycle
West Market St. SB at Central Ave.	$\overline{Q}_R$ "before"	12.7 veh/cycle	R, G, q, s, I ("before")	15.9 veh/cycle
	$\overline{Q}_R$ "after"	33.6 veh/cycle	R, G, q, s, I ("after")	∞ veh/cycle
	$\overline{Q}_R$ "after"	33.6 veh/cycle	R, G ("after"); q, s, I ("before")	∞ veh/cycle

R = red time (min);
G = green time (min);
q = arrival rate (veh/min);
s = service rate (veh/min green);
$\overline{Q}_R$ = average number of vehicles stopped at end of red interval; and
I = composite arrival-departure coefficient of variability.

computed at several intersections was generally found to equal 20 vehicles per minute of green time per lane.

Because the delay was never measured directly at any intersection, the other tests of Newell's model compare the average measured number of vehicles stopped per cycle ($\bar{Q}_R$) with the estimated value from Newell's equation.

$$\bar{Q}_R = \frac{Iq}{2s}\left[\frac{G}{R+G} - q/s\right]^{-1} + qR \qquad (5)$$

This equation introduces a new term (I), in which

$$I = \frac{\text{Variance } A}{\bar{A}} + \frac{\text{Variance } D}{\bar{D}} \qquad (6)$$

the composite of the arrival (A) and departure (D) coefficient of variability. Although a review of the variance of departures over a 1-hr period, discussed in Chapter Five, has shown that departures are generally a Poisson distribution (variance = mean), this general finding should not be assumed for specific approaches. In fact, a change of green time at an approach would probably alter the coefficient of variability for departures at that approach. It may be more reasonable to assume a general Poisson arrival distribution, using the logic that arrivals at one intersection are the departures from the upstream intersections. Calculations of I-values from measured Q_R counts at several intersection approaches showed that I ranged between 1.0 and 2.0.

This value of variability is not required when estimating the delay time per vehicle from actual counts. It seems reasonable, therefore, to assume that Newell's model would be more reliable to estimate delay time than to estimate Q_R. The problem found to occur at the West Market Street approach to Central Avenue, where the measured congestion results in an estimated infinite Q_R and infinite delay per vehicle when using Newell's model, can be avoided by adding values for each cycle rather than using average values. The method of summing the delay for each cycle has been explored by Sagi and Campbell (*7*).

TABLE 19

LEVEL-OF-SERVICE COMPARISON FOR SELECTED DOWNTOWN ROADWAYS

ROADWAY	TRAVEL SPEED (MPH)	DELAY PER MILE (SEC)	DELAY RATIO	LEVEL OF SERVICE
McCarter Hwy.	14.0	85	0.33	D
Central Ave.	16.0	47	0.21	C
Walnut St.	17.6	34	0.17	C
Brook St.	20.9	25	0.15	B
Second St.	16.6	57	0.26	C
Broadway EB	18.8	38	0.20	C
Broadway WB	14.6	70	0.28	D
Springfield Ave.	13.0	75	0.27	D
Halsey St.	10.1	254	0.71	E
Average	15.5	58	0.25	C-D

LEVEL-OF-SERVICE DEFINITION

The use of delay time as an evaluation measurement of traffic congestion on urban roadways and intersections can be related to the "level-of-service" term, where level of service is a qualitative measure of the effect of a number of factors. The *Highway Capacity Manual* (*1*, Chaps. 4, 6, 9, and 10) defines specific levels of service in terms of particular limiting values of certain factors and includes a narrative description of prevailing traffic flow conditions, which represent the levels of service for several types of roadways.

Downtown Streets

The *Manual* (*1*, pp. 232-33) does not develop a specific relationship between certain factors and a level-of-service rating for downtown streets but does provide a general scale based entirely on average over-all travel speeds. The measured values of delay per mile, delay ratio, and travel speed on several roadways in the downtown areas of Newark and Louisville are compared with the *Manual's* suggested level of service (Table 19). The delay per mile and the delay ratio for each roadway is compatible with the travel speed used to define the various roadways.

Table 20 compares the delay per mile and delay ratio with the level of service for roadways before and after signal progression changes. In all cases, the improvement in the predominant direction of flow represents an improvement in the level of service. The largest improvement, on the northern section of McCarter Highway northbound during the PM peak hour, represents a change from level of service E to level of service C.

Signalized Intersections

The level of service of downtown roadways is often controlled by the intersections along the roadway. If a downtown roadway is operating at level of service E, probably one or more intersection approaches along the roadway are also at level of service E. Chapter Six of the *Highway Capacity Manual* describes the conditions for the levels of service at isolated signalized intersection approaches and relates the conditions to the load factor. The load factor is the ratio of the number of fully used green phases to the total number of green phases available for an approach during a specific time period.

The number of loaded cycles, or green phases that are fully used by traffic, had not been recorded during field counts and is not a necessary measure of intersection evaluation developed for this project. Identification of a loaded cycle requires some judgment on the part of the field observer, and the load factor may not be the best measure of degree of congestion for intersections within a coordinated signal network system.

The delay time per vehicle would be an excellent measure describing driver satisfaction at a signalized intersection. However, delay has not been measured directly and requires the use of an estimation model. Some research would be required to develop the relationship between delay time and the load factor to define a level of service compatible with the *Highway Capacity Manual.*

The number of saturated cycles, where some drivers wait

TABLE 20

LEVEL OF SERVICE FOR SIGNAL PROGRESSION EXPERIMENTS

EXPERIMENT—ROADWAY	DIRECTION	TIME PERIOD	CONDITION	TRAVEL SPEED (MPH)	DELAY PER MILE (SEC)	DELAY RATIO	LEVEL OF SERVICE
B100—Broad St. northern section	NB [a]	PM	"Before"	8.9	182	0.45	E
	NB [a]	PM	"After"	14.3	57	0.23	D
	SB [b]	PM	"Before"	9.9	157	0.43	E
	SB [b]	PM	"After"	8.4	211	0.49	E
B93—McCarter Hwy. northern section	NB [a]	PM	"Before"	7.4	252	0.52	E
	NB [a]	PM	"After"	16.4	52	0.24	C
	SB [b]	PM	"Before"	12.6	106	0.37	D
	SB [b]	PM	"After"	14.2	97	0.38	D
B93—McCarter Hwy. southern section	NB [b]	PM	"Before"	15.7	89	0.39	C
	NB [b]	PM	"After"	13.6	97	0.37	D
	SB [a]	PM	"Before"	12.8	109	0.39	D
	SB [a]	PM	"After"	16.7	48	0.22	C
B88—Springfield Ave.	EB [a]	AM	"Before"	11.0	124	0.38	D
	EB [a]	AM	"After"	15.9	54	0.24	C
	WB [b]	AM	"Before"	15.8	56	0.25	C
	WB [b]	AM	"After"	14.8	71	0.29	D
B88—Springfield Ave.	EB [b]	PM	"Before"	10.9	118	0.36	D
	EB [b]	PM	"After"	10.7	141	0.42	D
	WB [a]	PM	"Before"	9.0	157	0.39	E
	WB [a]	PM	"After"	12.0	85	0.28	D

[a] Predominant direction of flow.
[b] Minor direction of flow.

more than one red interval, has been counted and does contain some relationship to a loaded cycle. Each cycle where the number of vehicles stopped at an approach during the red interval is greater than the number of vehicles clearing during the following green interval is a saturated cycle for that approach and is, in fact, also a loaded cycle. Although a loaded cycle need not be saturated, it usually represents a saturated condition except for near-perfect signal progressions. In no case, however, can the number of loaded cycles be less than the number of saturated cycles.

Table 21 indicates the approximate relationship between the factors used to describe the levels of service in the *Highway Capacity Manual* and the factors used for evaluation of downtown streets and intersections on this project. A condensation of the *Manual's* narrative description of traffic conditions is given in Table 21. The delay ratio and the saturation factor are clearly recognized in this description of traffic conditions and are certainly compatible with the *Manual's* definitions of level of service. The relationship of the delay ratio and travel speed for downtown roadway level-of-service definitions is shown in Figure 53.

NETWORK ANALYSIS

The final step of an experiment was to evaluate the degree of change in network flow resulting from a specific operational improvement. The measurements selected to express relative congestion on roadway sections and at individual intersections were developed for the entire network system so that the direct changes could be related to the total flow within the entire network. The specific measurements required for network analysis include total vehicle-miles of travel, total vehicle-hours of travel, over-all average travel speed, and the network delay ratio.

The network effects of any traffic operational change can be classified into two general categories—limited diversion of traffic or substantial diversion of traffic—depending on the area influenced by the change. The network analysis differs for each category. The network effects can be expressed directly from the measured local effects when the area of influence is limited to the immediate area of the operational change. A more complex analysis is required when the change results in substantial diversion of trips that may influence traffic beyond the immediate area of the operational change.

It has been assumed that the normal traffic operational changes would very seldom alter the total volume of trips within the network system, either by inducing new trips into the area or causing a change in the mode of travel to the area. It has also been assumed that traffic operational changes would not alter any trip origins or destinations within the area.

Limited Diversion of Traffic

The majority of the traffic operational changes implemented in Louisville and Newark during this project resulted in little or no measured diversion of traffic; that is, there were no significant directional hourly volume changes measured along the roadway or at the intersection approaches in the immediate area of the traffic operational change. When it had been determined that there were no volume changes (or trip diversions) as a result of the operational changes, the network analysis consisted of adjusting the base net-

TABLE 21

COMPARISON OF LEVEL-OF-SERVICE DEFINITIONS

	DOWNTOWN STREET			SIGNALIZED INTERSECTION APPROACH		
LEVEL OF SERVICE [a]	DESCRIPTION OF TRAFFIC CONDITION [a]	TRAVEL SPEED (MPH) [a]	DELAY RATIO [c]	DESCRIPTION OF TRAFFIC CONDITION [b]	LOAD FACTOR [b]	SATURATION FACTOR [d]
A—free flow	Some stops	≥25	≤0.05	No cycle fully used. No vehicle waits longer than one red indication.	0.0	0.0
B—stable flow	Delays not unreasonable	≥20	≤0.15	Occasional cycle fully used. Some restriction to choice of lanes.	≤0.1	≤0.1
C—stable flow	Delays significant but acceptable	≥15	≤0.25	Intermittent cycles fully loaded. Occasional vehicle waits longer than one red indication.	≤0.3	≤0.2
D—approaching unstable flow	Delays tolerable	≥10	≤0.40	Substantial delay short peaks; enough cycles with low demand to clear developed queues.	≤0.7	≤0.5
E—unstable flow	Congestion	Less than 10 but moving	≤0.60	Great delays; long developing queues; some vehicles wait several signal cycles.	≤1.0	≤0.8
F—forced flow	Jammed	Stop and go	—	Jammed—due to downstream restriction.	—	—

[a] *Highway Capacity Manual* (Table 10.14).
[b] *Highway Capacity Manual* (pp. 130-131).
[c] Ratio of average delay time to average total travel time.
[d] Ratio of the number of saturated cycles to total number of cycles.

work values in accordance with the direct changes measured at the immediate area of the improvement. The base network values were developed from the volume counts and travel time surveys conducted throughout the system. The direct changes measured at the immediate area of improvement are a result of the First Level Analysis.

Base Network Values

The base network surveys provided the distance in miles, the volume in vehicles per hour, and the average travel time in minutes for each section (link) of roadway, by direction, within the network system during a given time period. The total vehicle-miles of travel is equal to the sum of the products of the volume and distance of each section of roadway. The total vehicle-hours of travel is equal to the sum of the products of the volume and travel time of each section of roadway, converted from minutes to hours. The average over-all travel speed is merely the total vehicle-miles of travel divided by the total vehicle-hours of travel. The general relationship between the delay ratio and travel speed for downtown roadways (Fig. 53) provided an estimation of the total delay time within the network.

In Louisville, the base network values represented "straight-through" traffic only. The one-way-street pattern and the generally light pedestrian movements did not cause significant delays for turning vehicles. In Newark, the radial pattern of two-way streets and the heavy volume of pedestrians resulted in substantial delays for turning vehicles. Supplementary manual and time-lapse photography surveys were conducted at several complex intersections to determine the additional time for turning vehicles to be included in the base network total vehicle-hours of travel.

Twenty-nine miles of roadway were selected for the network analysis of Louisville, and approximately 35 miles of roadway within the downtown area were included in the Newark network. The network values of vehicle-miles and vehicle-hours were computed from the measured roadway distances, volumes, and travel times for the AM and PM peak hours in Louisville and for the PM peak hour in Newark. The delay ratio was estimated from the relationship with travel speed (Fig. 53). The network values are given in Table 22. Unlike Newark, the Louisville roadway system extends beyond the downtown area and includes some roadways with little or no congestion.

Network Evaluation

The First Level Analysis measured the statistical significance of any changes of traffic flow resulting from each operational change. When no significant volume change was observed as a result of the operational change, it was assumed that there was no diversion of traffic and that the total vehicle-miles of travel would not change. The observed changes of travel time over the applicable sections of roadway were used to compute the change in network vehicle-hours of travel. In the case of operational changes at intersections, the delay time estimated by the Newell model was used to determine the change of network vehicle-hours.

TABLE 22

BASIC NETWORK VALUES

ITEM	LOUISVILLE AM PEAK HR	LOUISVILLE PM PEAK HR	NEWARK PM PEAK HR
Vehicle-miles of travel	26,850	29,700	26,327
Vehicle-hours of travel	1,408	1,713	2,748
Average travel speed (mph)	19.1	17.3	9.6
Est. delay ratio	0.19	0.225	0.43
Est. vehicle-hour delay	268	386	1,183

As an example of determining the network effect of a traffic operational change at an intersection, the total delay at Washington Street and Market Street during the PM peak hour has been estimated for conditions before and after the pedestrian control and channelization revision, Experiment A33. The measured values and estimated delay time are given in Table 23. While the difference between "before" and "after" vehicles clearing is not significant, the "before" number of vehicles clearing each approach ($\overline{V}$) was used to estimate the total delay time (D). If the "after" number of vehicles were used, the total delay time (D) "after" would be 16.62 veh-hr rather than 17.03 veh-hr.

The difference of 5.90 veh-hr delay time (22.93 — 17.03) represents a 26-percent reduction of total delay time at this intersection during the PM peak hour. When related to the network total delay time of 1,183 veh-hr, the change represents a 0.5-percent reduction in delay time. In terms of the total travel time, the change represents a 0.2-percent reduction of the 2,748 veh-hr of travel. Although these changes at one intersection are not very significant when related to total network flow, similar changes at several intersections would result in a considerable improvement of network flow.

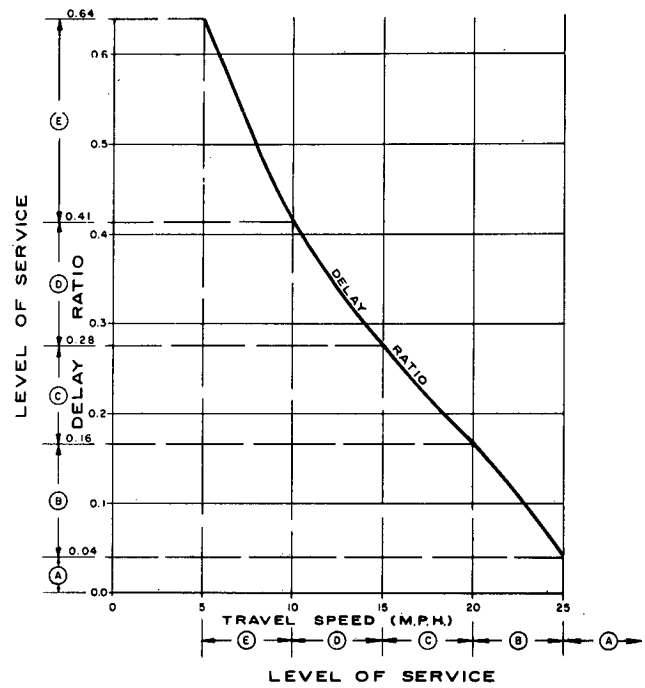


Figure 53. Level-of-service definitions, downtown roadways.

Substantial Diversion of Traffic

Several individual operational changes, such as altering the direction of travel along one or more roadways (B78), establishing turn lanes at critical intersection approaches (A7), revising the allocation of "green" time at an intersection to favor one movement (A69), signing to direct movements (D68), and establishing signal progressions along a roadway (B93), resulted in significant volume changes in the immediate area of the experiment. The combination of several intersection improvements along a

TABLE 23

ESTIMATED INTERSECTION DELAY TIME FOR EXPERIMENT A33 (DURING PM PEAK HOUR)

APPROACH LEG	CONDITION	CYCLE LENGTH, C (MIN)	RED INTERVAL, R (MIN)	AVERAGE QUEUE LENGTH PER CYCLE, $\overline{Q}_R$	AVERAGE VEHICLES THROUGH PER CYCLE, V	ARRIVAL RATE, q (VEH/MIN)	SERVICE RATE, s (VEH/MIN)	AVERAGE DELAY PER VEHICLE, $\overline{W}$ (MIN)	TOTAL DELAY, D (VEH-HR)
Washington St. NB—left turn	"Before"	1.50	0.90	7.6	13.0	8.7	24.7	0.41	3.55
	"After"	1.50	0.84	5.1	12.8	8.5	24.7	0.36	3.12
Washington St. NB—straight and right	"Before"	1.50	0.90	16.6	25.9	17.2	46.5	0.49	8.46
	"After"	1.50	0.84	12.0	24.6	16.4	46.5	0.36	6.21
Market St. WB	"Before"	1.50	0.60	13.8	22.9	15.3	32.9	0.52	7.94
	"After"	1.50	0.66	10.0	22.6	15.1	32.9	0.27	4.12
Market St. EB	"Before"	1.50	0.60	4.8	17.9	11.9	22.9	0.25	2.98
	"After"	1.50	0.66	3.3	17.9	11.9	22.9	0.30	3.58
Total intersection	"Before"	1.50	—	42.8	79.7	53.1	127.0	—	22.93
	"After"	1.50	—	30.4	77.9	51.9	127.0	—	17.03

roadway probably would also result in a redistribution of traffic within the network system.

Direct measurement of the network effects of such operational changes can be made by a complete survey of the entire network system before and after each change has been implemented. This method was used to evaluate Experiment E35, Network Signal Coordination in Louisville (see Chapter Two). Owing to the time limitations and the minor effect of these changes on any single roadway, complete volume counts were not obtained after implementing the signal changes. For purposes of analysis, it was assumed that no diversion of traffic occurred. Had diversions occurred and volume counts been available, the volume changes on each section of roadway would have been used to compute the "after" vehicle-miles of travel, vehicle-hours of travel, and vehicle-hours of delay time.

The test-car travel time surveys were used to compute the "before" and "after" vehicle-hours of travel time and delay time for each section of roadway. A comparison of the "before" and "after" network measurements of total vehicle-miles and vehicle-hours of travel, average travel speed, total vehicle-hours of delay time, and the delay ratio is given in Table 24. The delay ratio and total vehicle-hours of delay, estimated from the average travel speed, are also given in Table 24.

During the AM peak hour, the estimated delay ratio (0.19) of all roadways combined was only slightly higher than the measured "before" delay ratio (0.18). Although the "after" measured travel speed was reduced only from 19.1 to 18.4 mph, the measured delay ratio decreased to 0.154. The difference between the estimated delay ratio (0.20) and the measured delay ratio (0.154) is clarified by Figure 54, a plot of the "before" and "after" delay ratio and travel speed measurements for the individual roadways during the AM peak hour. Generally, the points fit the theoretical, or estimated, delay ratio reasonably well, except for Broadway westbound. The "before" measured delay ratio for Broadway westbound (0.33) was relatively high for 14.2 mph. The "after" ratio was decreased to 0.19 while the travel speed was only increased to 15.5 mph. The change in the delay ratio for this one roadway was substantial enough to reduce the delay ratio for the AM peak-hour total of all roadways. The measured delay ratios for the PM time period agree closely with estimated ratios (Fig. 55). The preference given to east-west traffic is evident from the relatively low measured delay ratio for east-west streets and the high measured ratio for north-south streets shown in Figure 54 and Figure 55.

Network Assignment Model

Although complete "before" and "after" surveys of the entire network system are a feasible method to evaluate network effects resulting from a major traffic operational change, this method becomes too time-consuming, especially when several major operational changes are contemplated. Therefore, it is desirable to develop a model to estimate network effects using, as input, the measured local changes. It is also desirable to develop a method to estimate the network effects of alternate operational changes prior to implementation.

Several existing traffic models were investigated for estimating the changes of total vehicle-miles and total vehicle-hours of travel within the entire network system due to operational changes producing a substantial diversion of traffic.

The traffic assignment process developed by the Bureau

TABLE 24

SUMMARY OF TRAVEL TIME DATA, EXPERIMENT E35, NETWORK SIGNAL COORDINATION, LOUISVILLE

ITEM	"BEFORE"	"AFTER"	DIFFERENCE	CHANGE (%)
Total vehicle-miles:				
AM	26,850	26,850	—	—
PM	29,700	29,700	—	—
Total travel (veh-hr):				
AM	1,407.9	1,460.3	+52	+3.7
PM	1,713.0	1,841.0	+128	+7.5
Average over-all speed (mph):				
AM	19.1	18.4	−0.7	−3.7
PM	17.3	16.1	−1.2	−6.9
Total delay (veh-hr):				
AM	255.1	224.5	−30.7	−11.5
PM	437.8	439.9	+2.1	+0.5
Delay ratio:				
AM	0.181	0.154	−0.027	−14.9
PM	0.256	0.239	−0.017	−4.6
Est. delay ratio:				
AM	0.19	0.20	+0.01	+5.3
PM	0.225	0.25	+0.025	+11.1
Est. delay (veh-hr):				
AM	268	292	+24	+9.0
PM	386	461	+75	+19.4

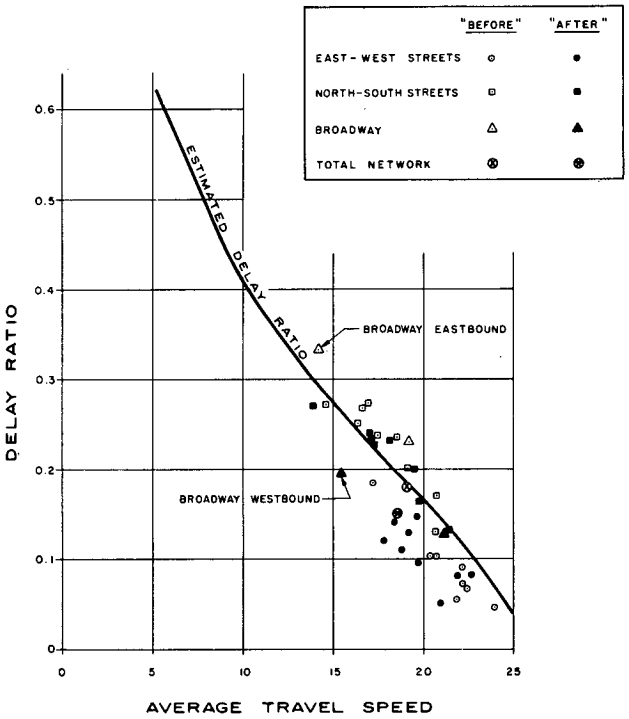


Figure 54. Experiment E35, Louisville roadway delay ratios, AM *peak hour.*

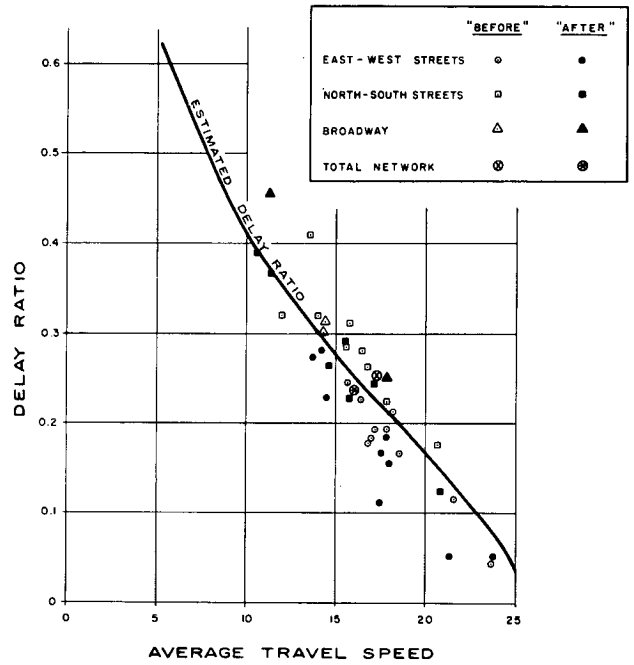


Figure 55. Experiment E35, Louisville roadway delay ratios, PM *peak hour.*

of Public Roads and documented in the *Traffic Assignment Manual* (*8*) was selected as the most promising existing simulation model to be tested for these purposes. The process is well documented and has been used extensively in comprehensive urban transportation studies throughout the United States. The development of this technique for a downtown area would be a logical extension of the urban area transportation assignment package.

Model development and testing of the traffic assignment model was performed for the Newark downtown area (Fig. 56) PM peak-hour network system, as detailed in Appendix C. Very briefly, the procedure was as follows:

1. Develop a fine-grained peak-hour directional trip table, using existing origin-destination data, selected volume counts, and parking inventory information.
2. Test and adjust the trip information, using measured volume counts conducted along various screen lines.
3. Build a fine-grained network system, using measured roadway distances and travel times from test-car travel time surveys.
4. Calibrate the simulated network, using measured vehicle-miles of travel, vehicle-hours of travel, and volume counts along various screen lines.
5. Test the network assignment technique for ability to evaluate the effects of an operational change.

The basic difference between the normal use of the transportation assignment package for urban regional areas and the adaption to a downtown area is in the size (or scale) of the over-all system. There was a total of 28,000 vehicle trips within the downtown area of Newark during the PM peak hour, whereas there are usually several hundred thousand daily trips within most urban areas. There were 136 traffic zones, each one usually representing a city-block area, within the downtown system, whereas this same area might be divided into only 20 or 30 traffic zones for urban systems. The downtown network was less than 2 sq miles in area, and every street was included in the network system, whereas urban areas are generally more than 200 sq miles and include major roadways only. The maximum roadway section (link) was limited to 0.63 mile and 0.63 min of travel time for the downtown system, whereas the maximum link is usually 6.30 miles for urban systems. Each direction of travel on a roadway was coded as a separate link, and turning movements at complex intersections were also coded as individual links for the downtown system, whereas roadway links are usually not so detailed for urban network systems. A sample portion of the coded network map is shown in Figure 57.

Model calibration required four assignments to adjust the downtown network. The first assignment revealed the need for a trip table adjustment, the second assignment led to an adjustment of link travel times along specific roadways, and the third assignment resulted in minor changes to the trip tables and to link travel times in critical areas. A comparison of the vehicle-miles and vehicle-hours of travel resulting from the fourth assignment with direct measurements is as follows:

Figure 56. Newark downtown area.

TABLE 25

TESTS OF NEWARK DOWNTOWN MODEL, PM PEAK HOUR

SECTION AND DIRECTION OF TRAVEL	DISTANCE (MILES)	MEASURED TRAVEL SPEED (MPH) [a]		WEIGHTED VOLUME ON SECTION (VPH) [b]					
				MEASURED			ASSIGNED		
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	CHANGE (%)	"BEFORE"	"AFTER"	CHANGE (%)
Test 1—Experiment B93, McCarter Hwy.:									
Northern section NB	0.941	7.4	16.4	830	1,100	+33	1,100	2,600	+136
Northern section SB	0.941	12.6	14.2	1,350	1,200	−11 [c]	1,050	2,060	+96
Southern section NB	1.357	15.7	13.6	1,100	1,050	−5 [c]	1,200	1,700	+41
Southern section SB	1.407	12.8	16.7	1,300	1,300	0	1,280	1,380	+8
All	4,646	—	—	4,580	4,650	+2	4,630	7,740	+67
Test 2—Experiment B78, Broad St.:									
Northern section NB	0.294	8.8	9.7	2,060	2,180	+6 [c]	2,180	2,260	+4
Northern section SB	0.294	13.0	10.1	1,070	1,160	+8 [c]	1,200	500	−58
All	0.588	—	—	3,130	3,340	+1	3,380	2,760	−18

[a] Measured travel speed = distance ÷ average travel time.
[b] Weighted volume on sections = total vehicle-miles ÷ total distance.
[c] Change in measured volumes not significant.

ITEM	VEH-MILES	VEH-HR	TRAVEL SPEED (MPH)
Measured	26,327	2,748	9.6
Assigned	27,846	2,869	9.7
% difference	+5.77	+4.4	+1.04

The assigned link volumes generally agreed reasonably well with ground-count roadway volumes. There were areas, however, where assigned volumes were considerably higher or lower than ground-count volumes, owing to the inherent limitations of an all-or-nothing assignment procedure. A manual adjustment of assigned volumes within these specific areas was performed to confirm the premise that the differences were due in fact to limitations of the assignment technique and were not the result of distortions within the trip tables or network system. Figure 58 is a procedural flow chart of the development and calibration of the network assignment model.

Model tests were performed to assess the ability of the assignment model to evaluate network traffic changes resulting from an operational change. For each test, the measured local changes of travel time from the First Level Analysis were used to revise the network link travel times for the specific roadway sections involved in the improvement. The calibrated network of the fourth assignment was used to represent the "before" conditions, and the revision of this network to express measured local changes was used for the "after" conditions of each test.

The first test of the network model was for the signal progression revision on McCarter Highway, Experiment B93. The direct effects of this experiment showed a substantial increase in the measured travel speed (from 7.4 to 16.4 mph) and in hourly volume (from 830 to 1,100 vph) in the northbound direction of the northern section of McCarter Highway. The second test was for the implementation of a reversible center lane on the northern section of Broad Street, Experiment B78. Prior to the signal revision (Experiment B100), the direct effects of Experiment B78 showed a substantial decrease in the measured travel speed (from 13.0 to 10.1 mph) in the off-peak southbound direction of Broad Street, with no significant volume change.

Test 1 showed that the model overreacted to a substantial increase in travel speed by assigning too many trips to that section of roadway. Test 2 showed that the model also overreacted by diverting too many trips away from the section of roadway, with a substantial decrease in travel speed. These results, given in Table 25, verify the need for an adjustment, or balancing technique, to obtain reasonable volumes on individual roadways from an all-or-nothing traffic assignment. It should be noted that "after" measurements followed rather closely on each improvement. If a longer period could have been allowed, more complete adjustment of traffic may have altered "after" measurements considerably.

In addition to providing an estimate of the maximum attraction (or diversion) resulting from an operational change, the assignment model does provide information useful for network analysis. The model estimates the "before" and "after" travel times and vehicle-hours of attracted traffic (or diverted traffic) that otherwise would require an extensive field survey of a large portion of the network. A range indicating the maximum and minimum anticipated benefits can be estimated. For example, in Test 1 the assigned "after" 2,761 veh-hr or total travel

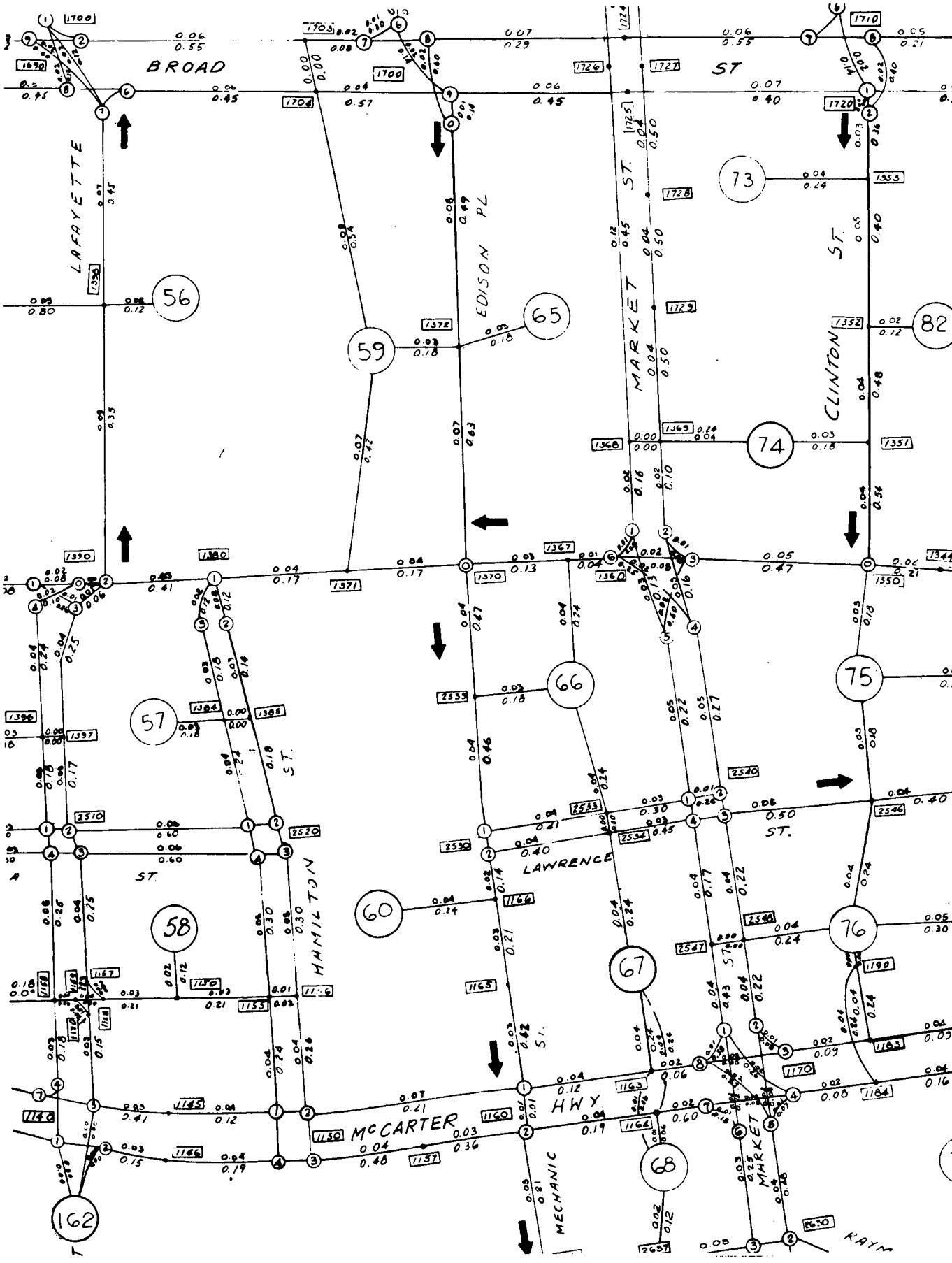
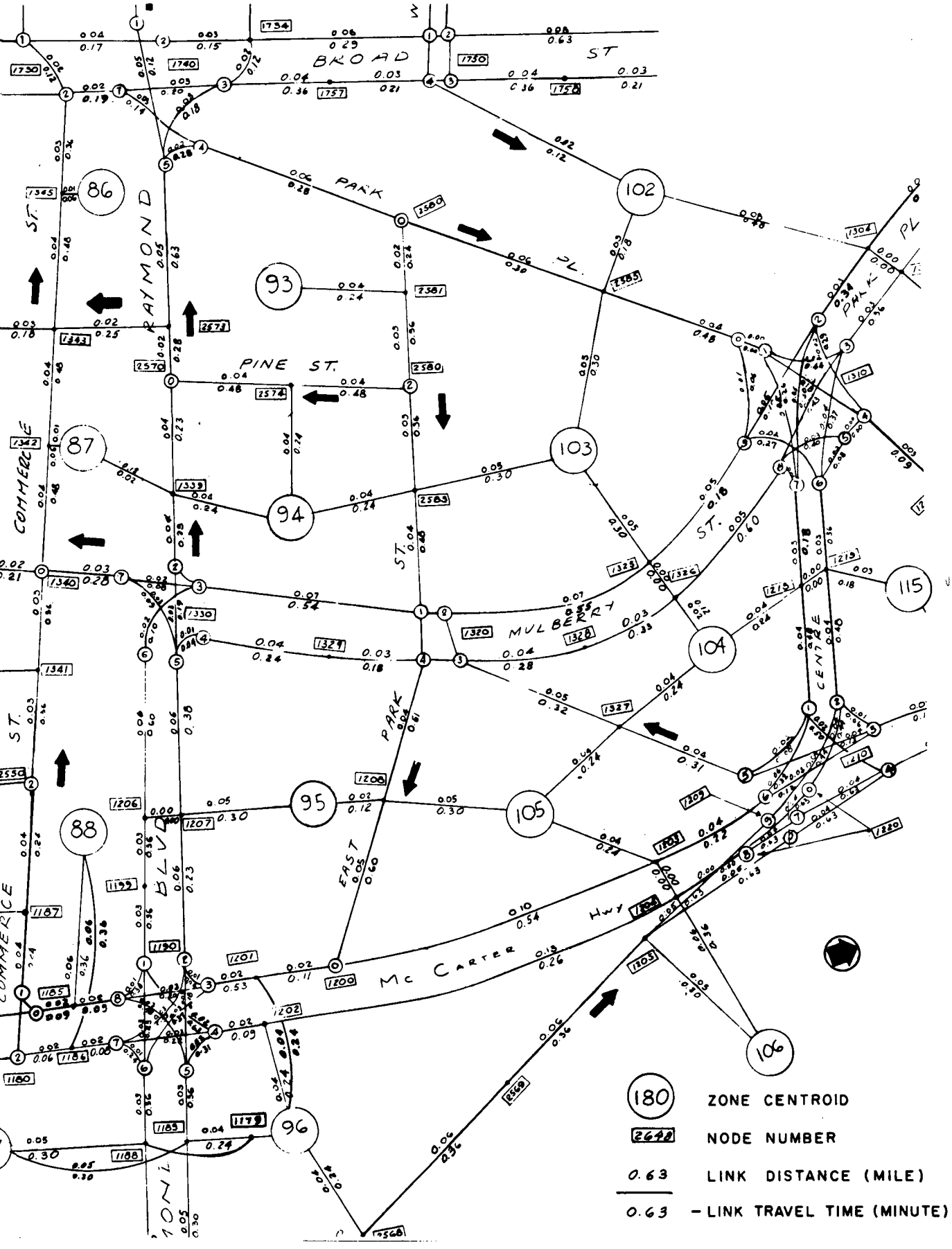


Figure 57. Newark coded network map, sample portion.


BROAD ST
PARK PL.
PINE ST.
RAYMOND ST.
COMMERCE ST.
MULBERRY ST.
PARK BLVD
EAST
Mc CARTER HWY
CENTRE
86
87
88
93
94
95
96
102
103
104
105
106
115
180 ZONE CENTROID
2648 NODE NUMBER
0.63 LINK DISTANCE (MILE)
0.63 - LINK TRAVEL TIME (MINUTE)

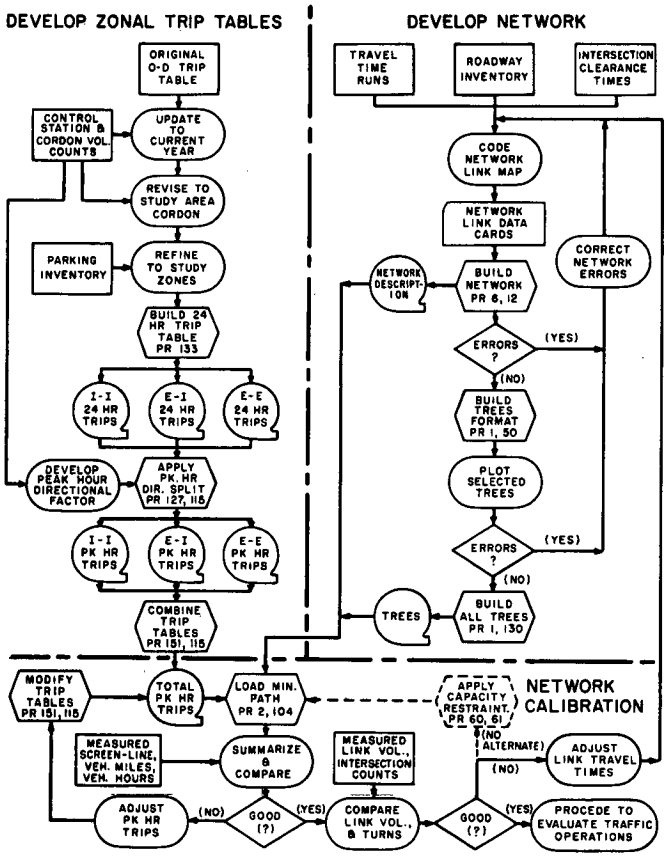


Figure 58. Network assignment flow chart.

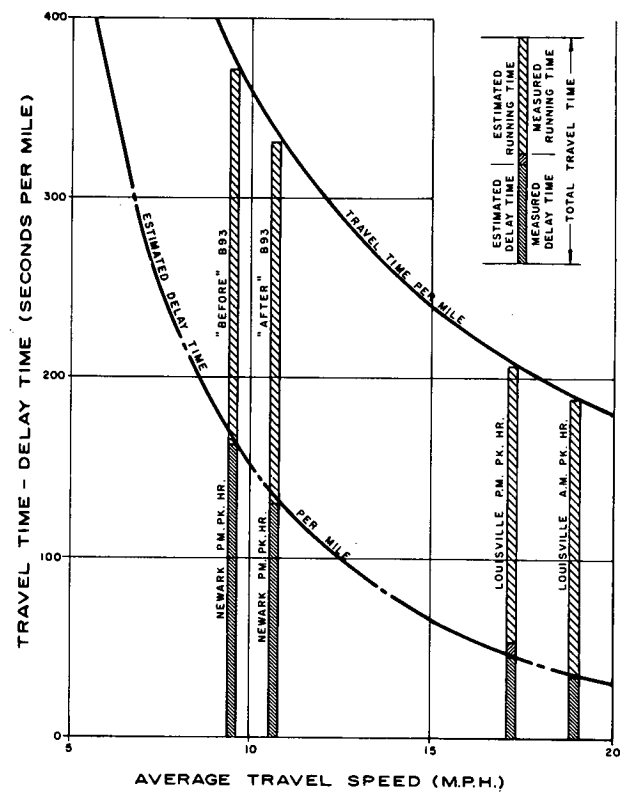


Figure 59. Network evaluation, travel time and delay time versus speed.

represented a maximum reduction of 108 veh-hr compared with the "before" 2,869 veh-hr. If there had been no traffic volume change, the minimum reduction would have been 94 veh-hr ("before" assigned volumes times the change in link travel times). The net saving for all attracted traffic is 108 minus 94, or 14 veh-hr. In Test 2 the assigned "after" travel was 2,872 veh-hr, an increase of 3 veh-hr. The increase would have been 5 veh-hr if there were no volume change; and the net saving for traffic diverted from Broad Street was 5 minus 3, or 2 veh-hr.

If the additional trips assigned to McCarter Highway had really used the roadway, the increase in volume would, no doubt, have resulted in an increase of travel time for all traffic on McCarter Highway. It must be remembered, however, that the assignment did not consider any decrease of travel time for the traffic remaining on the roadways from which the trips had been diverted. Similarly, the assignment did not consider any decrease in travel time

TABLE 26

NEWARK PM PEAK-HOUR NETWORK EVALUATION, EXPERIMENT B93, McCARTER HIGHWAY SIGNAL PROGRESSION

ITEM	"BEFORE"	"AFTER"— NO DIVERSION	CHANGE (%)	"AFTER"— FULL DIVERSION	CHANGE (%)
Vehicle-miles	27,846	27,846	0.0	29,699	+6.6
Vehicle-hours	2,869	2,775	−3.3	2,761	−3.8
Travel speed (mph)	9.7	10.0	+3.1	10.8	+11.3
Est. delay ratio	0.43	0.42	−2.3	0.39	−9.3
Est. vehicle-hour delay	1,205	1,165	−3.3	1,080	−10.4
Number of trips	27,931	27,931	0.0	27,931	0.0
Average trip length (miles)	0.997	0.997	0.0	1.06	+6.3
Average trip length (min)	6.16	5.96	−3.2	5.93	−3.7

for the traffic remaining on Broad Street in Test 2. If speed-volume relationships were approximately consistent for downtown roadways and all volume-capacity levels, the total assigned vehicle-hours would not change as a result of a capacity restraint balancing technique designed to obtain more realistic assignments; that is, the difference between assigned "before" and "after" vehicle-hours would represent an approximate network change, even though the assigned roadway volumes were inaccurate.

Because experimentation of this project has demonstrated that speed-volume/capacity relationships are not consistent for downtown roads, the usual capacity restraint techniques are not applicable to the downtown environment. The assignment model change in vehicle-hours of travel can be only an approximate estimation, representing a probable maximum saving or a probable minimum increase.

The network evaluation for Test 1 is summarized in Table 26. The changes for Test 2 (between 3 and 5 veh-hr) are too small to be significant when compared with network totals. The "before" values in Table 26 are from the network assignment 04; the "after" values—with no diversion—are from the assignment 04 volumes and the measured change in travel times on McCarter Highway. The "after" values—with full diversion—are from a reassignment with the updated travel times on McCarter Highway. The delay ratio and the vehicle-miles of delay were estimated from the computed average travel speed and the relationship between travel speed and delay time.

Figure 59 is designed to show two comparisons. First, the network evaluation for the Newark PM peak-hour roadway system "before" and "after" Experiment B93 is shown; and, second, Figure 59 compares Newark and Louisville network efficiency. Louisville's AM and PM peak-hour measured travel time and measured delay time are both used in making this comparison.

CHAPTER FOUR

FINDINGS—TRANSIT STUDIES

The street network of a city must serve many purposes. It must serve through and local trips by passenger car, bus, and truck; pedestrian movement; access to abutting properties; and delivery of goods. Therefore, in determining the effects of modifications to traffic operations for optimizing flow on existing street networks, it is necessary to consider the functional purpose of streets, the elements of the traffic stream, the street pattern, and the use of curb space. Because the movement of buses in the traffic stream and the operating characteristics of these vehicles are different from passenger cars, the effect of changes in traffic operations may be influenced considerably by buses. In almost every city there are some arterial streets where the volume of buses is a significant element of the total traffic flow.

Therefore, it is important to identify and measure the effect of those factors and conditions relating to the street network and traffic operations that influence bus movement.

SCOPE

Both study areas have extensive bus transit systems that were involved in several experiments for the description of the factors that influence bus operation in the downtown area. Work with buses, however, was somewhat restricted due to the need to obtain bus company approval as well as approval of the municipal officials for these experiments. Of course, investigations of doubtful benefit to the bus companies were not readily accepted. In addition, large-scale revisions, such as route changes, were not within the scope of the study. These would have required filing of formal application for approval by state officials. The changes, if approved, would then become permanent features of the bus operation and, therefore, could hardly be considered of a temporary and experimental nature. Bus routing should properly be assessed as part of the functional analysis of origin and destination information surveys that describe the public's preference for mode of transportation, the functional classification of streets, and other similar information usually developed as part of the urban area transportation study.

The investigations of this project were directed to the study of the optimum location for bus stops, passenger service operations, and organization of a major bus stop.

STUDY-AREA TRANSIT

Five bus companies operate a total of 282 buses in the Louisville study area (Table 27). Bus routes are located on most arterial and collector streets (Fig. 60). The largest company is the Louisville Transit Company, with a total of 216 buses operating on 14 separate routes, making 1,803 daily trips in the study area. The highest volume of 377 buses daily in one direction occurs eastbound on Market Street between Fourth Street and Third Street.

The recently completed transportation study found that in 1963, during the period from 3 to 6 PM, about 280 trips were scheduled through the study area, carrying an estimated 10,000 passengers.

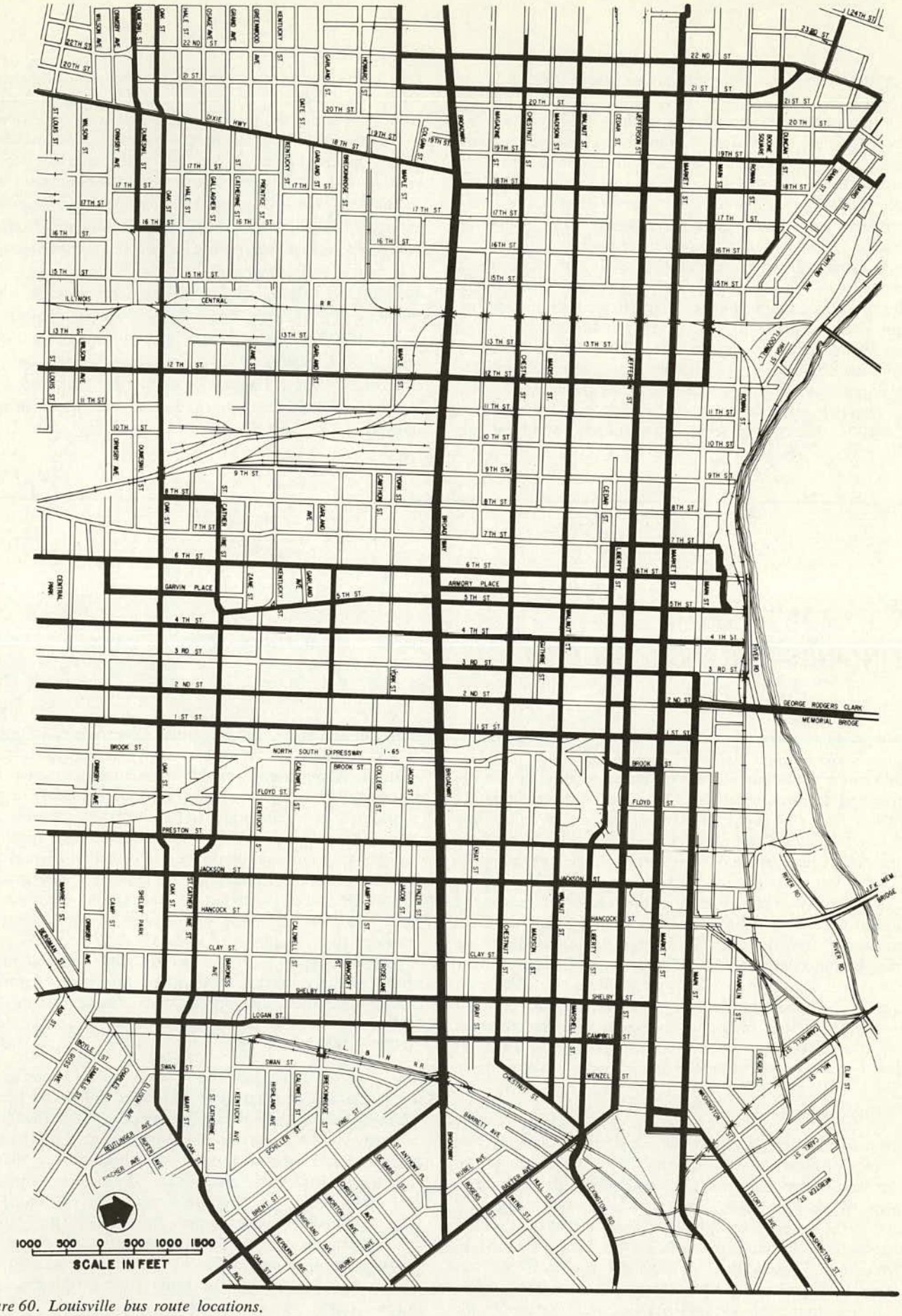


Figure 60. Louisville bus route locations.

TABLE 27

LOUISVILLE BUS TRANSIT OPERATIONS, MARCH 1968

BUS COMPANY	NO. OF BUSES	BUS CHARACTERISTICS LENGTH (FT)	BUS CHARACTERISTICS SEATS	NO. OF DESIGNATED ROUTES	DAILY TRIPS IN	DAILY TRIPS OUT
Louisville Transit Co.	4		31			
	10		36			
	50		40			
	20		45	14	915[a]	888[a]
	25		51			
	107	40	53			
Daisy Lines	25	35	39	1	48	49
Kentucky Bus Lines	20	35	41–45	3	25	25
Blue Motor Coach Co.	12	35	45			
	4	30	37	5	111	111
Bridge Transit Co.	5	35	45	2	68	68
All	282	—	—	25	1,167	1,141

[a] Includes 39 single-direction trips of Donwtown Shuttle Bus.
Note: Total passengers allowable equals 135% of seated capacity.

TABLE 28

BUS TRANSIT OPERATIONS, NEWARK STUDY AREA, JULY 1968

BUS COMPANY	NO. OF BUSES	NO. OF DESIGNATED ROUTES	DAILY TRIPS IN	DAILY TRIPS OUT
American Bus Co.	10[d]	1	10[e]	10[e]
Boro Buses	62	1	13	13
Bridgeton Transit	23	1	3	3
Browell Bus Co. (No. 26)	1	1	30[e]	30[e]
3 Central Ave. independents (No. 24)[a]	3[e]	1	53	53
Clinton Avenue Bus Co. (No. 16)	6[d]	1	30[e]	30[e]
Consolidated Shore Lines	25[d]	1	20[e]	20[e]
DeCamp Bus Lines	143	1	34	35
Graope Transportation Co. (No. 22)	2[e]	1	20	20
Mountain Coaches	9[d]	1	5[e]	5[e]
3 Newark-Elizabeth independents (No. 12)[b]	3[e]	1	40	44
New York-Long Branch Bus Line	10[d]	1	13	16
6 North Newark independents (No. 18)[c]	6	1	90[e]	90[e]
Public Service Coordinated Transport	2,537	84	3,739	3,735
Somerset Bus Co.	90	1	144[e]	144[e]
9 South Orange independents (No. 31/32)	13	1	161	161
Will Morris (No. 38)	2[d]	1	10[e]	10[e]
Total	2,945[e]	100	4,415[e]	4,419[e]

[a] Martucci Bus Co., Wohlgemuth Bus Co., E. Vanderhoff & Sons.
[b] E. Gersekowitz, William Seidler, Ben Hughes.
[c] A. Gersekowitz, Irving Transit Co., Patsy Palangio, John A. Palicastro & Co., Timothy J. Ryan, Van Buren Bus Co.
[d] Authorized by Public Utilities Commission.
[e] Estimate.

There are 34 bus companies operating an estimated 2,945 buses in the Newark study area (Table 28). The largest company is Public Service Coordinated Transport, with a total of 2,537 buses, operating 84 separately numbered routes (Fig. 61), making 7,474 daily trips in the study area. The highest daily volume of 1,979 buses in one direction occurs northbound on Broad Street between Clinton and Commerce Streets.

The Newark Transportation Study of 1961 found that an estimated 2,000 buses leave the CBD daily during the period from 2 to 6 PM. During the period from 3 to 6 PM some 51,000 passengers use bus transit facilities, with approximately 30,000 of these passengers outbound.

Figure 61. Newark bus route locations.

FACTORS INFLUENCING TRANSIT OPERATIONS

During the studies of transit operations, analysis of transit data and design, and implementation of experiments, many factors influencing transit operations were observed. Although these factors may not have been the subject of major interest in the experiments, they were often part of the design considerations or elements of obvious importance that are worthy of special notice.

Street Design and Physical Features

The street pattern itself may impose severe limitations on bus operations. The number of streets suitable for bus use may be limited, presenting difficulties in circulation and developing concentrations of buses on a few streets. In Newark, the pattern of radial arterial streets superimposed on a grid of collector distributor roadways imposes such limitations. Channelization of difficult intersections that result from this street configuration must consider the needs of bus movement, bus stops, and passenger transfers. Channelization that considers only the traffic flow may be seriously deficient in providing for these needs. A channelized intersection that does not provide adequately for transfers between bus routes may induce additional pedestrian interference as passengers cross through the channelized area to effect their transfer.

The curb-lane condition and curb height are factors that may prevent the bus from using the curb lane efficiently. In various experiments conducted on Central Avenue in Newark it was noticed that poor pavement severely restricted the use of the curb lane. Resurfacing of the street without resetting drain inlets had resulted in a severe depression at each inlet. Bus drivers avoided the use of this area of pavement both because of this condition and because of debris in the gutters. Vehicle placement in the curb lane was significantly affected by these conditions. Curb height is an important factor in the use of loading areas. Curbs should be from 6 to 9 in. high for proper operation. If curbs are too high, the bus will be prevented from moving close to the curb to load and discharge passengers. In Experiment F53, conducted at the intersection of Fourth and Liberty Streets in Louisville, asphalt pavement was placed in a far-side stop area, decreasing the curb height and lessening the pavement cross slope to improve the area for use as a bus stop. On Jefferson Street in Louisville, a severe cross slope in the curb lane in the area near Brook Street obstructs use of this lane for the loading and unloading of buses. An inadequate curb return radius at an intersection where buses must turn may also severely impede the use of adjacent areas for bus stops, forcing buses to stand out in the pavement to perform their passenger service operations.

Transit Operating Characteristics

Bus movements are characterized by frequent stops to load and discharge passengers, numerous accelerations and decelerations, lateral movements in and out of bus stops, and the need to follow prescribed routes. All these elements influence the traffic stream. To optimize bus movement is to minimize the adverse effect of these operational characteristics on the traffic stream. Careful study of route characteristics can establish the best location for bus stops. If the running time between stops and the average amount of time required for passenger service operations are known, the relationships that exist between stop locations and signals become apparent. The direction of cross-street traffic flow at the intersection and the parking conditions that prevail adjacent to the stop also influence the stop location.

In designing the bus operation, it is necessary to determine the average time spent in each bus stop for passenger service operations based on studies similar to those performed in Experiment F63. Then the time pattern of bus movement must be determined with respect to the time pattern established by the signal for progression. Significant advantages for near-side or far-side bus stops at signalized intersections may be evolved from such a study, inasmuch as the incidence of signal delay is a controlling factor. Not only can the best stop location be selected, but signal timing may also be adjusted to provide for bus requirements. In conducting Experiment B88 on Springfield Avenue in Newark, the effect of signal offset patterns, appropriate for average and peak traffic periods, was to substantially expedite bus movement as well as general traffic. This was confirmed by measurements made by the bus operators.

Exclusive bus lanes may be advantageous in congested downtown areas. Considering the usual persons-per-vehicle ratios, bus volumes of between 35 and 50 per hour, depending on loading, may be considered to be equivalent to a full lane of passenger cars and therefore justify consideration of the use of an exclusive bus lane. Of course, this is dependent on many other conditions. The remaining lanes must be adequate to provide for necessary movement; curb lanes must be cleared of parking and loading operations; proper signs and markings must be designed and made available; and all necessary regulations must be adequately enforced.

Adequate bus stops are essential to proper bus operation. In downtown Newark it has often been observed that bus operators, on arrival, find the stop location already occupied by several other buses and of necessity will discharge passengers in the second lane. Alighting or boarding passengers must therefore walk between the buses in the curb lane, creating an extremely hazardous condition. To avoid such situations, the entire curb lane of downtown arterials where bus volumes are large should be made available for bus loading and unloading during peak periods. If it is found that frequently more than three buses of different routes desire to occupy the same bus stop, consideration should be given to establishing more bus stops, using near-side, far-side, and mid-block locations as necessary. Available stop locations can be organized in accordance with procedures described under "Organization of Bus Stops," which describes the work of Experiment C110 on Market Street in Newark. Prior to this experiment it was observed that passengers waiting at a near-side bus stop would walk to a second or even third bus delayed by other buses already engaged in passenger service operations. When more than three buses were simultaneously engaged in loading from the same lane, passenger service became very erratic. Some operators properly kept the bus door closed

and moved into the loading position before accepting passengers, but others opened the door and loaded passengers who had walked back to this location. It was necessary to repeat the loading operation a second time for those passengers who had not walked back to the previous position. Of course this condition occurs only in extremely dense bus environments. On Market Street in Newark bus volumes were approximately 100 per hour during the peak afternoon period.

Pedestrian Interference

The location of pedestrian crosswalks and proper pedestrian controls can be extremely important elements of traffic operation both for bus movement and for general traffic. In conducting Experiment A7 on Raymond Boulevard and McCarter Highway in Newark, pedestrian controls were installed and coordinated with the traffic signals to keep the crosswalks clear during advanced green intervals for left-turning vehicles. Initially, pedestrian violations of these controls were so numerous that they severely impeded traffic flow. When cross-street traffic was stopped, pedestrians would immediately begin to cross the street, conflicting with left-turning vehicles. Vehicles stopped at the crosswalk, in turn, impeded through movement at the end of the brief advance phase. Traffic movement was stopped entirely until the pedestrians cleared the crosswalks and normal operation could be resumed. Because of the serious effect these violations had on moving traffic, police enforcement was instituted. Police at this intersection made a definite attempt to educate pedestrians to obey the signals. After a brief period, a substantial reduction in the number of violations was observed. However, the effect was only temporary. One week after enforcement was withdrawn, pedestrian violations increased to the point where they were again a serious impediment to traffic flow. Many buses use this intersection. These buses were seriously delayed by the pedestrian interference. It was reported that delays were substantially reduced during the period of police enforcement.

Enforcement of Traffic Regulations

Enforcement of traffic regulations is extremely important for efficient bus operation. Parking, standing, stopping, double parking and truck loading that interfere with bus movement should be investigated, regulated if necessary, and the regulations should be strictly enforced. At the intersection of West Market Street and Central Avenue in Newark, illegal parking in a bus stop forced bus operators to perform passenger service operations in the only remaining eastbound lane for through traffic. This created an unsafe condition and completely stopped all traffic in this direction until the buses moved on. Illegal parking of this type can seriously diminish the potential for improvements and must be brought under control as an integral component of the improvement program, if it is to be successful.

These observations, made during experimentation in the study areas, indicate the large potential that exists for improvements to bus operations. As traffic densities increase, it is almost inevitable that the center cities will become more and more dependent on mass transportation. The most flexible and most easily organized form of mass transportation is bus transit. Consideration of the relative importance of buses and autos in furnishing person trips indicates the importance that should be placed on expediting bus movement. In Newark, more people leave the downtown portion of the city at the end of each workday by bus than by all other modes of transportation. Transit operations gain even more importance when one considers the dependence of many families of lower income levels on this form of transportation. At the time of this writing, Newark is involved in a study designed especially to develop additional bus service between low-income districts and areas where employment for the "blue-collar" workers of these neighborhoods is available.

BUS STOP LOCATION

Two factors, found to influence the efficiency of bus stop operations, were compared for near-side and far-side bus stops in Experiment F64, which was conducted at eight bus stops in Louisville. These factors were the direction of cross-street traffic flow and whether parking was permitted or prohibited adjacent to the bus stop. Because traffic signals create a large portion of the delay at bus stops in a downtown area, this element of delay was given special consideration. Buses entering near-side stops are subject to traffic-signal delay when approaching the stop in a queue or after the passenger service operation has been completed. They are also subject to delays from traffic waiting in the bus stop area to make a right turn and from through or right-turning traffic in the adjacent lane. Of course, under conditions where cross-street traffic is from right to left, the element of right-turning traffic is eliminated. Buses entering far-side stops are subject to signal delay at the approach to the intersection and to delays occasioned by moving traffic in the adjacent lane when leaving the bus stop. Delays due to traffic in the adjacent lane are substantially eliminated if parking is prohibited in the curb lane, keeping this lane open for bus movement.

In conducting Experiment F64, bus stop operation time was measured between limits approximating the points of beginning deceleration for a bus moving into a near-side stop and completing acceleration for a bus moving out of a far-side stop. Field observations indicated that this was approximately 155 ft before the stop line at the intersection and 120 ft beyond the far-side crosswalk. These limits were accepted for establishing cordons within which bus stop operation time would be measured. To eliminate the effect of passenger service operations, this element was measured separately and excluded from the comparison of bus stop operations. Physical details of the intersections at which these measurements were made are shown in Figures 62 through 69. The number of observations under each condition is given in Table 29. On several occasions bus operators deliberately delayed completion of the bus stop operation, probably to maintain schedule. When this was observed, the "stalling" delay was measured and the bus stop operation time was adjusted to eliminate the influence of stalling.

The effect of signal delay was analyzed, using data

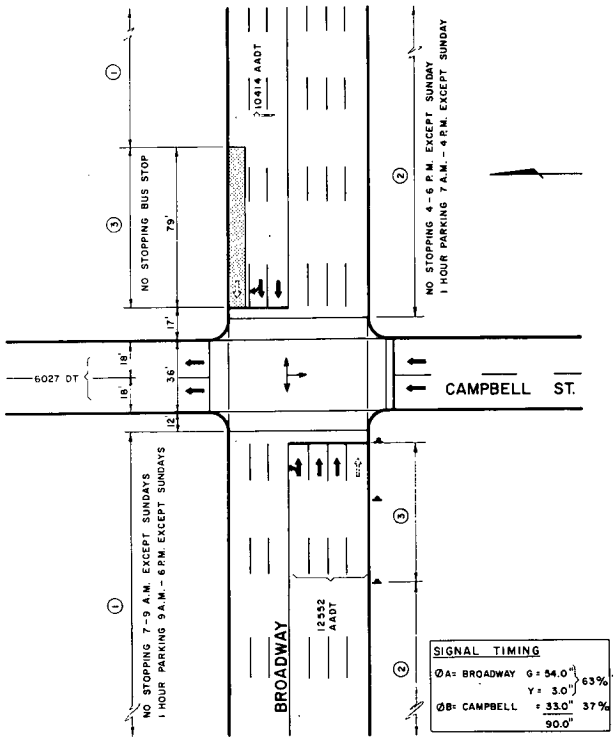


Figure 62. Optimum bus stop location, westbound Broadway at Campbell Street.

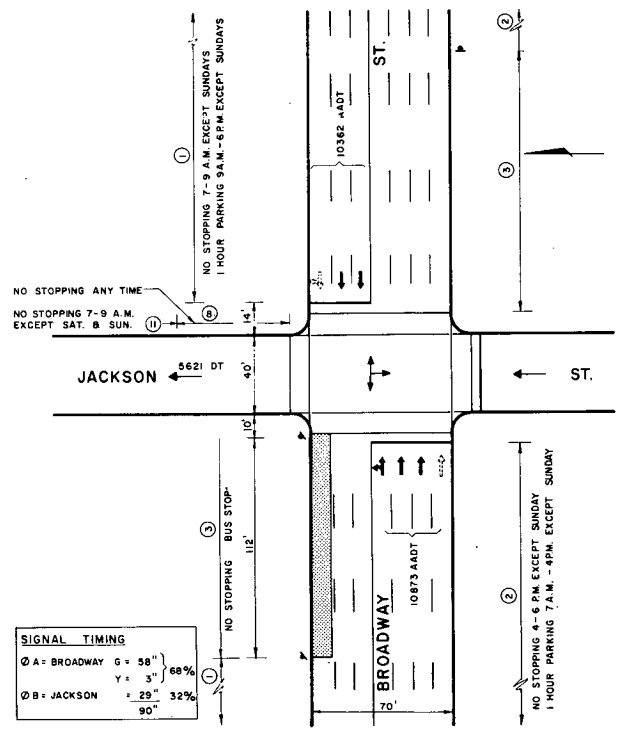


Figure 63. Optimum bus stop location, westbound Broadway at Jackson Street.

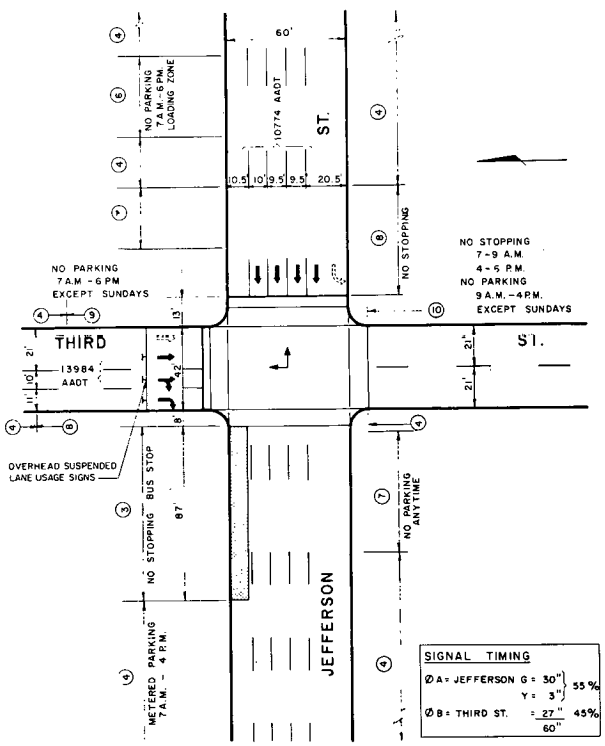


Figure 64. Optimum bus stop location, westbound Jefferson Street at Third Street.

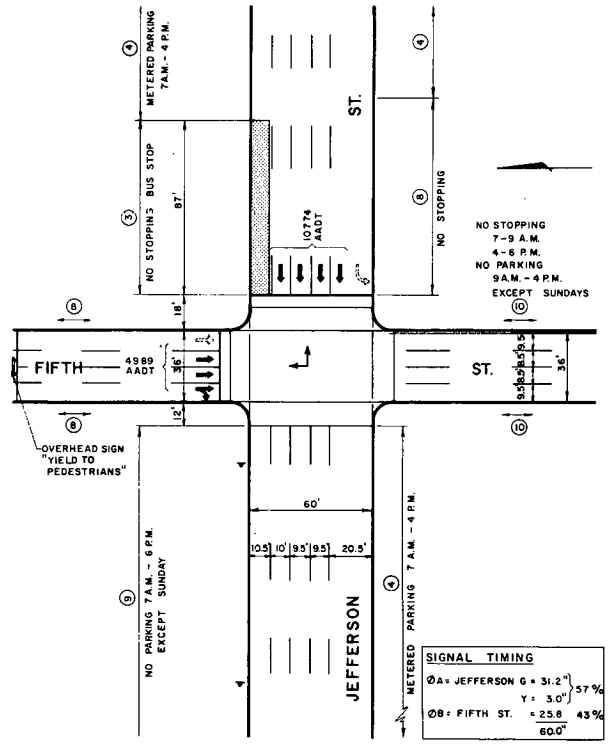


Figure 65. Optimum bus stop location, westbound Jefferson Street at Fifth Street.

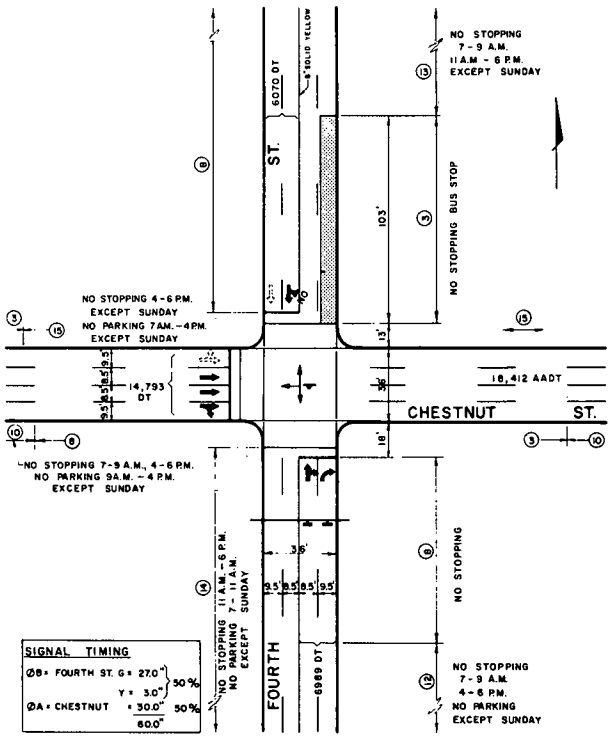


Figure 66. Optimum bus stop location, northbound Fourth Street at Chestnut Street.

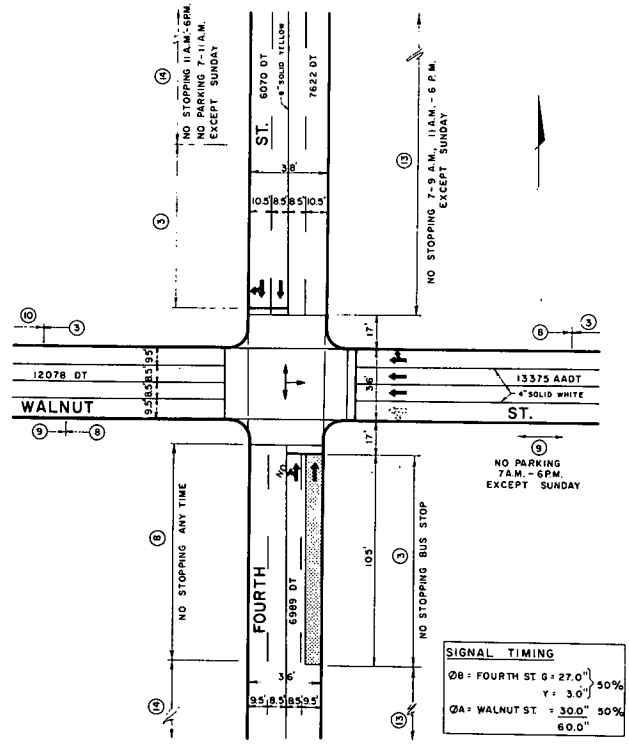


Figure 67. Optimum bus stop location, northbound Fourth Street at Walnut Street.

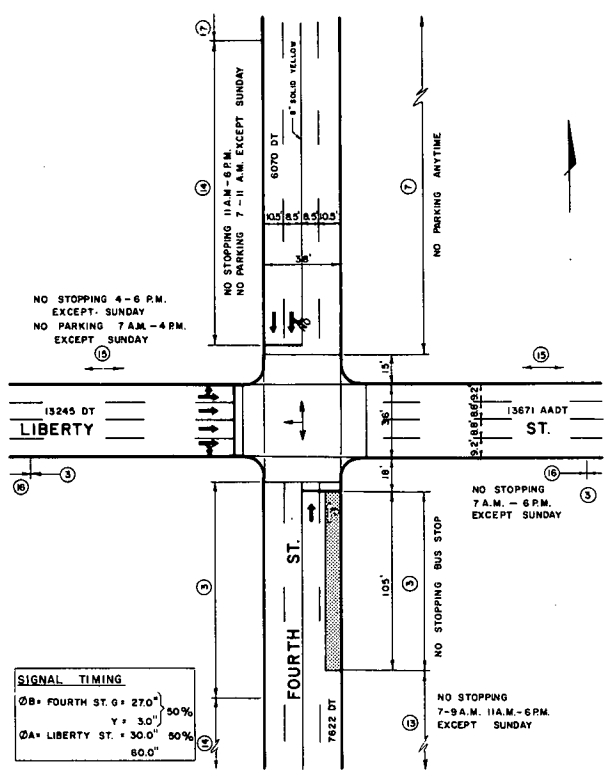


Figure 68. Optimum bus stop location, northbound Fourth Street at Liberty Street.

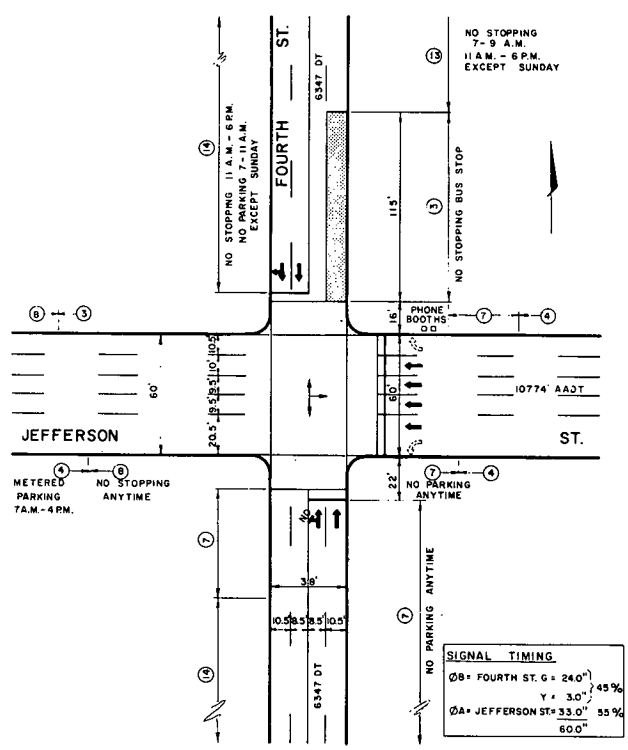


Figure 69. Optimum bus stop location, northbound Fourth Street at Jefferson Street.

TABLE 29

EXPERIMENT F64, OBSERVATIONS OF BUS STOP OPERATIONS, TYPE AND FREQUENCY OF DELAYS OBSERVED

	PARKING: PERMITTED												PARKING: PROHIBITED											
	CROSS TRAFFIC: LEFT TO RIGHT								CROSS TRAFFIC: RIGHT TO LEFT				CROSS TRAFFIC: LEFT TO RIGHT								CROSS TRAFFIC: RIGHT TO LEFT			
	NEAR SIDE				FAR SIDE				NEAR SIDE				FAR SIDE				NEAR SIDE				FAR SIDE			
TIME PERIOD	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*
	WB BROADWAY AT CAMPBELL				WB BROADWAY AT JACKSON				WB JEFFERSON AT FIFTH				WB JEFFERSON AT THIRD				NB FOURTH AT LIBERTY				NB FOURTH AT CHESTNUT				NB FOURTH AT WALNUT				NB FOURTH AT JEFFERSON			
AM									9	9	1	0	6	0	1	0																
Midday	12	5	0	1	9	3	0	0	8	7	0	0	10	1	0	0	12	12	2	0	7	5	1	0	10	8	0	1	21	1	1	7
PM	18	6	0	5	13	2	0	0	5	3	0	2	4	1	0	0																
All	30	11	0	6	22	5	0	0	22	19	1	2	20	2	1	0																
																	WB BROADWAY AT CAMPBELL				WB BROADWAY AT JACKSON											
AM																	10	5	0	1	14	1	0	1								

O = observations, total;
S = signal delay, number of occurrences;
T = traffic delay, number of occurrences; and
ST = "stalling" delay, number of occurrences.

collected at the Campbell Street and Jackson Street intersections with Broadway. From these data it was determined that bus stop operation time was approximately 43 percent greater for near-side locations than for far-side locations if signal delay was not present. If signal delay was present, total time was approximately 14 percent greater for the far-side stop location. This result is logical because at a near-side location a portion of the signal delay may be used for passenger service operations. Obviously, if the incidence of signal delay can be predicted through study of the bus's time relationship to the signal progression, this information can be used to advantage in selecting stop locations.

Conclusions reached by the statistical analysis of observed data are:

1. Bus stop operation time was observed to be greater at near-side locations, regardless of the direction of cross-street traffic and/or whether parking was permitted or prohibited. However, under the conditions of cross-street traffic left to right and parking prohibited, the differences were found to be not statistically significant.
2. A trend toward more frequent occurrence of signal delay at near-side stop locations was observed.
3. When signal delay occurred, total delay experience was greater for far-side stop locations.
4. When no signal delay occurred, total delay experience was greater for near-side stop locations.

A far-side stop location has a definite safety advantage, because the bus does not block pedestrians' view of approaching traffic from the crosswalk. Also, pedestrians do not conflict with the movement of the bus out of the stop location. These pedestrians may include alighting passengers who must walk in front of the bus at a near-side stop location but may pass safely behind the bus at a far-side stop, where their ability to see approaching traffic is not impaired and where they do not interfere with further movement of the bus.

Experiment F51, Relocated Bus Stop—Broadway and Campbell Street, and Experiment F53, Relocated Bus Stop —Fouth Street and Liberty Street, further investigated the relative efficiency of near-side and far-side bus stops. In both experiments, near-side bus stops were relocated to the far side. In Experiment F51, only minor changes in bus stop operation time were measured. Of these changes, only one was considered to be statistically significant. This change indicated an increase in bus stop operation time at the far side when signal delay was present. This finding agrees with the conclusions reached in Experiment F64. The lack of significant change for other conditions measured cannot be fully explained but may be due in some part to the light volume of right-turning vehicles at this location. The interference of right turns is a major factor in delays experienced at near-side stops.

In Experiment F53, the "after" measurements were made under two conditions. Phase 1 consisted of the basic relocation of the bus stop from its near-side position to the far side. Phase 2 measured the additional effect of installing overhead signs, curb-mounted signs, and pavement arrows to indicate that the curb lane was a mandatory right-turn lane. During Phase 1, Louisville Transit Company officials did not permit buses to use the left lane to bypass right-turning vehicles. They were concerned with safety and

issued this order to prevent collisions between buses entering the far-side position and other vehicles proceeding through the intersection in the curb lane. During Phase 2, this condition was controlled by the provisions for a mandatory right turn from the curb lane. In addition to the measurements of bus stop operation time, which were similar to those made for Experiment F64, travel-time data were obtained by speed and delay runs made on Fourth Street between Broadway and Main Street and also for right-turning vehicles traveling between Lines A and C, as shown in Figure 70. These measurements were designed to show the effect of the modifications on the general traffic stream. Measurements were made in three data groups. Data group 1 measured the "before" condition; data group 2 measured the conditions after moving the bus stop to a far-side location; and data group 3 showed results after implementation of the mandatory right-turn regulations. Measurements were taken in four time periods —the AM peak, between 7:30 and 9 AM; AM midday, between 9:30 and 11 AM; PM midday, between 2 and 3:30 PM; and PM peak period, between 4 and 5:30 PM. The results of these measurements are given in Tables 30, 31, and 32. The data in these tables indicate that travel time both for through and right-turning vehicles was generally improved and that the findings of Experiment F64 with regard to bus stop operation time are substantially confirmed.

PASSENGER SERVICE OPERATIONS

During the experimentation conducted in Louisville, many measurements were made of time required for passenger service operations. These measurements are analyzed in Experiment F63.

Passenger service operations are considered to include the alighting time for passengers leaving the bus and the boarding time for passengers entering the bus, and exclude all elements of bus movement in and out of the bus stop. Alighting time is defined as the interval of time measured from the moment the bus stops and the door has opened to the moment when the last alighting passenger has stepped from the bus, regardless of whether the bus door remains open or is closed. Boarding time is similarly defined as the interval of time measured from the moment the bus stops and the door has opened to the moment when the last passenger waiting to board has stepped onto the bus, regardless of whether the bus door remains open or closed and whether the last passenger boarding is able to move directly to the fare collection point. When passengers alight and board from the same door, boarding time begins at the end of alighting time. Excluded from these surveys were stragglers boarding the bus outside of the time period when the queue of waiting passengers boarded. Their inclusion would have introduced an unpredictable variable in the survey operation.

The factors that control passenger service operations include the number of alighting and boarding passengers, door use by alighting passengers, and methods of fare collection. Type of transfer activity and differences of passengers, such as commuters or shoppers, may have an additional, although probably minor, influence on the measurements. Human factors, such as age and physical impairment, were assumed to be uniformly distributed in the survey samples. Surveys were conducted during the period May 16 through May 29, 1968, inclusive. Passenger service operations were observed during AM peak periods, midday, and PM peak periods on a total of 467 buses. The range in time observed and recorded for passenger service was from 1 to 106 sec, with from 1 to 43 passengers being served. Figure 71 shows the range of observed passenger group size. Figure 72 shows the range of observed passenger service times.

It should be noted that, at the time of these surveys, the Louisville Transit Company charged a fare of $0.30 without zone limitations and with free transfer privileges between routes. Buses were operated on a "cash and change" system where fares could be paid, using any denomination of currency, and change would be made by the driver. Most buses were 40 ft in length and equipped with 53 seats. The maximum legal load was 72 passengers.

Separate analyses were made for the conditions of alighting only, boarding only, and combined boarding and alighting. The results of these analyses for each time period are given in Tables 33, 34, and 35. The general equations, which average the conditions for all time periods, were plotted and are shown in Figures 73, 74, and 75.

Rear-Door Alighting Only

To determine if a rear-door-alighting-only policy would result in reduced passenger service time, this situation was simulated. In this study, data for operations involving a front-door conflict between alighting and boarding passengers were revised by assigning the observed time for front-door alighting to the rear door. As a result of this simulation, which was applied to 233 observations, 164 showed a decrease, 63 an increase, and 6 no change in passenger service operation time. For all 233 observations, a total of 460 sec, or approximately 2 sec per observation, would have been saved. However, if a rear-door-alighting-only policy is to be used for all operations, obviously those that involve alighting only would require additional time. Investigation of such observations indicates that a total of 404 additional seconds of passenger service time would be required, substantially eliminating the benefit of this policy. If benefits of reduced passenger service time are to be obtained from a rear-door-alighting-only policy, it must be applied selectively to those situations where boarding and alighting conflict. Also, some means of achieving passenger cooperation and reducing confusion would be required. This might be assisted by means of a driver-operated, illuminated sign located in the front of the bus. Such a sign would permit selective application of this policy to particular bus stops or segments of routes where more passengers repeatedly board than alight, or to locations where the driver could assess the advantages in advance.

Exact Fare System

On November 10, 1968, the Louisville Transit Company changed its method of fare collection from a pay-enter, "cash and change" system to a pay-enter, "exact fare" system. In this latter system, the passenger is required to

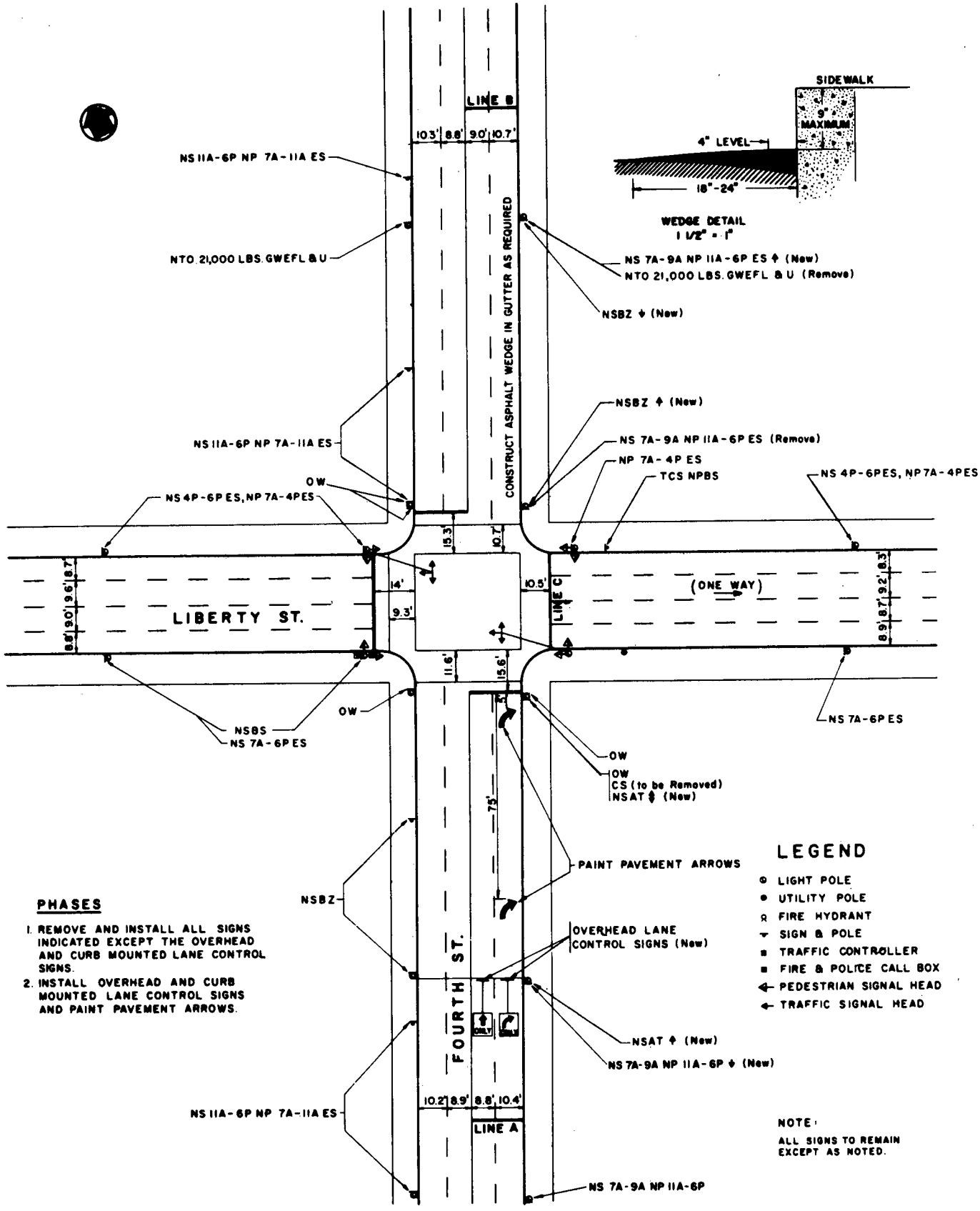


Figure 70. Vicinity map of relocated bus stop at Fourth Street and Liberty Street.

deposit the exact fare in a sealed box as he enters. If he does not have the correct fare, he can deposit a greater amount and receive script from the driver for the overpayment. The passenger can redeem this script at one of three locations. Presently, less than 0.1 percent of the total number of passengers receive script, according to Louisville Transit Company records. As an addendum to Experiment F63, surveys were repeated at the same locations for which data had been previously collected for the "cash and change" system. Regression equations were developed for

TABLE 30

EXPERIMENT F53, SPEED AND DELAY ANALYSIS, MEAN VALUES

TIME PERIOD	DIRECTION OF TRAVEL	MEAN VALUE (SEC), BY DATA GROUP 1	2	3	DIFFERENCE BETWEEN DATA GROUPS 1 AND 2	SIG. @ $\alpha=0.05$	2 AND 3	SIG. @ $\alpha=0.05$	1 AND 3	SIG. @ $\alpha=0.05$
AM	NB	161.5	174.1	176.8	12.6 (7.8%)	No	2.7 (1.6%)	No	15.3 (9.5%)	No
Midday		301.6	364.5	279.5	62.9 (20.9%)	Yes	−85.0 (−23.3%)	Yes	−22.1 (−7.3%)	No
PM		314.5	284.9	238.3	−29.6 (−9.4%)	Yes	−46.6 (−16.4%)	Yes	−76.2 (−24.2%)	Yes
AM	SB	285.2	319.1	288.1	33.9 (11.9%)	Yes	−31.0 (−9.7%)	Yes	2.9 (1.0%)	No
Midday		356.8	371.8	342.0	15.0 (4.2%)	No	−29.8 (−8.0%)	Yes	−14.8 (−4.1%)	No
PM		377.8	349.8	329.6	−28.0 (−7.4%)	Yes	−20.2 (−5.8%)	No	−48.2 (−12.8%)	Yes

TABLE 31

EXPERIMENT F53, TRAVEL TIME FOR RIGHT-TURNING VEHICLES, MEAN VALUES

TIME PERIOD	OCCURRENCE OF DELAY	MEAN VALUE (SEC), BY DATA GROUP 1	2	3	DIFFERENCE BETWEEN DATA GROUPS 1 AND 2	SIG. @ $\alpha=0.05$	2 AND 3	SIG. @ $\alpha=0.05$	1 AND 3	SIG. @ $\alpha=0.05$
AM peak	Yes	31.1	34.0	28.7	2.9 (9.3%)	No	−5.3 (−15.6%)	No	−2.4 (−7.7%)	No
	No	10.8	10.8	10.7	0.0 (0.0%)	No	−0.1 (−0.9%)	No	−0.1 (−0.9%)	No
AM midday	Yes	37.3	46.4	38.7	9.1 (24.4%)	Yes	−7.7 (−16.6%)	Yes	1.4 (3.8%)	No
	No	12.9	12.3	11.6	−0.6 (−4.7%)	No	−0.7 (−5.7%)	No	−1.3 (−10.1%)	No
PM midday	Yes	41.5	50.4	34.6	8.9 (21.4%)	Yes	−15.8 (−31.3%)	Yes	−6.9 (−16.6%)	Yes
	No	12.6	12.9	12.7	0.3 (2.4%)	No	−0.2 (−1.6%)	No	0.1 (0.8%)	No
PM peak	Yes	54.4	55.0	41.4	0.6 (1.1%)	No	−13.6 (−24.7%)	Yes	−13.0 (−23.9%)	Yes
	No	13.5	15.5	14.2	2.0 (14.8%)	No	−1.3 (−8.4%)	No	0.7 (5.2%)	No

TABLE 32

EXPERIMENT F53, BUS STOP OPERATION TIME, MEAN VALUES

TIME PERIOD	OCCURRENCE OF SIGNAL DELAY	MEAN VALUE (SEC), BY DATA GROUP 1	2	3	DIFFERENCE BETWEEN DATA GROUPS 1 AND 2	SIG. @ $\alpha=0.05$	2 AND 3	SIG. @ $\alpha=0.05$	1 AND 3	SIG. @ $\alpha=0.05$
AM peak	Yes	51.9	60.0	55.8	8.1 (15.6%)	Yes	−4.2 (−7.0%)	No	3.9 (7.5%)	No
	No	24.6	25.7	25.5	1.1 (4.5%)	No	−0.2 (−0.8%)	No	0.9 (3.7%)	No
AM midday	Yes	55.9	73.7	55.7	17.8 (31.8%)	Yes	−18.0 (−24.4%)	Yes	0.2 (0.4%)	No
	No	41.7	28.8	30.5	−12.9 (−30.9%)	Yes	1.7 (5.9%)	No	−11.2 (−26.9%)	Yes
PM midday	Yes	57.0	77.3	56.2	20.3 (35.6%)	Yes	−21.1 (−27.3%)	Yes	−0.8 (−1.4%)	No
	No	39.8	24.6	25.6	−15.2 (−38.2%)	Yes	1.0 (4.1%)	No	−14.2 (−35.7%)	Yes
PM peak	Yes	53.0	84.1	57.8	31.1 (58.7%)	Yes	−26.3 (−31.3%)	Yes	4.8 (9.1%)	No
	No	40.9	28.2	27.0	−12.7 (−31.1%)	Yes	−1.2 (−4.3%)	No	−13.9 (−34.0%)	Yes

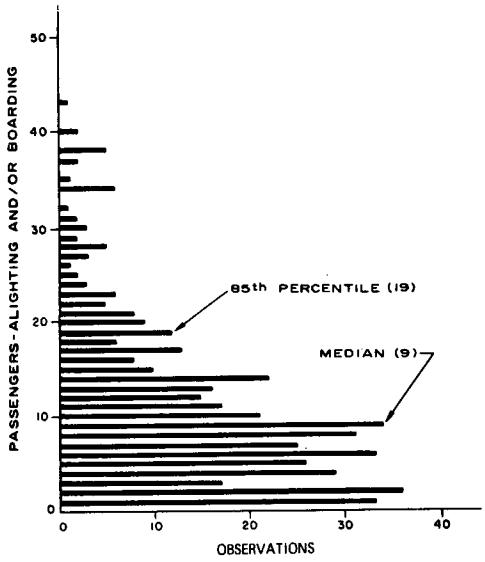


Figure 71. Range of observed passenger group size.

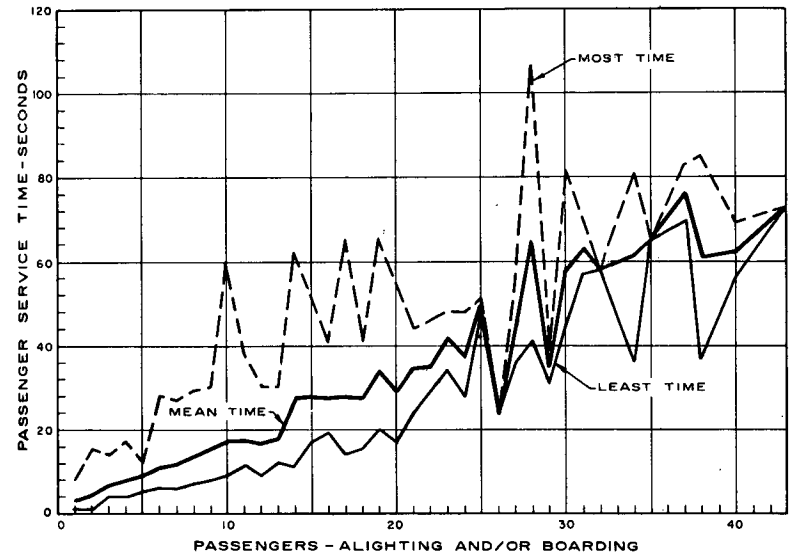


Figure 72. Range of observed passenger service times.

combined time periods for alighting only, boarding only, and boarding with alighting. These equations are given in Table 36 and are plotted in Figures 76, 77, and 78. Comparison of the "cash and change" system with the "exact fare" system indicates a saving of approximately 0.6 sec for each boarding passenger under the "exact fare" system.

TABLE 33

EXPERIMENT F63, REGRESSION EQUATIONS, PASSENGER SERVICE TIME, ALIGHTING ONLY, "CASH AND CHANGE" SYSTEM

TIME PERIOD	NO. OF OBS.	CORR. COEF.	STANDARD ERROR OF ESTIMATE	PREDICTIVE EQUATIONS FOR PASSENGER SERVICE TIME	ACCEPTABLE RANGE OF X_1
AM	27	0.90	2.245	$\hat{Y}=1.8203+0.9187X_1$	$1\leq X_1\leq 20$
Midday	71	0.89	3.096	$\hat{Y}=1.6067+1.2141X_1$	$1\leq X_1\leq 19$
PM	23	0.81	3.151	$\hat{Y}=2.0938+1.1725X_1$	$1\leq X_1\leq 17$
Total	121	0.87	3.069	$\hat{Y}=1.8437+1.1122X_1$	$1\leq X_1\leq 20$

X_1 = number of passengers alighting; and
$\hat{Y}$ = expected time required for passenger service (sec).

TABLE 34

EXPERIMENT F63, REGRESSION EQUATIONS, PASSENGER SERVICE TIME, BOARDING ONLY,[a] "CASH AND CHANGE" SYSTEM

TIME PERIOD	NO. OF OBS.	CORR. COEF.	STANDARD ERROR OF ESTIMATE	PREDICTIVE EQUATIONS FOR PASSENGER SERVICE TIME	ACCEPTABLE RANGE OF X_2
Midday	26	0.97	2.727	$\hat{Y}=0.2396+2.5288X_2$	$1\leq X_2\leq 14$
PM	12	0.97	3.183	$\hat{Y}=-0.6494+2.7169X_2$	$2\leq X_2\leq 16$
Total	41	0.97	2.907	$\hat{Y}=-0.0855+2.5855X_2$	$1\leq X_2\leq 16$

[a] Sufficient data for the AM period were not available for analysis.
X_2 = number of passengers boarding; and
$\hat{Y}$ = expected time required for passenger service (sec).

TABLE 35

EXPERIMENT F63, REGRESSION EQUATIONS FOR BUS PASSENGER SERVICE TIME, "CASH AND CHANGE" SYSTEM, BOARDING WITH ALIGHTING

TIME PERIOD	NO. OF OBS.	MULTIPLE CORR. COEF.	STANDARD ERROR OF ESTIMATE	PREDICTIVE EQUATIONS FOR PASSENGER SERVICE TIME	ACCEPTABLE RANGE FOR X_1 AND X_2
AM	43	0.89	3.850	$\hat{Y}=3.5985+1.0089X_1-0.0913X_1X_2+0.4653X_2^2-0.0215X_2^3$	$1\leq X_1\leq 22$ $1\leq X_2\leq 17$
Midday	191	0.94	6.383	$\hat{Y}=1.1762+1.3822X_1-2.3041X_2-0.0828X_1X_2+0.0013X_2^3$	$1\leq X_1\leq 21$ $1\leq X_2\leq 30$
PM	63	0.98	4.164	$\hat{Y}=0.4757+1.1987X_1+2.2614X_2-0.0423X_1X_2$	$1\leq X_1\leq 24$ $1\leq X_2\leq 36$
All	297	0.94	5.787	$\hat{Y}=1.7701+0.9727X_1+2.2756X_2-0.0234X_1X_2$	$1\leq X_1\leq 24$ $1\leq X_2\leq 36$

X_1 = number of passengers alighting;
X_2 = number of passengers boarding; and
$\hat{Y}$ = expected time required for passenger service (sec).

TABLE 36

EXPERIMENT F63, REGRESSION EQUATIONS FOR BUS PASSENGER SERVICE TIME, "EXACT FARE" SYSTEM

CATEGORY	MULTIPLE CORR. COEF.	STANDARD ERROR OF ESTIMATE	NO. OF OBS.	EQUATION FOR PREDICTING PASSENGER SERVICE TIME	ACCEPTABLE RANGE OF INDEPENDENT VARIABLES
I—alighting only	0.88	2.738	147	$\hat{Y}=2.2345+1.0792X_1$	$1\leq X_1\leq 26$
II—boarding only	0.97	2.702	31	$\hat{Y}=0.5863+1.9957X_2$	$1\leq X_2\leq 25$
III—boarding with alighting	0.91	6.556	359	$\hat{Y}=1.6043+0.9588X_1+2.154X_2-0.0202X_1X_2$	$1\leq X_1\leq 30$ $1\leq X_2\leq 33$

X_1 = number of passengers alighting;
X_2 = number of passengers boarding; and
$\hat{Y}$ = predicted time required for operation (sec).

ORGANIZATION OF A MAJOR BUS STOP

On sections of the downtown street system where heavy bus flows result in multiple use of bus stops, it may be necessary, for efficient operation, to extend the length of bus stops so that several loading positions are available. This need will probably begin to be apparent when directional average bus volume exceeds 40 per hour. Of course, the number of passengers to be loaded and the resulting time that each bus must spend in the bus stop also affect this relationship.

Experiment C110, conducted on Market Street in Newark, investigated the efficiency of various elements of bus stop organization where bus volumes were approximately 100 per hour during the afternoon peak period. During the surveys conducted before this experiment, as many as seven buses were observed to be engaged simultaneously in passenger service operations at the intersection of Broad and Market Streets. Initial observations revealed that buses spent nearly 3½ min traversing the block between Beaver and Broad Street, loaded an average of 17.3 passengers, with passenger service operations requiring an average of 65 sec. Allowing for the time required for normal movement through this area, it was estimated that over 1 min of delay time was being experienced by these buses due to operating conditions. Because of the extreme congestion of buses at this location, buses were permitted to load from two lanes. Buses in the second lane loaded passengers from a narrow area of pavement separated from moving traffic by portable barriers. In this operation, loading was confined to those buses that could be accommodated in the stop area on the near side of the Broad-Market Streets intersection. Buses were forced to wait for completion of passenger service operations by those buses ahead of them in the queue, with approximately three buses loading simultaneously in each lane during peak periods. Figure 79 shows a picture published in the *Newark Star-Ledger* on Thursday, December 20, 1956, which was taken at the inception of this operation. Prior to this time buses were confined to the curb lane, using only the near-side stop at Broad Street, and bus queues had often extended for several blocks as the queue of buses moved slowly up to the stop location. By comparison, the two-lane loading operation was considered to be a definite improvement.

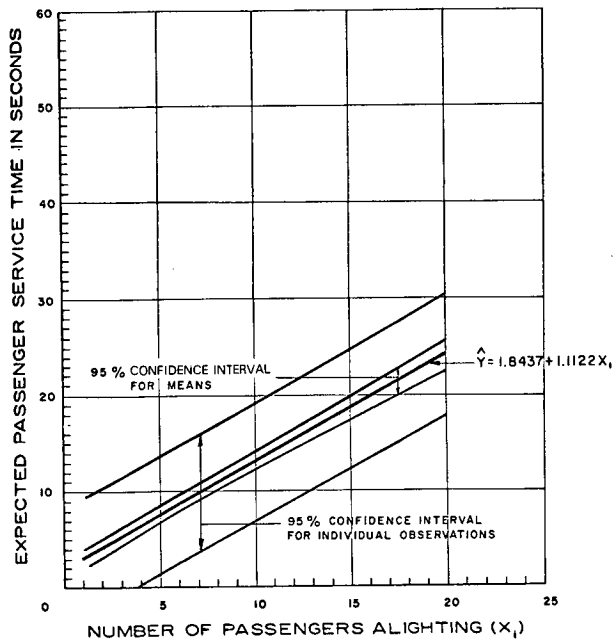


Figure 73. Expected passenger service time, alighting.

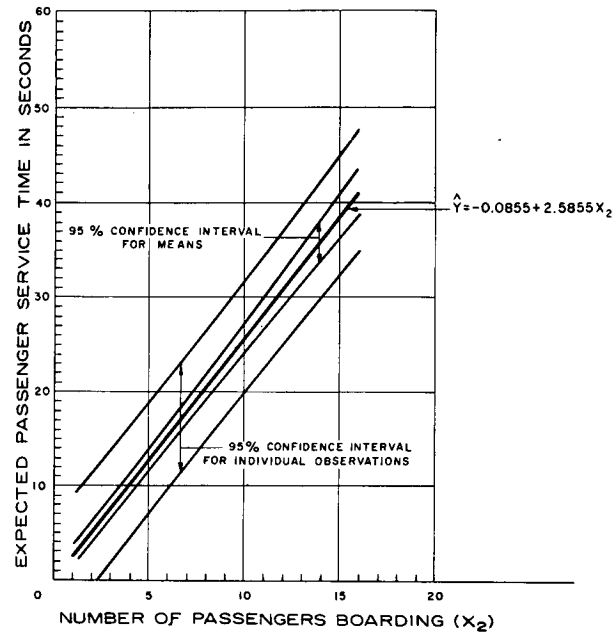


Figure 74. Expected passenger service time, boarding.

Part of the problem of getting officials of the bus company to agree to accept a plan for return to curb-lane-only loading was their fear of reverting to this initial situation.

Experiment C110 proposed that second-lane loading be abandoned and that the entire curb lane of 3½ blocks of Market Street, between Beaver Street and a far-side stop at Washington Street, be devoted entirely to bus loading. The second lane was proposed to be used for movement of buses through the area, bypassing those engaged in passenger service operations at other stop locations. Buses could thus proceed directly to their designated stop, perform their passenger service, and leave the stop without interference from other buses that might be simultaneously engaged in passenger service. Buses were allocated to stops that would group bus routes having the same community of interest at the same stops, and an attempt was made to divide buses equally between the available stop positions. Each bus route was assigned at least two stops in the 3½-block area. Field tests were performed to determine how much curb space would be required between stops. It was necessary to provide for two buses at each stop location and still have sufficient clearance behind the second bus so that the lead bus of the adjacent upstream stop could maneuver out of its loading position into the second lane. These tests checked initial layouts that had been made using data developed by Technical Committee 6-C of the Institute of Traffic Engineers (*9*). Signs designating the routes assigned to each stop position were constructed and permanently curb-mounted at the appropriate locations. Movable signs were made designating the exclusive bus use of the curb and second westbound lanes. Stop bars were painted in the curb lane to assist bus operators in properly positioning their buses. The final plan is shown in

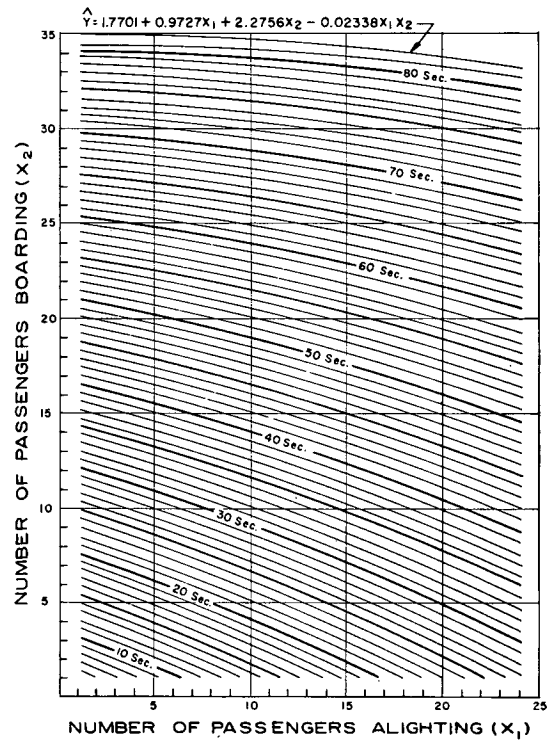


Figure 75. Expected passenger service time, boarding and alighting.

Figure 80. Figures 81 and 82 show Market Street and bus operations. The experiment was implemented on May 19, 1969. Prior to the implementation, notices were posted in

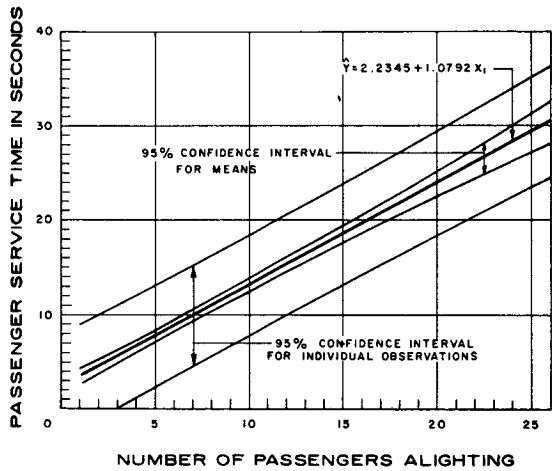


Figure 76. Expected passenger service time, "exact fare," alighting.

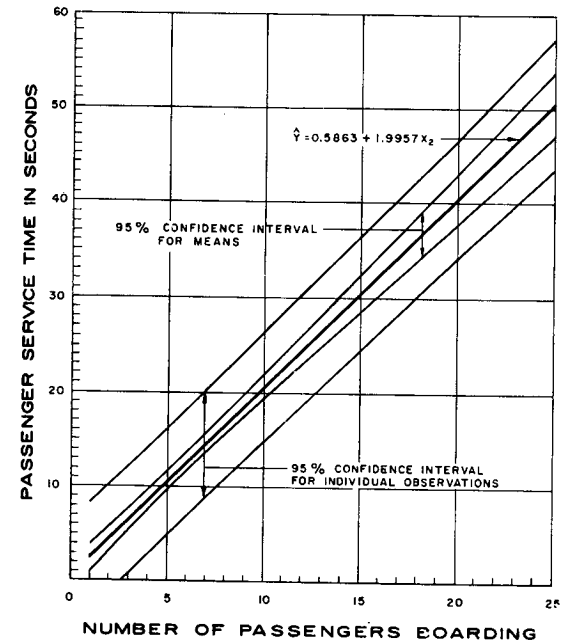


Figure 77. Expected passenger service time, "exact fare," boarding.

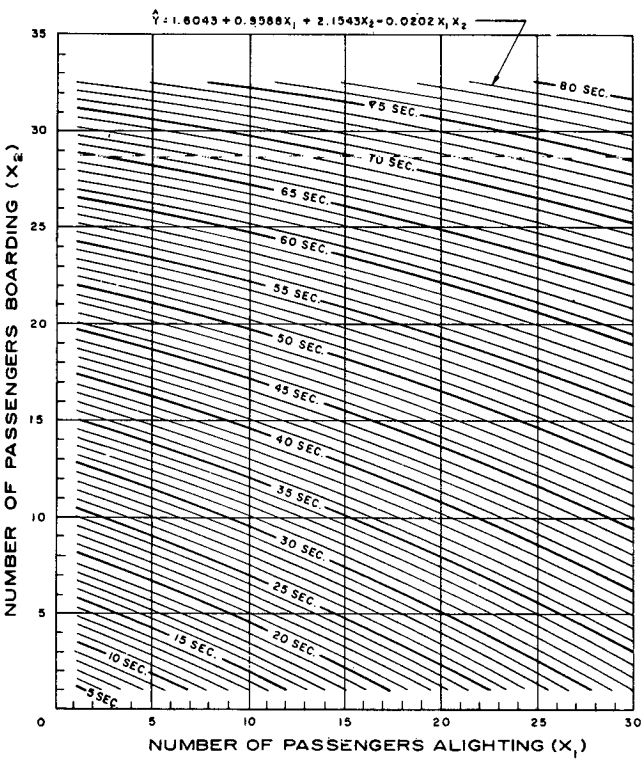


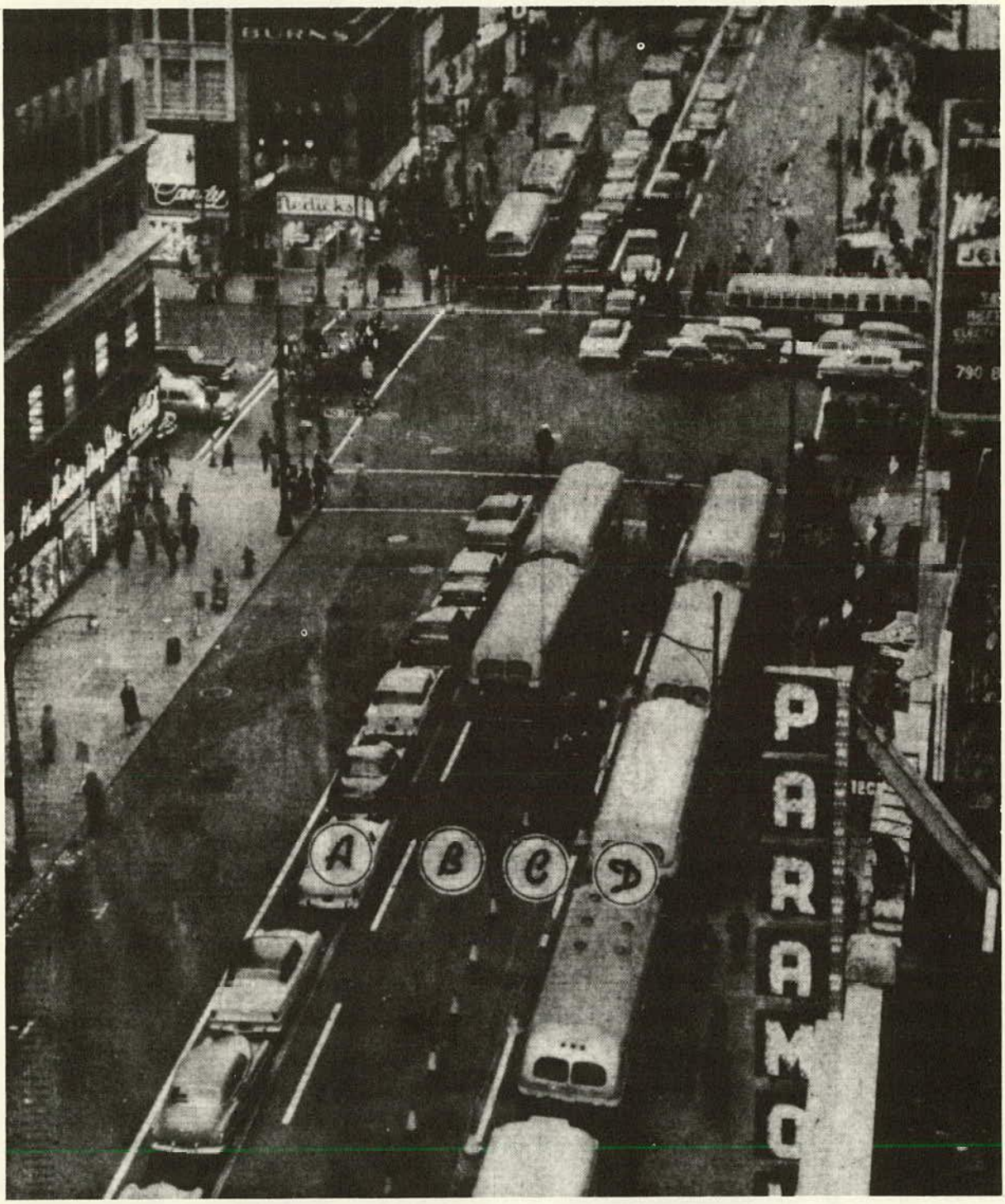
Figure 78. Expected passenger service time, "exact fare," boarding with alighting.

the buses, newspaper publicity was obtained, and during the initial days of the change supervisory personnel of the bus company assisted in educating both the bus drivers and the general public to these revisions. At 4 PM traffic engineers of the city of Newark assisted by placing the portable signs and a line of traffic cones between the opposing directions of traffic. These were all removed at 6 PM.

Public acceptance of the revised operations was quickly gained and comments overheard were generous in their praise of the changes. The bus company and city officials have accepted the revised conditions for permanent operation in the area and have indicated their satisfaction with the results of the study.

A survey, using time-lapse photography, was made with pictures taken at a rate of one frame per second continuously for the period from 4:30 to 5.30 PM for one day of the "before" and one day of the "after" conditions. The bus company, Public Service Coordinated Transport, surveyed bus running time between cordons at Beaver Street and Washington Street using their own personnel. The time-lapse survey indicated that average bus trip time for the 4:30 to 5:30 PM time period was reduced 36 sec. The bus company's survey, conducted over a longer period of time, indicated a larger saving of 55 sec per bus. The time-lapse photography study also indicated a reduction in westbound passenger-car trip time for this same period of 20.4 percent (25.5 sec). A significant decrease in the number of stops being experienced by passenger vehicles was also evident. Other advantages of the revised system were removal of the potentially hazardous bus loading areas in the center of the street and reduced concentrations of waiting bus passengers on the sidewalk. Although no measurements were made to quantify the benefits to pedestrians, it was very evident that their passage through these congested areas was considerably expedited.

The effectiveness of the revised operations is affirmed by the bus company's plan to remove two westbound buses while maintaining schedule for this time period. This saving is a sole result of the reduced turnaround time for the



MARKET ST. LANES—New traffic lanes on Market St. ease congestion during rush hours. "A" marks lane exclusively reserved for cars and trucks; "B" is outside bus lane; "C" marks pedestrian "safety zone" to use buses in "B" lane; "D" is second bus lane, for vehicles to be boarded from sidewalk.

NEWARK STAR-LEDGER Thursday, Dec. 20, 1956

Figure 79. Dual-lane transit operation publicity.

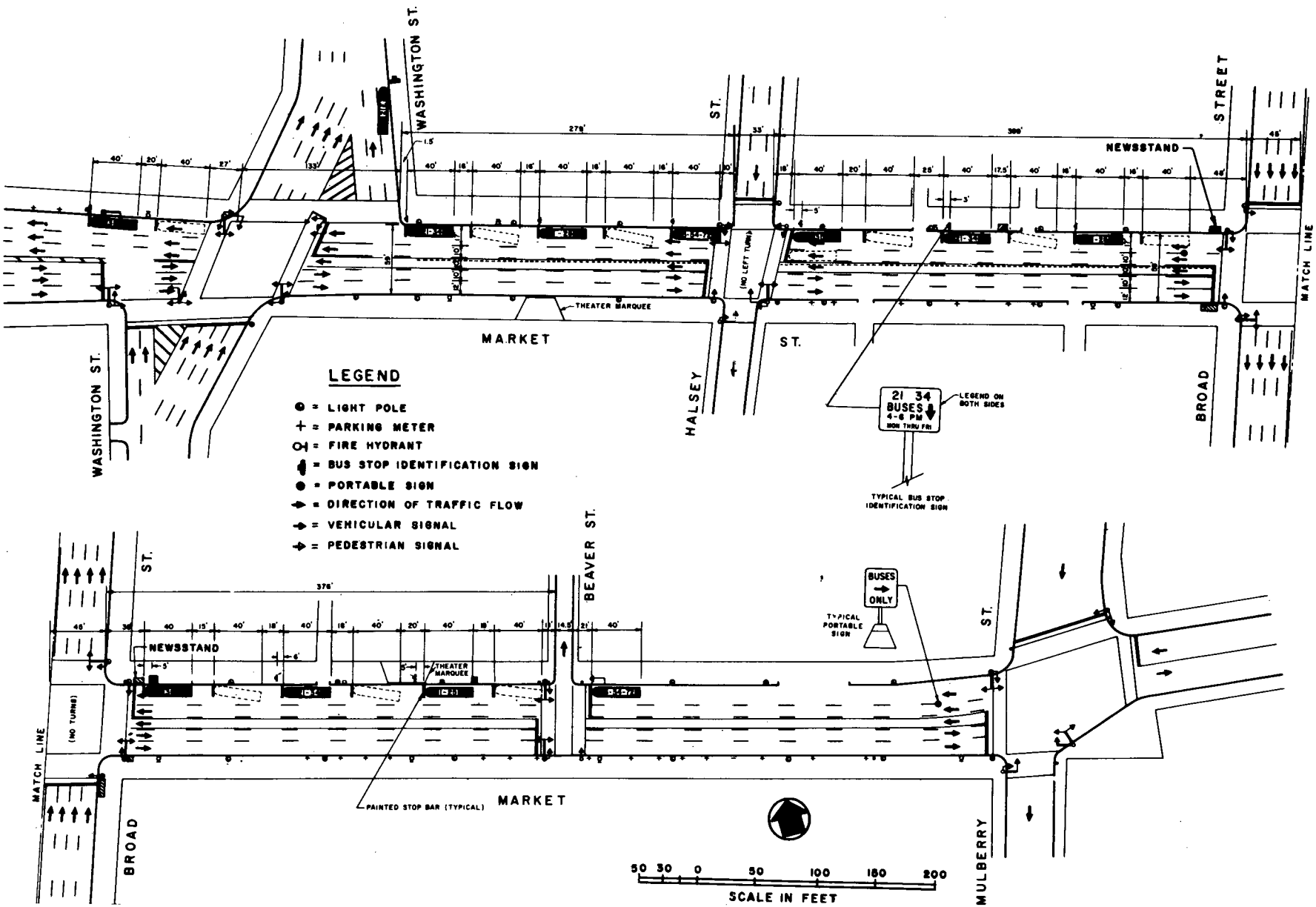


Figure 80. Experiment C110, vicinity map, "after."

Looking East From Halsey Street

Looking West From Halsey Street

Placing Cones For Additional Lane

Placing Signs For Bus Lane

Figure 81. Experiment C110, Market Street.

remaining buses. Also, they have stated their intention to consider similar operations for other areas of dense transit activity. The following quotation is taken from a letter to the Principal Investigator by J. T. Vrooman, Northern Division Manager:

> While the drastic change from our former practice of dual lane loading to the present system created considerable apprehension as to the success of the new system, it must be admitted that there is a marked improvement in the movement of all vehicles.

Experimentation conducted on this project indicates that the field of bus transit offers one of the most lucrative opportunities for expediting traffic flow in the downtown area. Surveys indicate that, during the peak period, buses on Market Street averaged more than 50 passengers per bus. Considering the 200 buses that move westbound during the 2-hr period, more than 10,000 people were accommodated by bus transit service on this one street alone. If these people had elected to use passenger cars, at current persons-per-vehicle ratios, this would be equivalent to the service provided by cars filling five lanes of city street. Allowing a saving of approximately 1 min per person, the time saved daily amounts to approximately 167 person-hr. Of course, opinions as to the monetary value of this saving

Passengers Queued at Sign

Passing Bus in Loading Zone

Figure 82. Experiment C110, bus operations.

could range widely. However, if a value of $2 per hour is allowed, the direct saving in a year would be approximately $86,800; and the capital value of the improvements, at an interest rate of 6 percent, could be set at more than $1,400,000. This does not include the operating benefits experienced by the bus companies or the benefits to other vehicles. The actual cost of these improvements was negligible, being only the cost of a few signs, a small amount of pavement marking, a brief period of education on the part of the bus company, and engineering.

CHAPTER FIVE

INTERPRETATION AND APPRAISAL

A very real consideration in the problems of downtown traffic is the multiplicity of services provided by the street system. While these services are largely oriented to transportation requirements of the downtown area, all of these requirements do not necessarily involve the expeditious movement of traffic. These include such items as the delivery of goods to commercial enterprises of the area, short-term parking, taxi stands, bus stops, pedestrian walkways, emergency services, mail distribution and pickup, maintenance work on public utilities, and a multitude of other essential services. Because of these many uses, changes made to the existing operational characteristics of the downtown street system may drastically influence aspects of the community's life other than those related directly to the movement of traffic. Control of impediments to traffic flow such as truck loading or parking may assume major proportions when all the political and economic facets of the problem are fully understood. Because of this intimate relationship between the life of the downtown area and its transportation network, it is essential that a complete and thorough functional analysis of the street system be performed. This analysis should precede any comprehensive optimization plan designed to expedite traffic flow. It should investigate all the secondary uses of the street system and integrate these uses with the primary function of moving traffic in order to diminish adverse effects on other activities of the community. For instance, in conducting Experiments D15 and D16, which involve the removal of curbside parking at the intersection of Oak and Shelby Streets in Louisville, it was found that no significant difference in traffic flows was caused by this change, even though a 50-percent increase in capacity at the Oak Street approach and a 20-percent increase on the Shelby Street approach were predicted by capacity analyses. Such a change, requiring the community to give up parking spaces while creating no significant betterment in flow characteristics, is obviously not in the best interests of the community. The functional analysis, therefore, should strive to describe and establish priorities for the various services that each street is required to provide. Undoubtedly, it will be found that much street capacity must be devoted to circulating traffic, truck loading, parking, pedestrians, etc. The functional analysis should attempt, as much as possible, to concentrate these activities on streets that are not required for the major traffic flows and to route major traffic movements to the arterial streets of the downtown area.

Arterial streets of this nature are the principal focus of this study. The functional analysis should be developed from a combination of transportation study outputs that specify trip origins and destinations, peak-hour volumes, etc., and an intimate knowledge of the aforementioned community characteristics. After the functional analysis has specified the purposes that each street must fulfill, the operational analysis develops ways and means to accomplish these objectives. The operational analysis, therefore, is directly involved with the physical characteristics of the street system and the composition and characteristics of the traffic stream. It is directly involved in providing an acceptable level of service, appropriate to the functions assigned, for each street of the downtown network. This study investigates some aspects of the functional analysis, but its primary interest is in the investigation of operational problems.

SYSTEM REQUIREMENTS

A primary purpose of this research was to design a practical and meaningful method for sampling traffic flow within an urban network of roadways to determine the degree of change resulting from any specific operational change within the system. Because it is impossible to perform complete physical measurements of all traffic within a network system, normal variations and relationships of flow must be understood and quantified to design adequate sampling techniques for statistical evaluation.

In selecting the type of measurements used to express changes of network flows, several aspects of the problem must be considered. The measurements should be meaningful to the driving public, useful to the design engineer, and practical to survey. Traffic flow consists of two broad elements that can be generalized as quantity of flow and quality of flow. Quantity of flow is a functional characteristic of traffic expressed in vehicles per unit of time, whereas quality of flow is an operational characteristic expressed in terms of relative congestion. There is, of course, a relationship between the two elements of flow; that is, on a specific roadway a given quantity or volume of traffic will experience a general quality of flow. The *Highway Capacity Manual* (*1*, Chaps. 4, 6, 9, and 10) developed relationships between service volumes (or quantity of flow) and a collective, qualitative measure of traffic flow (termed "level of service") for various types of highways and arterial roadways. This project has developed measurements of traffic flow for downtown street networks compatible with the *Manual's* level of service.

User Reactions

The primary interests of the driving public are safety, convenience of travel (including speed and travel time), freedom of movement, and reliability of travel. Safety measurements, through an analysis of accident records, require a long history before sufficient reliable data can be accumulated; and the information recorded may not be comprehensive enough for a meaningful analysis. The number of conflicts or potential accidents may be a more practical measure of safety; but considerable research beyond the

scope of this project would be required to develop relationships between potential conflicts and actual accident experience.

Convenience of travel and freedom of movement are general expressions of traffic congestion, or quality of flow. Although they are difficult to measure directly, these characteristics can be expressed by measurements of travel time, travel speed, delay time, running speed, number of stops, lane occupancy, etc. Reliability of travel can be expressed by a measure of the day-by-day variations in the quality characteristics of traffic flow.

Investigator

To locate and identify traffic problems and to be able to design operational improvements, the traffic investigator requires specific information concerning network flow. The human element must be uppermost in the investigator's mind when he is planning the functional use of roadways and designing operational changes within urban networks. Each and every vehicle in the flow of traffic is, after all, operated by a human being. The probable user reaction must be incorporated into the investigator's planning and design concepts. In addition to measurements of relative congestion, such as travel time, travel speed, delay time, and number of stops, within the roadway system, the investigator must understand the normal variations and interrelationships of traffic flow and be familiar with the trip origin-destination desires and other trip characteristics of the area, such as route preference and modal choice.

Surveyor

Any measurement selected to express traffic flow must be practical to collect in the field. Because it is physically impossible to measure all elements of flow within an urban system of roadways, the surveyor must be familiar with the normal variations of flow to design sampling techniques. A knowledge of traffic patterns by season, day of week, hour, and direction is essential to the collection and analysis of data.

The economics of collecting and summarizing counts will often determine the types of measurements and the technical method used to collect data. Installation of sophisticated electronic or mechanical equipment may prove too costly when considering the numerous locations required to measure network flow. Also, the number of decisions, judgments, and estimations required by the field observer or analyst should be kept to a minimum when designing measuring techniques.

BACKGROUND INVESTIGATIONS

The complexity of downtown traffic flow led to a search for one general measurement to describe quality of flow that would be meaningful to the driving public, useful to the design investigator, and practical to the surveyor. This measurement would combine the several aspects of travel time, running speed, delay time, and number of stops along the various roadways within the network system and would also be compatible with the several levels of service described in the *Highway Capacity Manual* (*1*, Chaps. 4, 6, 9, and 10).

Acceleration Noise—Mean Velocity Gradient

Acceleration noise, defined as the standard deviation of a vehicle's change in velocity over time, is a measure of the disturbance of a vehicle's speed from a uniform speed. This indicator of traffic congestion was sampled by injecting a test vehicle, equipped with special recording instruments, into the stream of traffic. The driver of the vehicle traveled selected prescribed routes using the standard "average car" technique. The recording instrument used for this project was the Marbelite Traffic Data Compiler, Model TD-1, designed by Jacob Greissman, that produces a graph with vehicle speed as the ordinate and travel time as the abscissa.

Acceleration noise has some obvious limitations as a complete measure of traffic congestion. A vehicle moving between 5 and 10 mph in a congested stream of traffic could easily display a lower value of acceleration noise than a vehicle traveling between 40 and 50 mph in a less congested traffic flow. The mean velocity gradient, defined as the acceleration noise divided by the mean velocity (or travel speed), is a more realistic measure of traffic congestion.

The equation to approximate acceleration noise is:

$$\delta_a^2 \cong \frac{1}{T} \cdot \frac{1}{\Delta T} \Sigma \Delta V^2 \qquad (7)$$

in which

δ_a^2 = acceleration noise (feet/second ^{2});
T = over-all travel time minus delay time (second); and
ΔV = change in velocity measured at equal increments of time ΔT (feet/second).

The mean velocity gradient (G) is defined as

$$G = \delta_a / \overline{V} \qquad (8)$$

in which $\overline{V}$ is the mean velocity (feet/second).

Data from a series of runs on Broad Street in Newark, River Road and Fort Nelson Way in Louisville, along with studies by Helly and Baker (*4*) on Park Avenue and Eighth and Ninth Avenues in New York were analyzed (see Appendix A) to study the relationships between mean velocity gradient and travel time. A straight-line relationship between mean velocity gradient and travel time (in seconds per mile) was developed for each of the roadways, using regression analysis. Plots of the equations are shown in Figure 83. The analysis indicated a high degree of correlation for each of the roadways, with a coefficient of correlation between 0.74 and 0.97.

It was concluded from this study and from logical theoretical data that, for a given roadway with a maximum running speed, there exists a linear relationship between the mean velocity gradient and the travel time; that is, for an urban roadway, where the maximum running speed is usually between 30 and 35 mph, travel time is as accurate an index of quality of flow as is the mean velocity gradient.

Capacity-Volume

Previous investigations had developed relationships between speed and volume for uninterrupted flow, as reported in the *Highway Capacity Manual* (*1*, pp. 59-65). Generally, as traffic volume increases, the space mean speed (the

average speed of a group of vehicles based on their average travel time over a section of roadway) decreases throughout the range of free flow up to the point of critical density. Beyond the point of critical density at maximum volume, both volume and space mean speed decrease with an increase in density. This relationship has led to the development of a factor based on travel speed, and the ratio of service volume to capacity to be used in identifying the level of service. The values of travel speed and volume-capacity ratio that define levels of service have been established in the *Highway Capacity Manual* for freeways, expressways, other multilane highways, two- and three-lane highways, and urban arterial streets. Values for downtown streets had not been established, and only approximate guides are presented in the *Manual* (*1*, Chap. 10).

Data collected from test-vehicle travel speed runs and volume counts on Broad Street and on Central Avenue in Newark were investigated to determine the relationship between travel speed and the volume-capacity (v/c) ratio. It was immediately obvious that an entire peak hour was too long a time period for meaningful results. Speeds for the same direction and section of roadway displayed as high a variation for runs within an hour period of the same day as runs within the same hour on different days. Therefore, individual test-car travel speed runs were matched with 15-min volume counts for the same direction and section of Central Avenue. To eliminate delays and speed changes caused by traffic signals, the running speed between the two signalized intersections on either side of the count location was used for this comparison. The volume counts were related to vehicles per travel lane for a 15-min period, and only runs made between 7:15 and 8:30 AM or between 4:30 and 5:45 PM were investigated.

Figure 84 is a plot of the relationship of running speed and 15-min counts of vehicles per travel lane on Central Avenue between West Market Street and South Eighth Street for four conditions: AM westbound, AM eastbound, PM westbound, and PM eastbound. This section of roadway has six lanes. The AM eastbound and the PM westbound are the heavy directions of flow and are allocated three lanes with parking prohibited. Parking is permitted on the westbound section during the morning and on the eastbound section during the evening peak periods, resulting in two travel lanes. Studying each of the four conditions separately, there is no evidence of any correlation between speed and volume. Studying the two-lane, with-parking sections combined, there is a definite trend downward in speed for the increase of volume from the AM westbound to the PM eastbound data. The reverse of this trend is evident, however, when comparing the three-lane sections with no parking. An increase of volume results in an increase of speed from the PM westbound to the AM eastbound data.

An explanation for this apparent conflict is shown in Figure 84, where the AM data and the PM data are compared separately. There is a similar trend downward in speed as volume increases for both the AM data and the PM data, although some of this slower speed may be attributable to the effect of curbside parking. More interesting is the definite lower level of speed during the PM period than during the AM period. In the peak direction, the PM

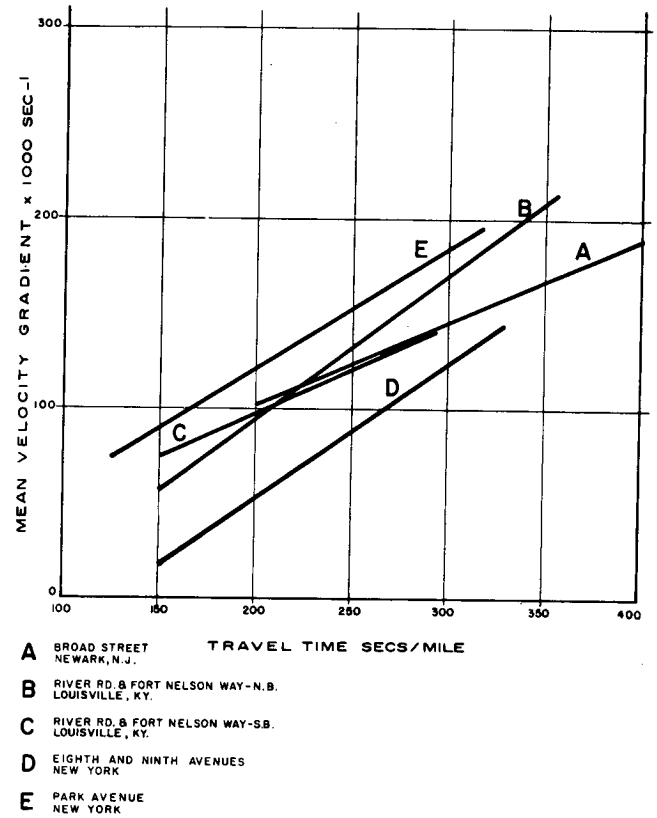


Figure 83. Mean velocity gradient versus travel time.

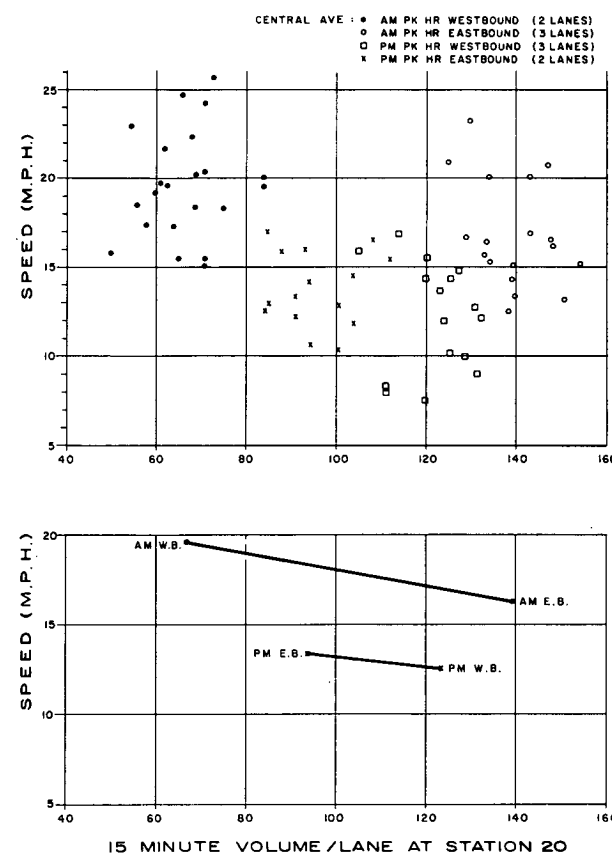


Figure 84. Running speed versus 15-min volume per lane.

volume is less than the AM volume; and the speed is about 3.5 mph slower on the average. The speed in the off-peak direction is reduced from 19.5 mph in the AM period to 14 mph in the PM period, whereas the volume during the AM is higher than the PM volume.

During the morning period of peak flow, there is probably a higher percentage of repetitive work-oriented traffic, less pedestrian activity, and a lower rate of parking activity than during the evening peak period, all of which would account for the higher running speed. Thus, the driver characteristics and external influences may be as important as the volume-capacity ratio when considering travel speed in downtown areas.

It should be noted that the large variation in running speeds within the same direction and time period is partially due to the short section of roadway used for analysis. However, similar results were obtained when over-all travel speeds on longer sections of Central Avenue and on Broad Street were compared with approximate 15-min lane volume counts. The comparisons indicated a slight downward trend of travel speed with a substantial increase of vehicles per travel lane, and a higher travel speed during the AM peak period than during the PM peak period, with similar volume levels.

The close spacing of important intersections with high percentages of turning vehicles, together with substantial curbside and off-street parking activities, makes estimation of traffic volumes over long sections of roadway very difficult in urban downtown areas. Meaningful capacity values are also difficult to estimate for downtown roadways where the effects of turning vehicles, parking activities, pedestrian crossings, mid-block frictions, bus stop interferences, etc., must all be evaluated. An estimation of intersection capacities is also difficult for individual intersections within a downtown network system.

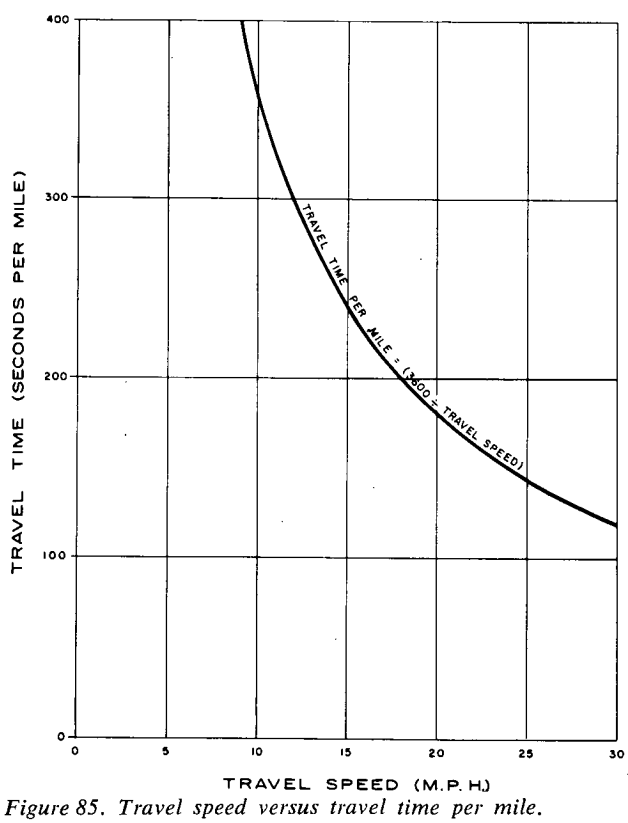


Figure 85. Travel speed versus travel time per mile.

Travel Time—Delay Time

The finding that the mean velocity gradient is proportional to travel speed for traffic on downtown arterials indicates that a relationship also exists between delay time and travel speed. The highest running speed attained on downtown roadways is usually limited to 30 to 35 mph. Except for periods of extreme congestion, this maximum running speed is commonly realized on most sections of arterial roadway. The difference between the maximum running speed and the over-all travel speed should therefore be proportional to the number of stops and amount of delay encountered.

The over-all travel speed, in miles per hour, can be expressed as travel time per mile, in seconds per mile, as shown in Figure 85. At any given travel speed, the travel time per mile is composed of delay time and running time per mile, where the running time includes the time for accelerations to and decelerations from a maximum running speed. It is anticipated that the delay time is negligible at travel speeds of 30 mph and increases substantially as the travel speed decreases.

Plots of observed data from test-car travel runs on various roadways in Louisville and Newark indicated that the relationship between running time (travel time minus delay time) per mile and travel speed is curvilinear. Equations in the form of

$$y = A + Bx + Cx^2 \qquad (9)$$

in which y is the running time per mile and x is the travel speed, were found to result in good estimations of this relationship (Fig. 86). The equations and coefficients of correlation for the data of each roadway tested are given in Table 37. The estimation curve within the range of observed travel speed is plotted for several different types of roadways (Fig. 87).

In Newark, McCarter Highway is a major arterial road, generally providing two lanes for each direction of travel with curbside parking prohibited, with a traffic volume of about 36,000 vpd. Central Avenue is an urban arterial serving the downtown area, generally consisting of two lanes for each direction, with parking prohibited on the major direction during peak period only. It accommodates approximately 17,500 vpd. Halsey Street is a one-way downtown street in the heart of the CBD, with traffic volumes of about 5,000 vpd. Generally, Halsey Street provides two lanes for moving vehicles, with a third lane used by vehicles making curbside pickups.

In Louisville, Broadway is a major downtown arterial, generally providing four lanes for eastbound traffic and three lanes for westbound traffic with parking prohibited, and with an average traffic volume of 32,000 vpd. Walnut Street is a one-way downtown arterial, providing four travel lanes with parking prohibited and a traffic volume of about 8,000 vpd. Brook Street is a one-way arterial serving as a collector-distributor roadway for ramp traffic to and

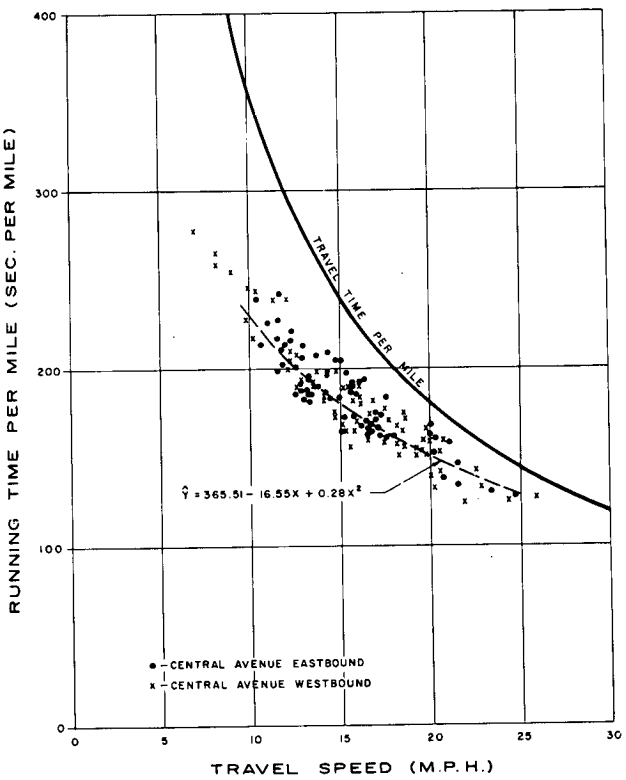


Figure 86. Running time per mile versus travel speed, Central Avenue.

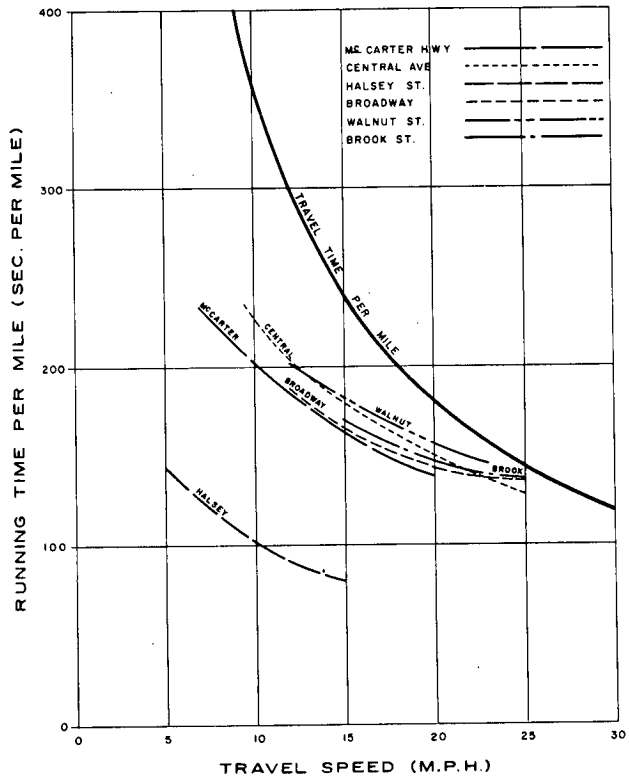


Figure 87. Running time per mile versus travel speed, selected roadways.

from an urban freeway and serves between 7,000 and 10,000 vpd on from two to four travel lanes. Signalized intersections are closely spaced, from five to ten per mile, along each of the test roadways. The length of roadway sections tested varied from 0.7 mile for Halsey Street to 2 miles for Walnut Street.

With the exception of Halsey Street, the plots in Figure 87 are similar. For any single travel speed, the amount of

TABLE 37

RUNNING TIME PER MILE AND TRAVEL SPEED RELATIONSHIPS

STREET AND DIRECTION	n	x	y	EQUATION	r
McCarter NB	108	13.82	173.90	$\hat{Y}=327.99-15.64\,x+0.31\,x^2$	0.827
McCarter SB	102	14.16	172.51	$\hat{Y}=322.26-14.13\,x+0.25\,x^2$	0.835
McCarter NB & SB	210	13.98	173.22	$\hat{Y}=323.15-14.64\,x+0.27\,x^2$	0.830
Central WB	74	16.1	177.5	$\hat{Y}=385.51-18.67\,x+0.34\,x^2$	0.948
Central EB	68	15.7	179.3	$\hat{Y}=322.45-12.00\,x+0.18\,x^2$	0.908
Central WB & EB	142	15.95	178.40	$\hat{Y}=365.51-16.55\,x+0.28\,x^2$	0.941
Walnut	119	17.62	169.75	$\hat{Y}=315.58-11.73\,x+0.19\,x^2$	0.880
Brook	98	20.91	146.92	$\hat{Y}=315.96-13.51\,x+0.26\,x^2$	0.652
Second	54	16.56	160.85	$\hat{Y}=302.82-12.78\,x+0.25\,x^2$	0.647
Broadway EB	106	18.80	153.69	$\hat{Y}=336.72-16.46\,x+0.34\,x^2$	0.830
Broadway WB	103	14.61	176.18	$\hat{Y}=243.67-4.34\,x+0.02\,x^2$	0.615
Springfield EB	74	13.09	198.62	$\hat{Y}=350.08-16.96\,x+0.38\,x^2$	0.824
Springfield WB	85	12.97	203.96	$\hat{Y}=464.08-31.24\,x+0.79\,x^2$	0.861
Halsey	50	10.07	104.48	$\hat{Y}=210.62-14.96\,x+0.42\,x^2$	0.766

n = number of observations;
$\bar{x}$ = mean travel speed (mph);
$\bar{y}$ = mean running time per mile (sec); and
r = multiple correlation coefficient.

delay for Halsey Street traffic is about 100 sec per mile more than the delay for traffic on the remainder of roadways tested. Halsey Street is not an arterial roadway. It receives less than 50 percent green time at most intersections, has a large volume of pedestrian interference, and contains a high portion of circulating traffic and turning vehicles. Halsey Street demonstrates the unique delay characteristics of minor circulation streets.

A composite equation, plotted in Figure 88, was developed from the combined data of several of the test roadways. The resulting equation is

$$\hat{y} = 320.722 - 13.55x + 0.25x^2 \qquad (10)$$

with a multiple correlation coefficient of $r = 0.808$. Considering the different characteristics of roadways selected, the inherent variability in traffic measurements, and the large number of observations, the magnitude of r is satisfactory. This equation will provide a reasonable estimation of running time per mile for a measured over-all travel speed on major downtown streets or urban arterials with closely spaced signalized intersections. Roadways with special conditions, such as an unusually slow planned speed of signal progression, a disproportionate allocation of green time, an exceptionally high level of mid-block interferences, or an extremely fast maximum running speed, would not necessarily experience the same relationships.

The relationship between running time per mile and over-all travel speed can be expressed in terms of delay time per mile, because delay equals travel time per mile minus running time per mile. A delay ratio, the ratio of the amount of delay time to the total travel time, can be developed from the delay time-travel speed relationship. The relationship between the delay ratio and travel speed is shown in Figure 89. The delay ratio equation is rather complex

$$D_R = 1 - (890.9x - 37.64x^2 + 0.694x^3)\ 10^{-4} \qquad (11)$$

in which D_R is the delay ratio and x is travel speed. However, the graph shows that the relationship is almost linear between practical travel speeds of 10 and 20 mph.

STATISTICAL EVALUATION OF FLOW DATA

To measure and evaluate changes in network traffic flow resulting from specific operational improvements, some knowledge of the normal variations and fluctuations of traffic flow within a downtown system must be obtained. The variations in traffic flow must also be understood to identify specific problems and to design adequate operational improvements. Some general characteristics of traffic flow in urban network systems have been developed from measurements collected during this project. Although the influences of various factors on traffic flow will not be identical for different urban areas, the relationships observed during this project can be a useful guide when designing procedures to sample traffic flow in other urban areas.

Numerous factors, both internal and external, contribute to the inherent variability and dynamic nature of traffic flow in urban areas. These effects, due to the repetitive character of urban travel, are experienced as variations in flow by

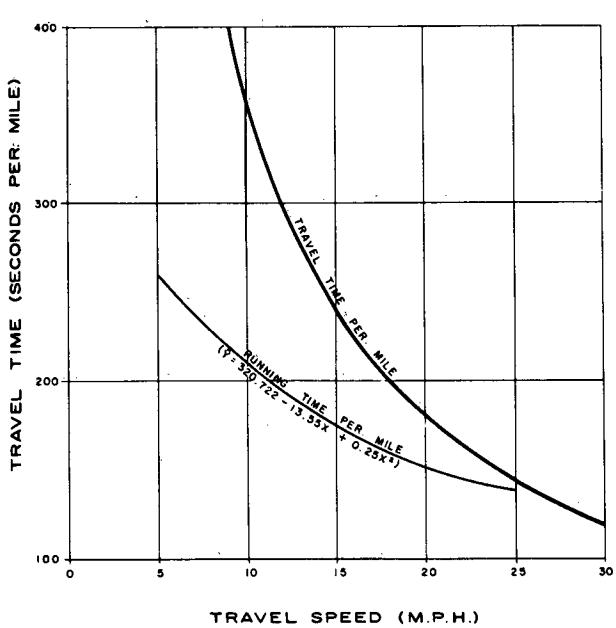


Figure 88. Composite running time per mile versus travel speed.

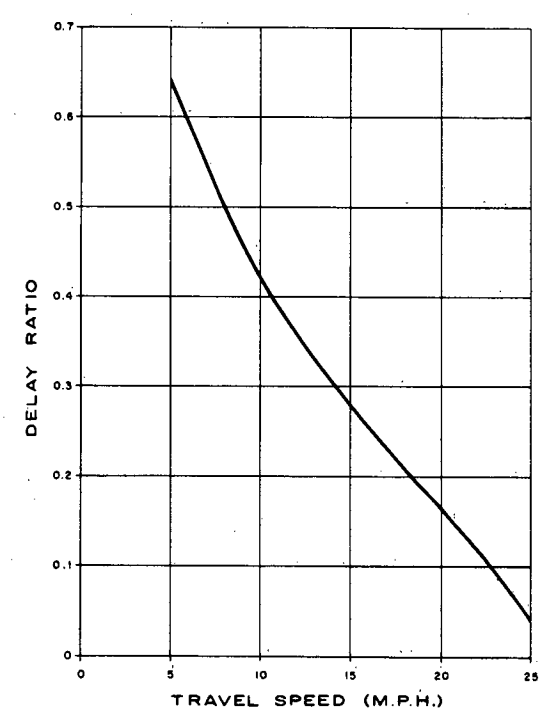


Figure 89. Delay ratio versus travel speed.

time period. Variations of traffic volume counts and travel speed measurements are expressed as seasonal, daily, and hourly variations.

Volume Counts

The number of vehicles passing over a given section of roadway during a specific time period expresses the quantity aspect of traffic flow. The volume per day and the volume per hour, usually counted by automatic traffic recording equipment, are probably the most widely used measurements in traffic engineering. Counts at intersection approaches are usually taken manually and are expressed in terms of vehicles per hour, per 15-min period, or per signal cycle.

Daily Volume Counts

From data derived at five counting stations in Newark operated one week each month (Monday through Friday) for a year, an analysis of the seasonal variation indicated no significant difference between summer, fall, winter, and spring. The seasonal variation was computed for each station by dividing the annual average daily traffic (AADT) volume by the average daily volume for each 3-month season. Table 38 gives the seasonal variations and the AADT volume for each of the five stations and the over-all seasonal variation of all five stations. Considering the much larger variations due to other factors, the 3-percent difference between a fall high point and a winter low point cannot be considered significant. There are, of course, particular events of shorter duration, such as the Christmas holiday season, that produce large variations. These events require special consideration.

Day of the week was observed to be a significant factor in Newark. Eight locations were analyzed using the analysis of variance on the weekday factors. This factor was computed for each weekday by dividing the annual average weekday traffic volume by the average volume for each weekday. A low factor represents a weekday volume higher than the average for a week. These factors are shown in Figure 90, with Tukey's multiple comparison limits. The limit lines for two means must not overlap in order to declare two days statistically different. Fridays have an appreciably higher volume of traffic than any other day of the week; Tuesdays and Thursdays are generally low-volume days under conditions prevailing in Newark; Mondays are somewhat higher than average, but not as high as Fridays. This study indicates that the day of the week is a factor to consider in developing traffic volume data for an urban study.

Hourly Volume Counts

Both direction of travel and time of day show a large effect on volume counts. Particular time-day combinations will almost certainly have higher or lower volumes than the average. Figure 91 shows the hourly pattern for each direction of flow during a typical 24-hr period at a counting station on Broad Street in Newark. The hourly patterns for three different days on Washington Street, a one-way street in Newark, are shown in Figure 92. The morning and evening peak traffic periods, generally between 7:30 and 8:30 AM and between 4:30 and 5:30 PM, are evident on each plot. It is these periods of peak traffic flow that are most important to traffic investigators.

In all experimental situations there is a certain amount of random variability. The "within" random variability observed at the Newark hourly counting stations gives an average estimate of standard deviation of about 110 vph during peak periods of flow. This variability was determined after removing effects for time of day and day of

TABLE 38

SEASONAL VARIATION, FIVE STATIONS IN NEWARK STUDY AREA

STATION	SUMMER (JUNE JULY AUG.)	FALL (SEPT. OCT. NOV.)	WINTER (DEC. JAN. FEB.)	SPRING (MAR. APR. MAY)	AADT
1 NB	1.04	0.98	0.99	0.99	17,088
2 SB	0.99	0.96	0.99	1.06	21,556
5 NB	0.98	0.97	1.03	1.03	16,479
SB	1.00	0.99	1.03	0.95	8,811
6 EB	1.02	0.99	1.03	0.95	8,811
WB	1.03	0.98	1.01	0.98	9,076
8	1.01	1.01	0.99	1.00	18,185
9	0.94	0.98	1.00	1.00	4,961
Average	1.00	0.98	1.01	1.00	

$$\text{Factor} = \frac{\text{AADT}}{\text{seasonal average}}$$

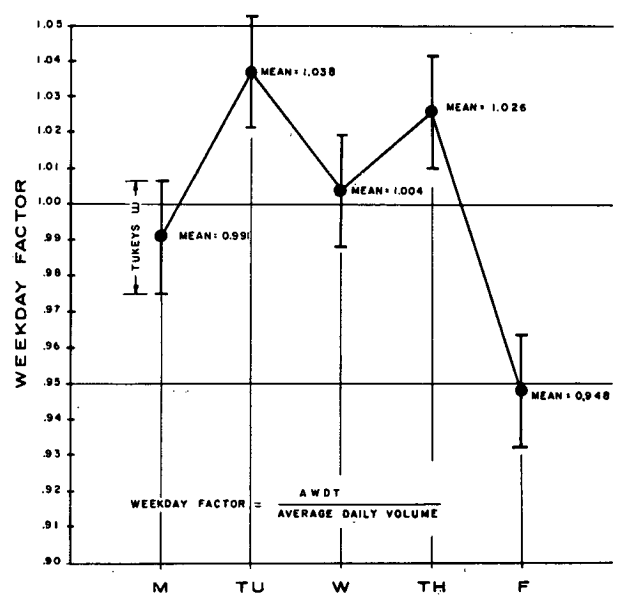


Figure 90. Weekday variation with Tukey's limits, Newark control stations.

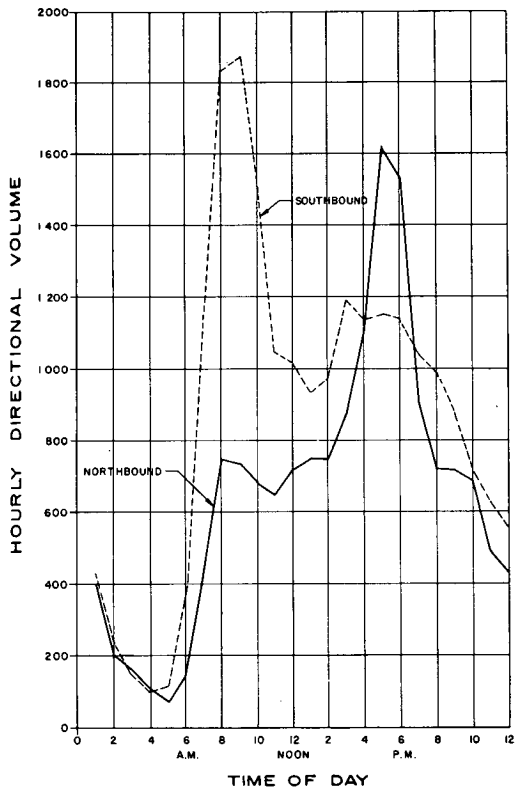


Figure 91. Broad Street hourly volume pattern, Station 2.

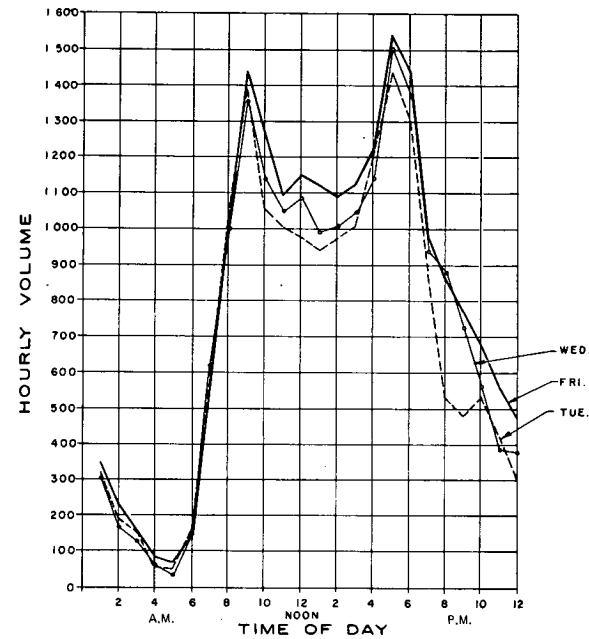


Figure 92. Washington Street hourly volume pattern, Station 8.

week. Standard deviations computed for samples derived at these stations range from 70 to 150 vph.

As an example of a sample size determination, suppose it is desired to measure the average peak-hour traffic volume on a certain street within $d = 100$ vph of the true average number of vehicles, with probability 0.95. Using

$$n = \left[\frac{Z_{(\alpha/2)}S}{d}\right]^2 \qquad (12)$$

in which

$n =$ number of samples;
$Z_{(\alpha/2)} =$ standard normal deviate (1.96 for $\alpha = 0.05$);
$s =$ standard deviation; and
$d =$ allowable error;

the number of samples is $\left(\frac{1.96 \times 110}{100}\right)^2$, or approximately 5. Hence, hourly counts should be taken for each of five days to determine the volume within 100 vehicles on either side of the true mean, with probability 0.95. These calculations are made assuming that the standard deviation is about 110. After some data have been collected at a specific location, it will be possible to refine this estimate.

Depending on the expected volume count at the station of interest, the choice of a value for d may be determined. For example, if a relatively low hourly volume is anticipated, a choice of $d = 100$ may be too large to be reasonable. However, it should be pointed out that the value of n increases with the square of d. Using $s = 110$, the various sample sizes necessary to detect within $\pm d$ vehicles per hour of the mean may be estimated as follows:

INITIAL SAMPLE SIZE DETERMINATION

$\pm d$	n
100	5
75	8
50	19
25	74

In summary, experience in the Newark study area indicates that seasonal variations in traffic volume on urban arterial streets may not be significant. However, certain special events that are known to affect traffic flow should be avoided. Samples should be taken on several days of the week; for instance, in Newark it was found that sampling as often on a Monday or Friday as on a Tuesday or Thursday eliminated bias due to any difference in days. Of course, time of day is important. The directional peak flows that largely establish the time periods of interest are very significant and should be carefully defined.

Intersection Counts

In the analysis of traffic flow at signalized intersections, it is necessary to determine the average number and the variance of vehicles passing through the intersection from each approach for each cycle during a given time period.

The number of vehicles through per cycle, as measured in many project experiments, can be described fairly com-

pletely from a statistical point of view. At any particular intersection, time of day, day of the week, and direction of travel drastically influence the magnitude of the measurements. It is, therefore, necessary that several days of the week be sampled for each of the time periods of interest.

In addition, because the cycle-by-cycle variations are quite large for this type of measurement, it is recommended that a survey of at least an hour's duration be made during each day and each time period. Because large variations in vehicles through are to be expected, numerous cycle measurements must be taken to average out the effect.

An apparent relationship between the mean and variance of vehicles through per cycle was observed after studying the results of many experiments. For vehicles through with means less than or equal to 30 per cycle, a straight-line relationship was hypothesized. Figure 93 is a plot of the means versus the variances for 27 experimental combinations. These data were fit by a simple linear curve of the form

$$y = A + Bx \tag{13a}$$

in which x is the mean and y is the variance. The method of least squares produced the line

$$\hat{y} = 1.983 + 0.923x \tag{13b}$$

with a correlation coefficient of $r = 0.844$ and a standard error of estimate at 3.7589. Both of these values were very satisfactory, considering the general pattern of variability experienced on cycle-by-cycle counts.

A plot of the derived regression line over the interval $0 < \bar{x} < 30$ is shown in Figure 93. A 95-percent confidence band for the predicted mean values is also shown. Hence, if an intersection has approximately 20 vehicles through per cycle, one might expect the variance to range between 18.8 and 22.2, with the best estimate of the variance at 20.5 vehicles.

Many standard tests have illustrated statistical tests by checking to see if the arrival rates are from a Poisson distribution. Among other assumptions for data to be Poisson, it is necessary that the mean and variance be equal, on the average. If the mean and variance are equal, then the slope (B) of the regression line ought to be about 1.00. To test this hypothesis form, the test statistic (5)

$$t = \frac{B - 1.00}{s\sqrt{1/\Sigma(x - \bar{x})^2}} \tag{14}$$

in which

t = test statistic;
B = sample slope;
s = standard deviation;
x = survey sample; and
$\bar{x}$ = survey mean.

Almost all of the calculations necessary for this test are found on the standard output for a simple linear regression. For the foregoing 27 observations, it was found that $t = -0.66$, which is not significant. This indicates that $B = 1$ would be reasonable for this set of data.

If the mean and variance are equal in the population, then the intercept value (A) should be equal to zero. For the foregoing set of data, the hypothesis that $A = 0$ was

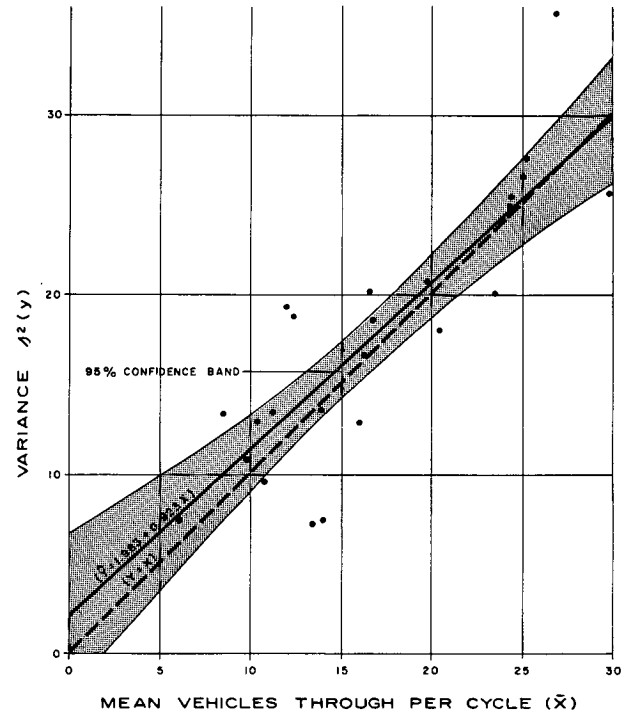


Figure 93. Variance of vehicles through per cycle (27 selected observations).

accepted, indicating that the line could go through the origin. Figure 93 shows the regression line and its limits, with the line $y = x$ overlaid. Notice that this line lies within the confidence band.

From the foregoing it can be concluded that vehicles through per cycle appear to be representable by a Poisson distribution.

Two intersection counts were chosen to investigate the cycle-by-cycle distributional nature of the measurements. Table 39 and Table 40 are a summary of the chi square goodness-of-fit test on two intersections: one in Newark and one in Louisville. Using a chi square test, it was determined that both sets of data can be reasonably well represented by Poisson random variables.

The foregoing results are encouraging. If the approximate mean number of vehicles through per cycle at a particular intersection is known, an estimated cumulative density histogram for that intersection could be constructed. As an example, suppose it is known from field observations that approximately eight vehicles per cycle on the average go through an intersection approach, and assume that it is desired to determine the 80-percent upper limit. Figure 94 shows the cumulative distribution function (Molina's functions) for Poisson data with mean equal to 8. By using Figure 94, one sees that 80 percent of the time no more than 11 vehicles should go through this intersection approach during one cycle. Limits in this manner could be used for development of simulations as well as for analysis.

The number of vehicles stopped each cycle were counted at numerous approaches to signalized intersections during

the project experimentation. The number of vehicles stopped (queue length) at the end of each red interval serves as a measurement of the quality of flow at an intersection and is particularly useful under conditions where revisions to signal timing are anticipated.

The cycle-by-cycle variations in vehicles stopped were even larger than the variation for vehicles through. Although the average number of vehicles stopped normally is less than the average number of vehicles through, the variability associated with vehicles stopped was generally more severe. Figure 95 shows 37 points with means plotted against variances, for experiments with the average number of vehicles stopped less than or equal to 20. This restricted set was used primarily because of lack of stability associated with long uncountable queues observed at some locations in Newark.

Using the method of least squares for a straight line yields

$$\hat{y} = 1.7271 + 1.8157x \qquad (15)$$

with a correlation coefficient of $r = 0.7945$ and a standard error of estimate at 6.4963. When testing the hypothesis that the population slope $B = 1$, this hypothesis was rejected, indicating that although means and variance are related, they are not equal. Hence, the vehicles stopped measurements cannot be represented by a Poisson distribution.

Figure 95 also shows the fitted regression line and a 95-percent confidence band for predicting limits of s^2 at a given value of x. As an example of the use of this figure, suppose one expected roughly 12 vehicles stopped per cycle on the average at a particular intersection approach. Using Figure 95, one finds that the expected variance should range between 20.8 and 26.8. The "best" guess is $s^2 = 23.8$.

TIME AND SPEED MEASUREMENTS

Traffic volume measurements are somewhat limited in their usefulness for network analysis. A more descriptive measurement is travel time for a unit distance. After all, the public is interested mainly in how much time their various travel needs require. Total delay time and the number of delays per trip certainly reflect the "smoothness" of a trip.

TABLE 39

GOODNESS OF FIT, CENTRAL AVENUE WESTBOUND AT HIGH STREET, NEWARK

VEHICLES THROUGH PER CYCLE	FREQUENCY, f		RELATIVE FERQUENCY, f/N	EXPECTED RELATIVE FREQUENCY	EXPECTED FREQUENCY	
13	0		0.0000	0.006	0.72	
14	3		0.0250	0.006	0.72	
15	1		0.0083	0.010	1.20	
16	1	13	0.0083	0.016	1.92	11.04
17	6		0.0500	0.022	2.64	
18	2		0.0166	0.032	3.84	
19	5		0.0416	0.042	5.04	
20	3		0.0250	0.051	6.12	
21	5		0.0416	0.062	7.44	
22	10		0.0833	0.071	8.52	
23	6		0.0500	0.076	9.12	
24	9		0.0750	0.079	9.48	
25	10		0.0833	0.080	9.60	
26	6		0.0500	0.076	9.12	
27	10		0.0833	0.071	8.52	
28	9		0.0750	0.063	7.56	
29	10		0.0833	0.055	6.60	
30	11		0.0916	0.045	5.40	
31	3		0.0250	0.037	4.44	
32	3		0.0250	0.029	3.48	
33	5		0.0416	0.021	2.52	
34	1		0.0083	0.016	1.92	
35	0		0.0000			
36	0	13	0.0000			16.44
37	0		0.0000			
38	0		0.0000		4.08	
39	0		0.0000			
40	1		0.0083	0.001		

$\bar{x} = 25.2$;
$N = 120$;
$\chi^2 = 13.981$;
χ^2 for df $= 13$ @ $\alpha = 0.50 = 12.34$;
$\therefore$ significant @ $\alpha = 50\%$.

TABLE 40

GOODNESS OF FIT, OAK STREET EASTBOUND AT SIXTH STREET, LOUISVILLE

VEHICLES THROUGH PER CYCLE	FREQUENCY, f		RELATIVE FREQUENCY, f/N	EXPECTED RELATIVE FREQUENCY	EXPECTED FREQUENCY	
1	0		0.0000	0.0001	0.016	
2	1		0.0063	0.0007	0.111	
3	0		0.0000	0.0026	0.411	
4	2	14	0.0127	0.0074	1.169	9.128
5	2		0.0127	0.0170	2.286	
6	9		0.0570	0.0325	5.135	
7	10		0.0633	0.0535	8.453	
8	6		0.0380	0.0769	12.150	
9	24		0.1519	0.0982	15.516	
10	12		0.0759	0.1129	17.838	
11	12		0.0759	0.1181	18.676	
12	18		0.1139	0.1131	17.870	
13	16		0.1013	0.1001	15.816	
14	13		0.0827	0.0822	12.988	
15	9		0.0570	0.0630	9.954	
16	11		0.0696	0.0453	7.157	
17	5		0.0316	0.0306	4.835	
18	6		0.0380	0.0196	3.097	
19	0		0.0000	0.0119	1.880	
20	1	13	0.0063	0.0068	1.074	11.945
21	0		0.0000	0.0037	0.585	
22	0		0.0000	0.0020	0.316	
23	1		0.0063	0.0010	0.158	

$\bar{x} = 11.5$;
$N = 158$;
$\chi_2 = 17.183$;
χ^2 for df $= 11$ @ $\alpha = 0.25 = 13.70$;
$\therefore$ significant @ $\alpha = 25\%$.

Test cars, using either the Marbelite Traffic Data Compiler or manual observers, have been used in travel time studies performed during this project. To compare several streets within a city, travel time surveys developed measurement of total travel time, average speed, running speed, total delay time, delay time per mile, number of stops, and stops per mile.

Average Speed

The same independent variables for speed measurements were investigated as in the traffic volume measurements; namely, time of day, day of week, and direction of travel. An analysis, using data recorded for several streets in Newark, indicated that day of week does not affect the average speed; that is, although some volume changes were observed, no appreciable speed differences were noted. Time of day, however, was observed to significantly influence the speed.

Figure 96 is a plot of the mean speed versus the "within" standard deviation for 107 experimental combinations. Separate equations were developed for the Newark and the Louisville data and for the combined data of both cities. Table 41 gives the equations, correlation coefficients, sample sizes, over-all mean, and standard deviation for each data group. Also included are the 15-percent and 85-percent speed values that represent the middle 70-percent distribution of speeds in each city. The equation for each city and for the combined data is shown in Figure 96 for the 70-percent range of travel speed.

The mean speed of the travel runs selected for Newark is about 4.7 mph less than the mean speed of the Louisville runs, and the mean 85-percent value for Newark (14.4 mph) is slightly less than the mean 15-percent value for Louisville (14.6 mph). Although the 70-percent range

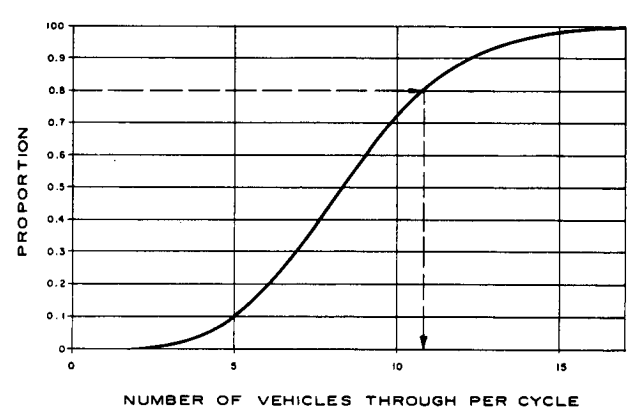


Figure 94. Cumulative Poisson with mean equal to 8.

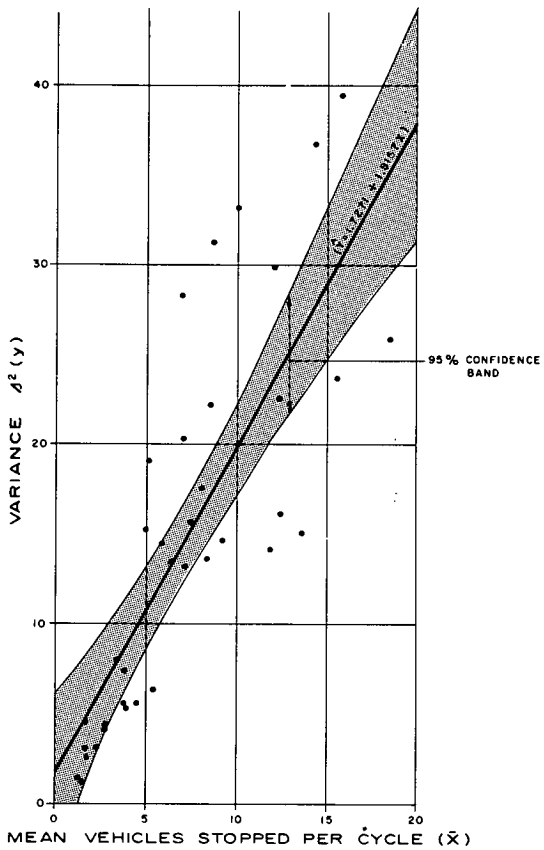


Figure 95. Variance of vehicles stopped per cycle.

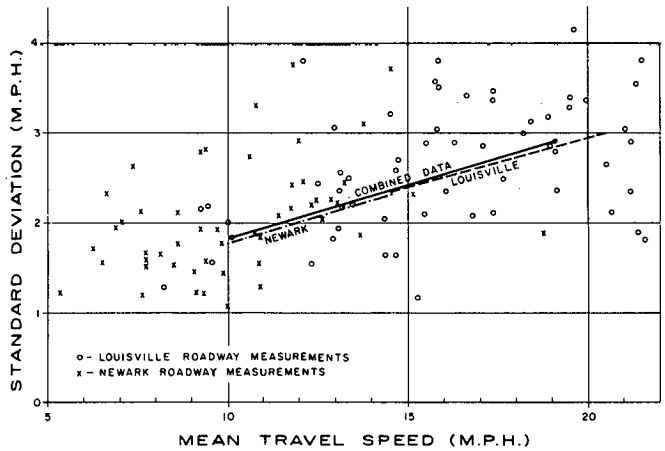


Figure 96. Standard deviation of mean travel speeds with regression lines for 70-percent central values.

reveals two entirely different sets of speed data for Louisville and Newark, the combined data for both cities illustrate a continuous consistent relationship between the standard deviation and the mean travel speed.

Generally, a standard deviation of 2 mph can be expected for a mean travel speed of 10 mph, and a standard deviation of 3 mph can be expected for a mean travel speed of 20 mph. Because there is a definite lower limit for travel speed, it is logical that the standard deviation would increase with an increase of mean travel speed.

With an estimation of the standard deviation for a given mean travel speed, an approximate sample size requirement can be determined. Table 42 gives the approximate number of samples required with probability 0.95 for allowable errors within 0.5, 1.0, 1.5, and 2.0 mph and mean travel speeds of 10, 15, and 20 mph.

For example, if it is desired to come within 1 mph of the true average travel speed, and the travel speed is expected to be about 15 mph, 24 sample runs are required.

Average Delay Time Per Stop

The average delay time per stop was found to be fairly consistent for data collected within each study area. Data from 47 test-car runs on various roadways in Louisville and from 89 test-car observations in Newark were converted to delay time per mile and number of stops per mile for comparison purposes. A stop was considered to occur when the test vehicle traveled at speeds of less than 2 mph for 6 sec. Because the majority of signals in Louisville have a 60-sec cycle length, and all of the signals in Newark have a 90-sec cycle length, the data for each study area were analyzed separately.

A simple linear regression equation

$$\hat{y} = 1.3787 + 23.3821x \qquad (16)$$

with a correlation coefficient of $r = 0.946$, was developed

TABLE 41

SUMMARY FOR SPEED DATA

CITY	EQUATION	CORR. COEF.	SAMPLE SIZE	MEAN SPEED (MPH)	STANDARD DEVIATION (MPH)	MEAN 15% VALUE (MPH)	MEAN 85% VALUE (MPH)
Newark	$\hat{y}=0.5622+0.1219\,x$	0.4129	50	12.2	2.05	10.0	14.4
Louisville	$\hat{y}=0.8265+0.1058\,x$	0.4146	57	16.9	2.61	14.6	20.4
All	$\hat{y}=0.6522+0.1155\,x$	0.5440	107	14.7	2.35	10.1	19.1

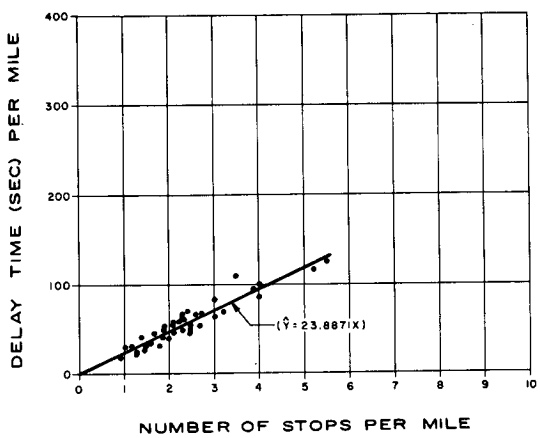


Figure 97. Delay time versus stops, Louisville.

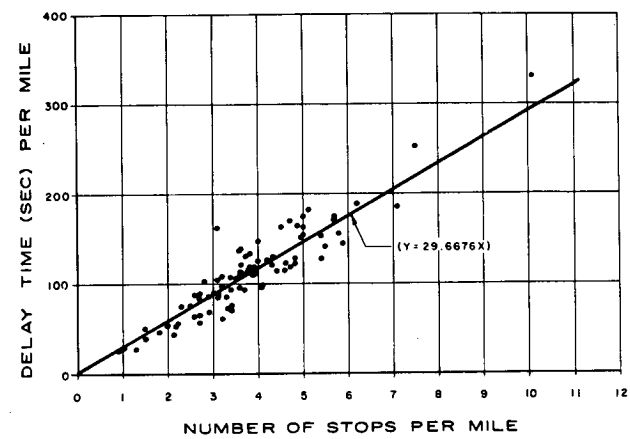


Figure 98. Delay time versus stops, Newark.

from the Louisville data. When forced through the origin, the equation was

$$\hat{y} = 23.887x \tag{17}$$

or 23.9 sec of delay per stop. This estimation line and the data for Louisville are shown in Figure 97. The equation developed from the Newark data was

$$\hat{y} = -5.5231 + 30.8592x \tag{18}$$

or 29.7 sec of delay per stop when forced through zero, as shown in Figure 98.

The difference between the delay time per stop for each set of data is probably due to the longer cycle length in Newark. The consistent relationship between delay time and number of stops within each area indicates the strong influence of signal delay within downtown areas. Although mid-block interferences are frequent, their effect is usually a reduction of running speed, resulting in an additional signal delay.

TABLE 42

ESTIMATED SAMPLE SIZE REQUIREMENT [a]

ALLOWABLE ERROR (MPH)	EST. SAMPLE SIZE REQUIREMENT, BY MEAN TRAVEL SPEED EXPECTED (MPH)		
	10	15	20
±0.5	62	96	139
±1.0	15	24	35
±1.5	7	11	16
±2.0	4	6	9

[a] With probability 0.95.

CHAPTER SIX

APPLICATIONS

This research project, unlike many other projects, has been involved in the study of many subjects. Methods of optimizing flow have been investigated in 37 experiments. Statistical analysis of flow data has described the variance and distributions of these data and developed information for control of surveys in the downtown area. Study of models for use in the analysis of downtown problems has resulted in the use of several models for these purposes. A method for describing a level of service for downtown roadways has been developed from the study of travel time. Problems of mass transit, enforcement, safety, public convenience, and many other aspects of traffic engineering in the downtown area have all entered into some phase of this project. Because of the varied nature of the project, many

of the applications of particular studies have been discussed in the chapters or appendices that deal with the development and analysis of these subjects. Therefore, the purpose of this chapter is not to recapitulate the details of every application of every investigation that has been performed, but rather to describe the application of these elements in a broader sense to develop a systematic approach to analysis of the traffic problems of a downtown area. As such, it is directly oriented to the fourth objective of the project statement which requires the project studies to "outline a procedure for the practical application of the results of this research to street networks in general."

RELATIONSHIP TO THE URBAN AREA TRANSPORTATION STUDY

The study of the problems of a downtown area should proceed logically from the base that has been established in the urban area transportation study. This study, required for all urban areas of more than 50,000 population, will provide the basic information with which to begin analysis. The urban area transportation study should be able to provide a description of origins and destinations of travelers to and through the downtown area, records of counting stations in the area, and description of trips by purpose, time period, and mode of travel. The transportation study should also define the area to be given further study as the "downtown area." If practical, screen lines should be developed about this area and screen-line crossings should be described. The functional classification of roadways probably will not be as detailed as is required for downtown analysis but will have established at least the major components of the arterial system of the downtown area. If parking has been analyzed as part of the transportation study, valuable information for the analysis of the downtown area will be parking turnover rates, duration, parking accumulations, together with forecasts of supply and demand. Projections of future needs will also be developed as part of the transportation study, as well as a description of community goals related to land-use planning, redevelopment programs, and other similar developments. All these aspects of urban area transportation planning that emanate from the transportation study will form the base on which the study of the downtown area is built.

FUNCTIONAL ANALYSIS

The functional classification of streets for a downtown area can be a fairly complex subject, because it must recognize the many uses of the downtown street system and the changing patterns of traffic by time of day. For instance, a street that is an arterial during hours of peak traffic flow may become a land service road at other times of the day. The functional classification of roadways begins with the definition that has been handed down from the transportation study and develops closer definition based on additional analyses. These analyses may require some initial survey and reconnaissance, together with much of the background of historical use of these roads, which should develop an appreciation for those other uses that are an important part of the downtown environment. The functional classification should recognize these needs and develop a flexible situation which, while providing for the movement of traffic, does not ignore the need for pedestrian movement, loading and unloading of merchandise, short-term parking, delivery services, bus operations, taxi stands, emergency services, and a multitude of other components of life in the downtown area. As much as possible, those elements that do not have a direct bearing on the movement of traffic should be relegated to off-street areas. This is especially true for those periods of peak traffic when many normally acceptable roadway uses may be banned to develop additional capacity for movement of traffic. The major problem of functional classification is to develop sufficient arterial capacity to meet the changing traffic needs of each time period of the day. Because of the heavy directional movements of peak periods, a street that is not required for arterial purposes during periods of light traffic may be impressed into use as an arterial to meet the needs of peak traffic flow. To maintain this flexibility, it is suggested that downtown streets be classified into four functional categories, as follows:

1. Arterial streets—These are the elements of the road system that serve as major traffic arteries throughout all time periods of the day. On-street loading, parking, and other uses, not related to movement of traffic, will be severely restricted, and enforcement should be rigid.
2. Limited arterial streets—This element of the street system serves a dual purpose, being cleared for maximum movement of traffic during peak periods while serving the general circulating and land service purposes of the community during other time periods.
3. Circulating roadways—Although these roadways are used for movement through the downtown area, they are generally used not for long trips but rather for circulation from point to point within the area and for access to parking. This element of the street system would, when necessary, be used to fulfill the needs of the community for short-term parking, loading and unloading of merchandise, and other similar services that cannot be accommodated off-street.
4. Land service roadways—These minor elements of the street system are seldom used by traffic in movement through the area. Their major function is access to adjacent properties. They include the back alleys and narrow streets not suitable for traffic movement. Occasionally they may be found suitable for truck loading and parking, if this does not interfere with their access function.

The arterial and limited arterial roadways are of major interest to the traffic study of the downtown area. These roadways should include those that are used for major bus movements, and the organization of bus stops along these roadways is a matter of prime importance. Also, during periods of peak traffic, exclusive bus use of certain lanes may be warranted if the number of buses is sufficient. Traffic signals for intersections of arterial and limited arterial streets should, at a minimum, be totally interconnected, fixed-time with sufficient dials to accommodate the changing traffic patterns by time period (usually three).

In planning the use of arterial streets by time period, the Network Assignment Model will be a useful tool for testing the effect of various patterns of street use. For instance, one-way pairing of streets or reversible-lane operation on major arterials could be tested prior to implementing the change to assess its probable effect. The Signal Analog Model also will be useful for developing signal offset relationships of signals of the arterial street system. This knowledge may then assist in the selection of limited arterial streets that best fit the pattern set by traffic progressions for the arterial streets.

The functional classification of streets should produce a complete description of desired street use in the downtown area by time period, with particular consideration of curb-lane use, detailing the traffic regulations that are necessary to implement the desired changes in street use during various time periods. The functional analysis will establish a basis for evaluation of the operation of the arterial system.

OPERATIONAL ANALYSIS

After selection of the arterial and limited arterial streets of the downtown street system, surveys should be conducted to describe, in more detail, the traffic volume fluctuations by time of day and the travel time for each arterial and limited arterial street. Volume counts and travel time surveys should be organized to develop the information described in Chapter Five. Appropriate sample sizes and controlling characteristics of the survey may be assessed using the methods described in Chapter Five. On completion of the travel time surveys, a level-of-service definition may be developed for each street surveyed, based on the analysis of travel time described in Chapter Three. Deficiencies then may be identified in general form by those areas having a low level of service. Study priorities may also be established in relation to this definition. Inspection of the results of the travel time surveys will make possible identification of those locations where repeated delays have occurred. These locations may then be subjected to field reconnaissance to determine the cause of congestion, the need for additional surveys, and the appropriate analysis. In the study of intersections, the analysis method described in Chapter Three may be applied. This involves the surveying of vehicles stopped on red, vehicles through, and number of saturated cycles. Newell's Intersection Model may then be used to estimate, based on surveyed data, the delays at each intersection approach, showing the imbalance that may exist. The need for additional lane capacity, turning lanes, separate signal phases, etc., may then be assessed and the appropriate action may be taken.

In performing the operational improvements, it would be logical to implement analysis and improvements in the following order:

1. Effect major changes in road use resulting from the functional analysis, such as one-way streets, reversible lanes, and reversible roadways.

2. Assess local conditions that require channelization, elimination of parking, provision of turning lanes, guide and directional signing, lane striping, parking prohibitions, control of truck-loading operations, and all other such elements designed to eliminate friction in local areas.

3. Develop signal phasing, cycle length, and offset relationships of adjacent signals. These should be studied to develop the best possible progressive movement for each time period of the day on a network basis, considering arterial and limited arterial streets.

4. Finally, the study should progress to consideration of transit movement in the system, using the methods described in Chapter Four.

For purposes of transit analysis, surveys should be conducted to assess the average number of passengers loaded and discharged at each stop location by time periods of the day, and these data should be used to predict passenger service time. Information developed in Chapter Four describes those elements that control location of a bus stop. This will assist in selecting the best stop location from an operational standpoint. Because a major element in this consideration is the incidence of signal delay, a study should be made of the optimum bus position with respect to the signal progression. If a time-space diagram is developed showing the red and green time bands for each arterial street, bus passage through the time bands may be depicted as a line beginning at the upstream signal and extending through the green time band to the first bus stop. At this location the time line for the bus will allow for the estimated passenger service operation time required at that stop. Then it will proceed through the next available green time band to the next stop location. This study will determine those locations where the bus may anticipate a red signal and, therefore, use a near-side stop, taking advantage of red signal time for passenger service operations. If a green signal is anticipated, a far-side stop location should be used. This study may be continued down the entire length of the arterial street, describing the probable passage of a bus through the downtown area. The coordinated signal system acts, in this case, as a huge clock by which bus movement is controlled and that the bus operator should be instructed to recognize in his passage through the downtown area.

On completion of the physical improvements, "after" travel time surveys and field reconnaissance should confirm that the anticipated flow improvement has actually been realized. This is necessary, because in many cases changes to traffic flow may produce unexpected results. Violations by drivers and pedestrians may nullify the intended improvement and may require additional consideration of the problems of that particular location. Traffic may be diverted because of an improvement and the traffic patterns may change, requiring adjustment of signal timing or of other aspects of the improvement. Often the most efficient use of a street is more susceptible to disruption by violators than less efficient use. For instance, if 50 ft of pavement are divided into five lanes to provide a center lane for left-turning traffic, the 20 ft left for directional movement are not adequate for two lanes of through traffic if vehicles are parked in the curb lane in violation of parking ordinances. However, if the 50 ft of pavement had been marked with a center line only, the 25 ft of pavement on each side could

still accommodate two lanes of traffic if parking violations occurred. Obviously, the more efficient pavement use requires more efficient enforcement of the regulations. Where such sensitive areas are recognized, they should be described to the traffic police for their consideration of appropriate enforcement. As the operational analysis and the implementation of improvements proceed, it will be necessary to develop the body of traffic regulations that will protect the value of the engineering efforts and financial investment in the improvements. It is essential that the proper ordinances be enacted that will provide the basis for effective administration and enforcement.

CONTINUING REVIEW

The need for a continuing review of the traffic operations in the downtown areas is similar to the need for updating of a transportation study. Redevelopment programs, highway construction, changing land uses, and other elements of urban growth and change will undoubtedly be reflected in the traffic flows of the downtown area. As these changes occur, it will be necessary to make appropriate adjustments. Counting stations should be established for systematic assessment of the growth of traffic and change of patterns by time periods of the day. A travel time survey should be implemented periodically and compared to the historical record of travel time to pinpoint developing situations. Field reconnaissance by competent traffic engineers is an important part of a continuing review to recognize and define the need for further traffic engineering action. This study does not attempt to develop a timing for such surveys, but certainly it should not be greater than that which has been accepted for the updating of transportation studies (5 yr). Comprehensive reviews should probably be made at similar intervals, with minor attention being given to other areas on a flexible basis as the need is determined. Undoubtedly, the need for review will coincide with major changes influencing traffic, such as the opening of an adjacent highway or the addition of a large traffic generator.

CHAPTER SEVEN

CONCLUSIONS AND SUGGESTED RESEARCH

In general, it may be concluded that significant benefits to traffic flow may result from relatively minor operational improvements to the downtown street network. In most cases, the benefits to the traveling public obviously far outweigh the costs of analysis, engineering, and construction of the improvement.

In prescribing an order for the implementation of improvements in a downtown area, those elements that involve the functional use of streets (such as one-way patterns, reversible-lane operations, and major parking prohibitions) should be developed first. This should be followed by analysis and correction of all the minor influences that create frictions in the traffic stream, such as turning movements, truck loading, pedestrian interference, proper allocation of signal time to the various approaches of an intersection, and other traffic problems of the area. When local frictions have been reduced sufficiently, proper platooning of traffic for implementation of signal progressions may be possible. When this has been successfully accomplished, the optimum location of bus stops may be considered and bus movement in the progressive system may be developed.

As a result of the analysis of travel time, it is concluded that a level-of-service definition describing quality of flow on downtown streets is possible. This level-of-service definition should be based on the delay ratio measured by travel time surveys.

A review of the results attained by project experimentation for the improvement of traffic flow indicates that large improvements may be associated with poor "before" levels of service, whereas minimal improvements may be associated with high "before" levels of service. Therefore, it is concluded that level-of-service definition is a reliable indicator on which to base the priorities for traffic improvements.

As a result of investigations for use of various models in the analysis of downtown network problems, it is concluded that the Network Assignment Model, Newell's Intersection Model, and the Analog Traffic Signal Model are all useful tools in their respective fields of application for these analyses.

Based on experience of the experimental program, it is concluded that strict enforcement of necessary traffic regulations is an essential component of traffic engineering in the downtown area. In several experiments, improvements designed to optimize road capacity resulted in hazardous situations and inefficient road use because of violations. Under these conditions the value of an improvement may not be realized unless enforcement sufficient to educate the public to correct driving habits is available.

SUGGESTED RESEARCH

Road User Behavioral Studies

In the downtown area a high level of regulation must be imposed on road users. Relatively slow traffic movement and the many frictions caused by circulating traffic, parking and unparking, truck loading, dense signalization, and heavy traffic volumes all affect the road user by restricting his freedom. Driver reactions to this high level of control are varied. Most drivers will accept and recognize the need for such control. However, a sizable number of drivers place their personal interest first and are willing to violate regulations, to the detriment of the other road users. It is suggested that research should be considered that would test driver reactions to various controls to develop information describing the best possible way of implementing these controls to attain the highest level of acceptance by road users.

Enforcement of Regulations

In conjunction with the foregoing paragraph, a parallel approach to the same problem of obtaining driver acceptance of regulations would be to work in the field of enforcement. This research should investigate the most advantageous ways of reducing violations of traffic regulations, molding the public opinion, dissemination of information, education of police in the technique of educating the public, enlisting the aid of citizen groups in an effort to create public awareness, and production of demonstration media such as signs, placards, films, newspaper articles, and radio broadcasts. Careful observation of the number and types of violations occurring before and after such a program should be able to demonstrate the value of public information and enforcement programs.

Extension of the Level-of-Service Concept for Downtown Roads

Research conducted on this project has demonstrated that the delay ratio is a reliable indicator of the level of service for downtown streets. However, this research is based on measurements made in only two study areas. While these study areas differ widely in traffic density, driver characteristics, and road network characteristics, it is possible that other elements that were not evident within these study areas may influence this determination. Therefore, it is suggested that measurements of a similar type be made in other study areas, selecting those having widely differing characteristics, to confirm these findings or amend them if necessary.

Development of a Traffic Data Coordination System

The present TOPICS program presents an opportunity for developing much information based on improvements to be made under this program. Research should be conducted to determine the best methods to be used for coordinating data gathering from this program for specific purposes, detailing the purposes for which data are required, developing criteria for necessary measurements, and preparing data forms and specifications so that uniformity can be attained in collection of these data. Such information, for instance, could be used to assess the separate influence that each type of improvement has on the delay ratio in a wide number of applications reflecting varying densities of traffic, road types, geographic areas, and other conditions.

Extension of Bus Studies

Opportunities did not exist within the study areas for effectuating a complete redesign of a bus route using the data and methods described in Chapter Four and the applications described in Chapter Six. These elements of bus operation could be tested only in local areas. It is suggested that opportunities be sought to determine the increase in efficiency that would result from the complete engineering of a bus route in a downtown area, where its activities could be coordinated with the movement of traffic on arterial streets that have been subjected to thorough engineering analysis to eliminate traffic frictions and establish efficient signal progressions. The center cities depend heavily on bus transit which, because of their high persons-per-vehicle ratio, should be given special consideration in the operations of the downtown area.

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APPENDIX A

ACCELERATION NOISE AND MEAN VELOCITY GRADIENT

Widely used measures of traffic congestion or quality of traffic flow are travel speed or travel time. However, other indicators of relative traffic congestion that have been proposed include parameters termed "acceleration noise" and "mean velocity gradient." An assessment of the applicability and reliability of these parameters as indicators of traffic congestion on downtown streets was undertaken, using previously published data and data collected as a part of this research project.

Acceleration noise is defined as the standard deviation over time of a car's acceleration, and can be considered as a measure of the disturbance of a vehicle's speed from a uniform speed. If the vehicle's speed is fairly uniform, the distribution of acceleration would be as shown in Figure A-1.

However, if the accelerations are more violent and frequent, the distributions of accelerations would exhibit a wider pattern, as shown in Figure A-2.

Even under ideal conditions, a vehicle traveling on a roadway will generate some acceleration noise due to the inability of the driver to maintain a constant speed; and it is reasonable to assume that the more adverse the driving conditions, the greater will be the acceleration noise.

Acceleration noise is calculated by use of the following equations:

$$\delta^2 = \frac{1}{T}\int_0^T \left[\frac{dv}{dt} - \frac{d\bar{v}}{dt}\right]^2 dt \qquad \text{(A-1)}$$

$$\delta^2 \simeq \frac{1}{T}\ \frac{1}{\Delta T}\Sigma\Delta V^2 \qquad \text{(A-2)}$$

in which

δ = acceleration noise, ft/sec^2;
T = running time (over-all travel time minus stopped time), sec;
dv/dt = acceleration, ft/sec^2;
$d\bar{v}/dt$ = average value of acceleration; and
ΔV = change in velocity, ft/sec, measured at equal increments of time, ΔT.

While a vehicle is stationary, there is no variation in velocity, and noise is not generated. The running time (over-all travel time minus stopped time) is therefore used in computing noise levels. It should be noted that if over-all travel time were used as a basis for computing noise levels, the noise level would decrease with increasing delays, giving a somewhat unrealistic measure of traffic flow.

The mean velocity gradient is defined as:

$$G = \delta_a/\bar{V} \qquad \text{(A-3)}$$

in which

δ_a = acceleration noise, ft/sec^2;
$\bar{V}$ = mean velocity (including stops), ft/sec; and
G = mean velocity gradient, sec^{-1}.

This index was suggested as a better indication of traffic flow by Helly and Baker (*4*) as a result of traffic studies in New York City.

DATA SOURCES

Sources of data for this study include a series of trips on Broad Street in Newark, N.J., and on River Road and Fort Nelson Way in Loiusville, Ky., using a Marbelite Traffic Data Compiler. The Marbelite graphs show a plot of velocity in miles per hour (1 in. = 20 mph) and travel time in seconds (1 in. = 60 sec). The acceleration noise and the mean velocity gradient were computed for each trip, using equal time intervals of 3 sec. With velocity readings in miles per hour and time increments of 3 sec, Eq. A-2 reduces to:

$$\delta_a \simeq \sqrt{\frac{0.717}{T}\ \Sigma\Delta V^2}\ \text{ft/sec}^2 \qquad \text{(A-4)}$$

in which

ΔV = change in velocity (mph) measured at 3-sec intervals.

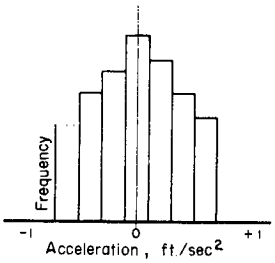


Figure A-1. Distribution of acceleration with minor deviations.

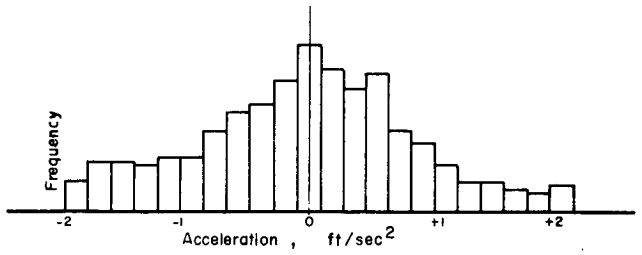


Figure A-2. Distribution of acceleration with larger deviations.

The results of studies of noise and mean velocity gradient on Park Avenue and Eighth and Ninth Avenues in New York City, reported by Helly and Baker (*4*), were also analyzed.

In addition, the data from a number of theoretical trips at different velocities with a varying number of stops were analyzed to study possible relationships between noise or mean velocity gradient and travel time.

Broad Street

Acceleration noise and mean velocity gradient were calculated for 25 trips on Broad Street from Poinier Street to Orange Street, a distance of approximately 2 miles.

The straight-line relationship that exists between noise and travel time (in seconds per mile), shown in Figure A-3, was developed using regression analysis. The noise level slowly decreases with increasing travel time. Also shown in Figure A-3 is a straight-line relationship between mean velocity gradient and travel time. This relationship, which has a correlation coefficient of 0.97, also was determined using regression analysis.

River Road and Fort Nelson Way

The data used to compute noise levels were originally collected as part of Experiment E31, Revisions of One-Way Patterns on River Road and Fort Nelson Way. There was a substantial improvement in speed in a northbound direction from Sixth Street and Main Street to Third Street and River Road. A comparison between the "before" and "after" noise and the "before" and "after" mean velocity gradient also indicated a substantial improvement.

There was a definite trend for noise to increase with increasing traveltime (Fig. A-4). A linear relationship with a coefficient of correlation of 0.96 was established between the mean velocity gradient and travel time, using linear regression analysis. A curve of the form

$$y = a + b \log x \qquad \text{(A-5)}$$

gave a somewhat better correlation; however, as Figure A-4 shows, there is little difference between the curves.

There was no significant improvement in traffic flow in the southbound direction from Third Street and River Road to Seventh Street and Main Street. This lack of improvement is confirmed by comparing the noise and the mean velocity gradient for "before" and "after" conditions (Fig. A-5). The noise level appears to be virtually independent of travel time, and this was confirmed by a regression analysis. A straight-line relationship with a coefficient of correlation of 0.74 was established between travel time and mean velocity gradient.

Park Avenue and Eighth and Ninth Avenues

The noise and mean velocity gradients were originally determined by Helly and Baker (*4*), using over-all travel time as the basis for computing noise. The values of noise and mean velocity gradient were recomputed using running time, to conform with the definitions of noise and mean velocity gradient used in this study.

Straight-line relationships were developed between noise and travel time for Park Avenue and Eighth and Ninth Avenues (Figs. A-6 and A-7). Noise on Park Avenue is virtually independent of travel time. The number of samples of noise on Eighth and Ninth Avenues is insufficient to decide with any certainty whether noise is strongly dependent on travel time. The elimination of only the two lower values of noise would lead to the conclusion that the noise is virtually independent of travel time.

Straight-line relationships were developed between mean velocity gradient and travel time for Park Avenue and Eighth and Ninth Avenues, as shown in Figures A-6 and A-7.

Eighth and Ninth Avenues have progressive signalization ("green waves") that dictate a maximum speed of approximately 25 mph (140 sec per mile). To achieve this maximum, it is necessary to travel very smoothly in synchronization with the traffic signals. The mean velocity gradient is, therefore, quite low at about 140 sec per mile. The "green wave" is a disciplinary measure to urge drivers into smooth behavior, provided the traffic is light enough for the "green wave" to be followed.

Park Avenue, with "stop-go" signalization, is quite different. If a driver wishes to maximize his speed, he must accelerate and brake as violently as possible under competing traffic conditions. This would explain why the mean velocity gradient is generally higher on Park Avenue than on Eighth and Ninth Avenues.

Theoretical Trips

A series of trips with different maximum speeds and a varying number of stops were analyzed. Typical trips are shown in Figure A-8. The noise and mean velocity gradient

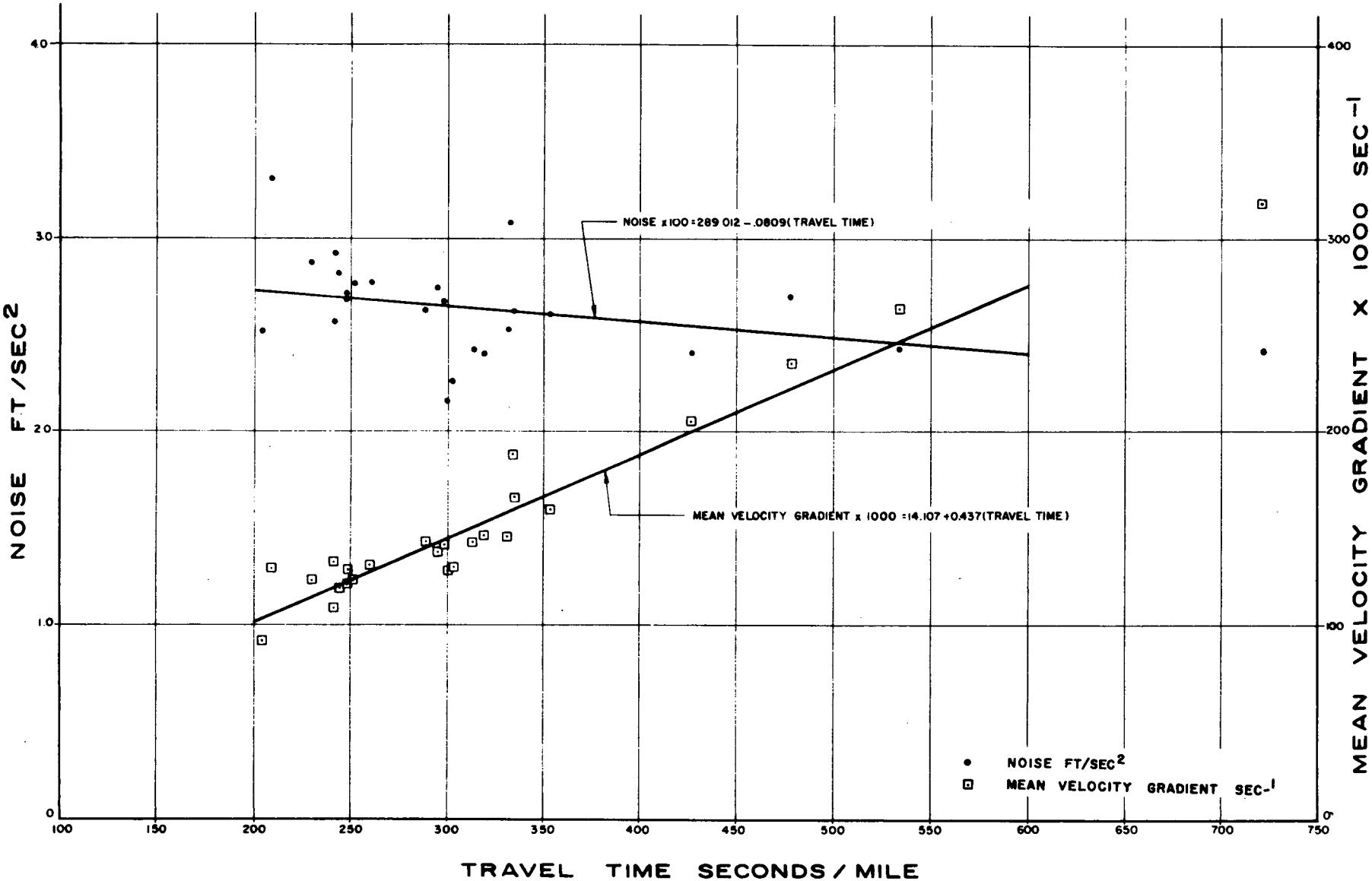


Figure A-3. Noise and mean velocity gradient, Broad Street, Newark.

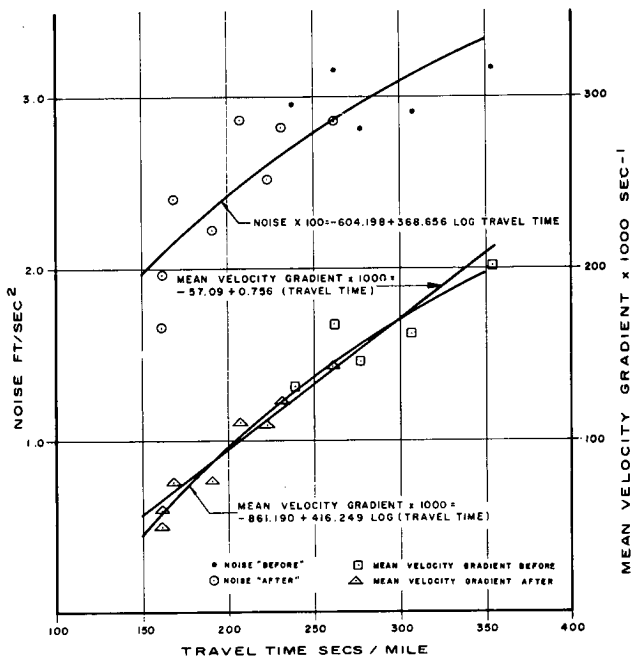


Figure A-4. Noise and mean velocity gradient, River Road and Fort Nelson Way northbound, Louisville.

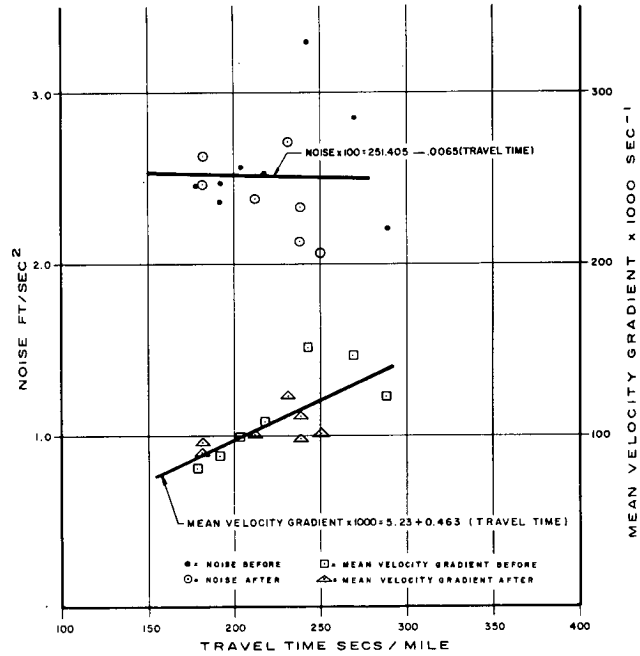


Figure A-5. Noise and mean velocity gradient, River Road and Fort Nelson Way southbound, Louisville.

were calculated for each trip; the results are summarized in Figure A-9. Although noise increases with increasing travel time, it is not a linear relationship. Noise increases at a diminishing rate for longer travel times. There was essentially a straight-line relationship between mean velocity gradient and travel time for any series of trips with

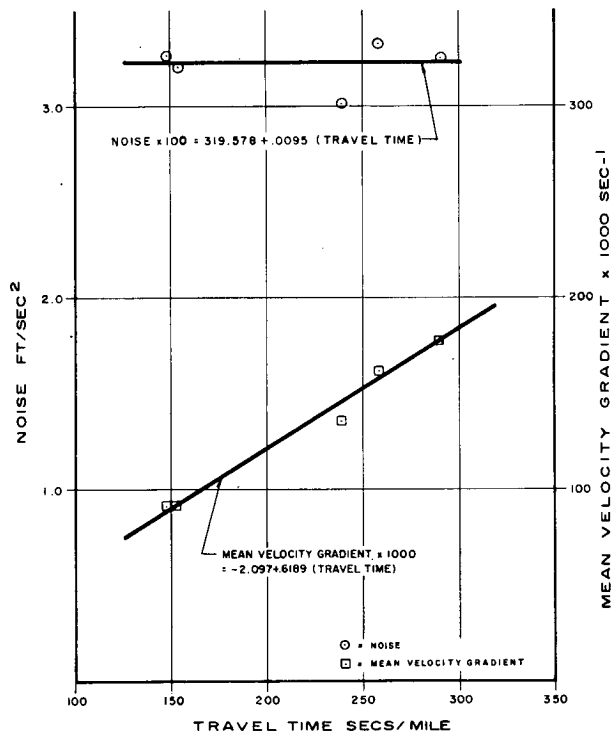


Figure A-6. Noise and mean velocity gradient, Park Avenue, New York.

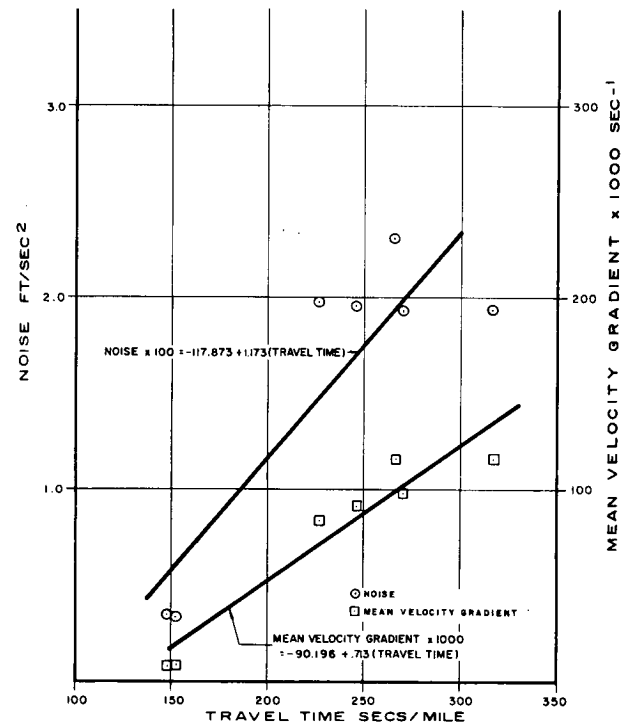


Figure A-7. Noise and mean velocity gradient, Eighth and Ninth Avenues, New York.

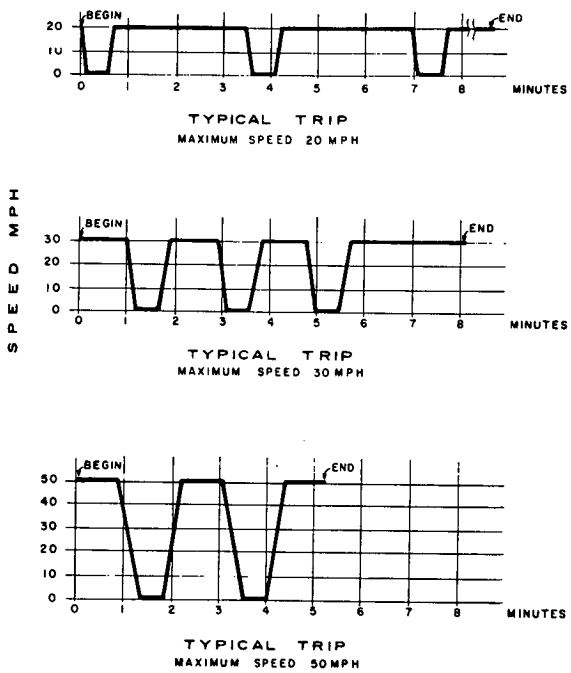


Figure A-8. Noise and mean velocity gradient, variation of speed with time for theoretical trips.

a given maximum speed. On city streets the top speed attained does not vary appreciably, and the travel time is governed principally by the number of stops. It therefore seems reasonable to expect that for any given street the mean velocity gradient would be directly proportional to the travel time.

SUMMARY

An analysis of noise and mean velocity gradient recorded on city streets in Louisville, Newark, and New York City, together with the analysis of a number of theoretical cases, indicates:

1. The noise level as measured on city streets was found to have varying relationships with travel time, depending on the street in question. Noise from Broad Street, for River Road and Fort Nelson Way southbound, and for Park Avenue appears to be virtually independent of travel time. However, there appears to be a definite relationship between noise and travel time for River Road and Fort Nelson Way northbound. It was not possible on the basis of the data for Eighth and Ninth Avenues to determine with any certainty whether there was any definite relationship between noise and travel time, as the elimination of two values would lead to the conclusion that noise is virtually independent of travel time. A study of the theoretical trips

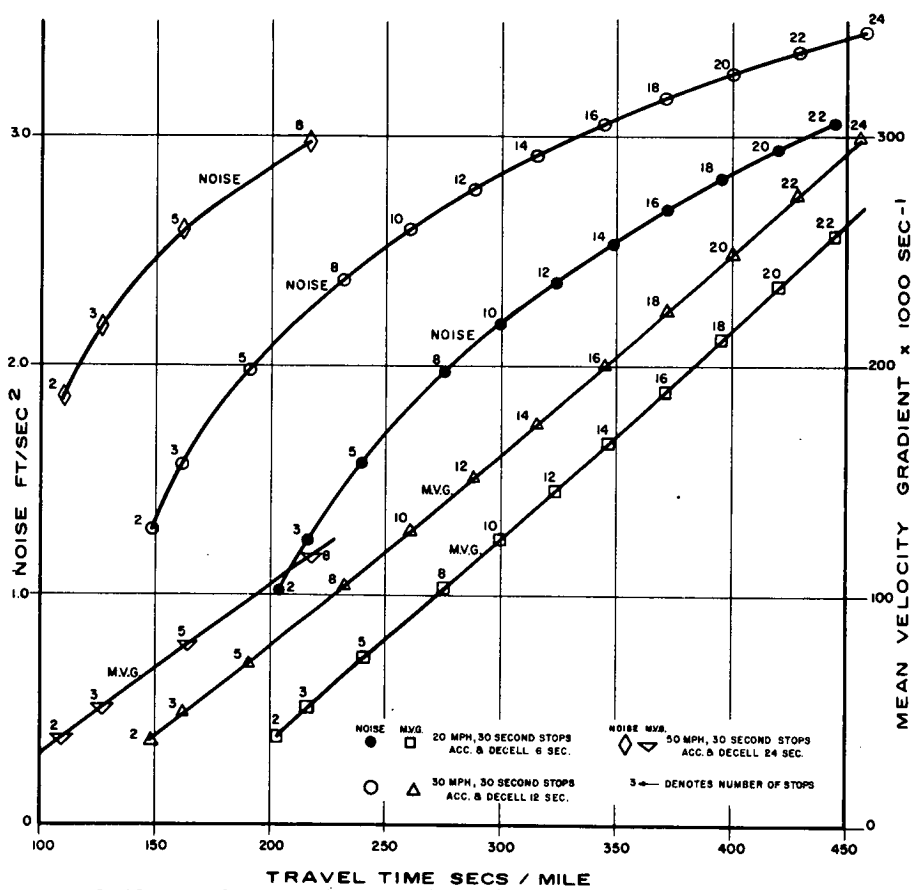


Figure A-9. Noise and mean velocity gradient, theoretical trips.

indicated that there should be a definite relationship between noise and travel time and that the noise level should increase in proportion to increased travel time. Because travel time is a direct measure of traffic flow and the relationship between noise and travel time varies so widely, noise is not considered a good measure of traffic flow or congestion.

2. Linear relationships between mean velocity gradient and travel time were established for a number of city streets; and it is reasonable to assume that, for a given street with a given type of signalization and use, the relation between mean velocity gradient and travel time is constant (Fig. A-10). Changes in traffic operations, as exemplified by the "before" and "after" conditions for River Road and Fort Nelson Way northbound, did not appreciably alter the relationship between mean velocity gradient and travel time. Whether changes in signal operation from "stop-go" to progression on any particular street system would lead to differences in mean velocity gradient of the order indicated by comparing Park Avenue data with Eighth and Ninth Avenue data (Fig. A-10) is debatable. In general, it would seem that minor changes in traffic operations will not appreciably alter the relationship between mean velocity gradient and travel time.

Mean velocity gradient is a reliable measure of traffic flow. However, on any particular city street the mean velocity gradient was found to be directly related to travel time. Minor modifications to traffic conditions on a particular street system did not alter this relationship. Therefore, there seems to be no particular advantage in using mean velocity gradient rather than the more easily measured travel and delay time to assess changes in traffic flow on a city street unless very radical changes are made to the street system.

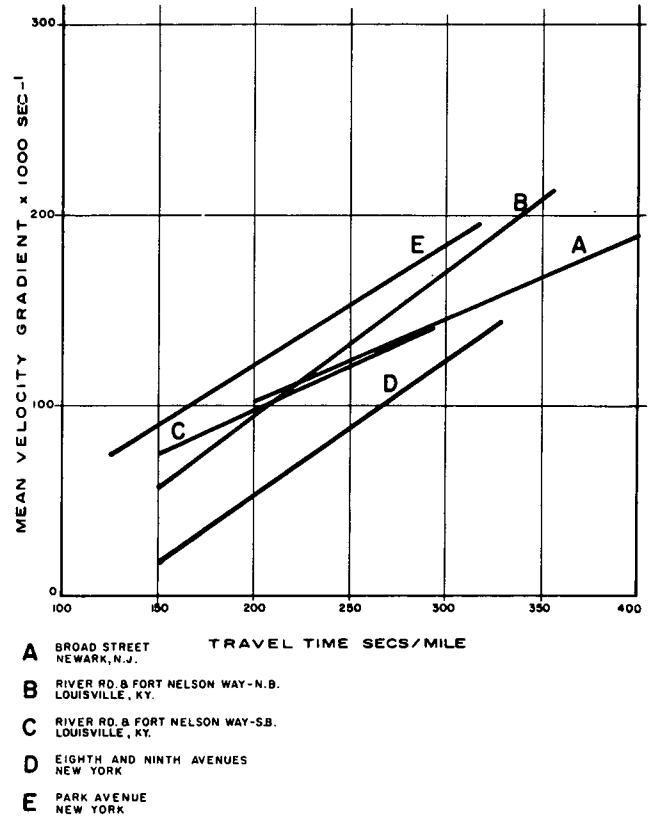


Figure A-10. Summary of mean velocity gradient versus travel time.

APPENDIX B

VALIDATION OF THE NEWELL INTERSECTION DELAY MODEL

In a congested street grid, considerable opportunity for the improvement of traffic flow often lies in the optimization of the traffic signal settings at intersections. Where signals are interconnected to offer a progressive green wave for an arterial, there is little scope for individual intersection improvement beyond fine tuning for minimum waste in serving both the main and cross traffic. However, for isolated intersections and for those arterials where green wave platoons cannot be maintained, there is scope for locally optimizing the intersection performance, with due attention given to both the mainstream and the sidestream traffic. It is for these intersections that a reasonably accurate delay model can be useful.

Measurements of vehicular delay at an intersection proved difficult to obtain through direct field observations during the experimental phase of this project. Numerous observations are necessary for statistical reliability, and considerable judgment by the observer is often required. A proven technique for estimating delay at an intersection, using parameters that can be measured without great difficulty, would be most useful to evaluate intersection improvements.

A number of existing intersection models were considered for this purpose, notably those by Clayton, Webster, Miller, Gazis, and Newell. The Newell model (*6*) was selected for testing against actual intersection performance as measured during the experimental program of this project. The model is straightforward, it requires parameters that can be measured or estimated without great difficulty, and it gives promise of reasonable accuracy for intersections in the typical range of interest; namely, where there is sufficient traffic to produce some backups.

DERIVATION OF THE NEWELL MODEL

The following derivation is simplified from Newell's original text (*6*), which should be consulted for further theoretical development. The derivation applies to traffic reaching the intersection from one direction. The superposition of opposing traffic streams can be based on the results. Let

R = time duration of red phase of signal cycle;

G = time duration of green phase of signal cycle;

Note: $(R + G)$ = total duration of one signal cycle. It is assumed that the amber phase is apportioned between red and green. For very aggressive traffic, all of the amber time would be assigned to the green phase;

$F = \left(\frac{G}{R+G}\right)$ = fraction of the time that the intersection is open to flow in the studied direction;

q = mean arrival rate in vehicles per unit time;

s = mean service rate in vehicles per unit *green* time. *Note:* The service rate (perhaps better called the service capacity) is the rate at which vehicles are served when the intersection approach is constantly busy during the green phase;

A = the number of arrivals in a cycle. Over a large number of cycles, this has mean value $\bar{A}$ and variance $\mathrm{Var}(A)$;

D = the number of departures through the intersection during a cycle. Over a large number of cycles, this has mean value $\bar{D}$ and variance $\mathrm{Var}(D)$;

$I = \frac{\mathrm{Var}(A)}{\bar{A}} + \frac{\mathrm{Var}(D)}{\bar{D}}$ = composite arrival-departure coefficient of variability;

$\bar{w}$ = average delay time per vehicle, due to queueing for the signal light;

$\bar{Q}(0)$ = average queue size for stopped vehicles at the beginning of the red phase; and

$\bar{Q}(R)$ = average queue size for stopped vehicles at the end of the red phase.

The Newell derivation for $\bar{w}$ is an approximation wherein $\bar{w} = \bar{w}_1 + \bar{w}_2$ and

$\bar{w}_1$ = the average delay per vehicle, assuming that all vehicles are served during the green phase so $\bar{Q}(0) = 0$; and

$\bar{w}_2$ = the added average delay per vehicle resulting from the fact that $\bar{Q}(0) \neq 0$ so that some vehicles are delayed for an extra cycle.

Derivation of $\bar{w}_1$

Assume that vehicle arrivals are not correlated with signal timing. Then the average number of vehicles arriving during one red phase is qR. The average arrival time is the middle of the red phase, so that the average delay during the red phase is $(R/2)$. The total average delay for all qR vehicles, during the red phase, is $(qR^2/2)$.

Suppose time $t = 0$ at the end of the red phase. The average number of vehicles waiting at $t = 0$ is qR, the average number of arrivals during the red phase. During the green phase, cars arrive at a rate q. As long as there is a queue, they are served at the rate s. Hence, the number of cars waiting at time t (as long as there still is a queue) is

$$qR - (s - q)t \tag{B-1}$$

The queue disappears at that value of time for which this expression is equal to zero; namely, at $t = (qR)/(s - q)$. Because the reduction of queue size with time is linear in this approximation, the average delay for the queue is one-half of the time from the beginning of green until the queue disappears. Thus, the average delay is $(qR)/2(s - q)$. Because the queue size at the beginning of green is qR, the total delay during the green phase is $(q^2R^2)/2(s - q)$.

The total average delay per cycle is the sum of the total during the red phase plus the total during the green phase; namely:

$$\frac{qR^2}{2} + \frac{q^2R^2}{2(s-q)} = \frac{qR^2}{2(1 - q/s)} \tag{B-2}$$

The total number of cars arriving per cycle is $q(R + G)$. Therefore, the average delay per car is

$$\bar{w}_1 = \frac{R^2}{2(R+G)(1 - q/s)} \tag{B-3}$$

Derivation of $\bar{w}_2$

Now it is assumed that $\bar{Q}(0) \neq 0$. The average number of cars waiting in the queue is shown in Figure B-1. Note that the decline in average queue size is not linear during the green phase, even though it was assumed to be for the $\bar{w}_1$ calculation. The queue size $\bar{Q}(T)$ can be broken up into two components, $\bar{Q}(R)$ and $\bar{Q}(0)$. It is the upper, shaded component of the queue that produces delay beyond that calculated in the derivation for $\bar{w}_1$. The shaded component consists of an ever-present average of $\bar{Q}(0)$ vehicles. The total delay per cycle for $\bar{Q}(0)$ waiting vehicles is $\bar{Q}(0)$ times the cycle length $(R + G)$. This total delay, $(R + G)\bar{Q}(0)$, is divided by the total number of arrivals per cycle, $(R + G)q$, to obtain the average delay per vehicle due to the fact that $\bar{Q}(0) \neq 0$:

$$\bar{w}_2 = \frac{\bar{Q}(0)}{q} \tag{B-4}$$

It remains to derive $\bar{Q}(0)$. This is done approximately by means of a technique borrowed from statistical thermodynamics. Let

$F(z)$ = probability that the queue size, at $T = R + G$, is less than z; and

$F(x)$ = probability that the queue size, at $T = 0$, is less than x.

In equilibrium, these two functions, $F(z)$ and $F(x)$, should have the same form because the queue size distribution, at

any particular moment in the cycle, should not be dependent on the particular cycle considered.

Now expand $F(z)$ in a Taylor series:

$$F(z+\Delta) = F(z) + \frac{dF(z)}{dz}\Delta + \frac{1}{2!}\frac{d^2F(z)}{dz^2}\Delta^2 + \ldots \tag{B-5}$$

Let $\Delta = x - z$. Then

$$F[z+(x-z)] = F(x) = F(z) + \frac{dF(z)}{dz}(x-z) + \frac{1}{2!}\frac{d^2F(z)}{dz^2}(x-z)^2 + \ldots \tag{B-6}$$

Note that $(x-z) = -(A-D)$, in which A is the number of arrivals and D is the number of departures during the cycle. Also $(x-z)^2 = (A-D)^2$. In Eq. B-6, replace $(x-z)$ by $-(A-D)$, truncate the series to second order, and rearrange:

$$F(x) - F(z) \doteq -(A-D)\frac{dF(z)}{dz} + \frac{(A-D)^2}{2}\frac{d^2F(z)}{dz^2} \tag{B-7}$$

Take the expected (= mean) value of this equation to get:

$$E[F(x) - F(z)] = E\left[-(A-D)\frac{dF(z)}{dz} + \frac{(A-D)^2}{2}\frac{d^2F(z)}{dz^2}\right] \tag{B-8a}$$

Because in equilibrium the two cumulative distributions $F(x)$ and $F(z)$ are the same,

$$E[F(x) - F(z)] = 0 \tag{B-8b}$$

Further

$$E[dF(z)/dz] = dF(z)/dz \tag{B-9}$$

and

$$E[d^2F(z)/dz^2] = d^2F(z)/dz^2 \tag{B-10}$$

because $F(z)$ is a distribution function and not a random variable. So

$$0 \doteq -E(A-D)\frac{dF(z)}{dz} + E[(A-D)^2]\frac{d^2F(z)}{dz^2} \tag{B-11}$$

The solution to this linear differential equation is

$$F(z) \doteq 1 - e^{-Bz} \tag{B-12}$$

with constant of integration B evaluated from the boundary conditions $F(\infty) = 1$ and $F(0) = 0$:

$$B = -2\frac{E(A-D)}{E[(A-D)^2]} \tag{B-13}$$

In equilibrium, the mean number waiting in the queue at $T = 0$ is the same as the mean number waiting at $T = R + G$. This is given by

$$\bar{Q}(0) = \bar{Q}(R+G) = E(z) = \int_0^\infty \frac{dF(z)}{dz}dz = 1/B \tag{B-14}$$

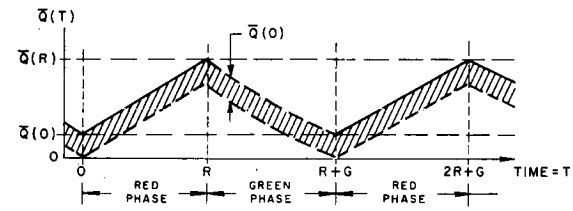


Figure B-1. Average number of cars in queue as a function of time.

Therefore,

$$\bar{Q}(0) = \frac{-E[(A-D)^2]}{2E(A-D)} \tag{B-15}$$

This derivation is for the situation where there is a substantial queue $\bar{Q}(0) >> 0$. When this is the case,

$$E(D) \doteq sG \tag{B-16}$$

In any case,

$$E(a) \doteq q(R+G) \tag{B-17}$$

So

$$\bar{Q}(0) \doteq \frac{E[(A-D)^2]}{2sG - q(R+G)} \tag{B-18}$$

Always,

$$E[(A-D)^2] = \mathrm{Var}(A-D) + [E(A-D)]^2 \tag{B-19}$$

With a substantial queue in equilibrium, the second term can be ignored. Actually, of course, $E(A-D) = 0$ in equilibrium; the situation being considered is the "pseudo-equilibrium" situation where the queue is large and, hence, $E(D)$ is minutely larger than $E(A)$. Therefore,

$$E[(A-D)^2] \doteq \mathrm{Var}(A-D) = \mathrm{Var}(A) + \mathrm{Var}(D) \doteq \left[\frac{\mathrm{Var}(A)}{q(R+G)} + \frac{\mathrm{Var}(D)}{q(R+G)}\right]q(R+G) = Iq(R+G) \tag{B-20}$$

in which I is the composite arrival-departure coefficient of variability, defined previously. Hence,

$$\bar{Q}(0) \doteq \frac{Iq(R+G)}{2sG - q(R+G)} = \frac{Iq}{2s}\left[\frac{G}{R+G} - \frac{q}{s}\right]^{-1} \tag{B-21}$$

So, finally, combining Eqs. B-3, B-4, and B-21:

$$\bar{w} \doteq \bar{w}_1 + \bar{w}_2 = \bar{w}_1 + \frac{\bar{Q}(0)}{q} = \frac{R^2}{2(R+G)(1-q/s)} + \frac{I}{2s}\left[\frac{q}{R+G} - \frac{q}{s}\right]^{-1} \tag{B-22}$$

USE OF THE MODEL

The equations of the previous section, for delays at a fixed-cycle signal where vehicle arrival times are not correlated with the signal timing, are summarized as follows:

1. The average number of vehicles in the queue at the beginning of the red phase is given by Eq. B-21.

2. The average delay to vehicles is given by Eq. B-22:

$$\bar{w} = \bar{w}_1 + \bar{w}_2 = \bar{w}_1 + \frac{\bar{Q}(0)}{q} = \frac{R^2}{2(R+G)(1-q/s)} + \frac{\bar{Q}(0)}{q}$$

3. The average number of vehicles in the queue at the end of the red phase is $\bar{Q}(0)$ plus the average number of arrivals during the red phase, Eq. B-23:

$$\bar{Q}(R) = \bar{Q}(0) + qR \qquad \text{(B-23)}$$

Input Parameters

For any real intersection subject to a fixed signal cycle, the terms R and G are known or are found by direct measurement. Now, suppose that traffic is observed and counted for N cycles, all of which are chosen to represent a particular level of intersection use. Thus, the N cycles might all be during the morning peak hour.

Let D_i = number of departures during the ith cycle green phase; $i = 1, 2, \ldots, N$;
Let A_i = number of arrivals during ith cycle; $i = 1, 2, \ldots, N$.

Then,

$$\bar{D} = \frac{1}{N}\sum_{i=1}^{N} D_i, \text{ and } \mathrm{Var}(D) = \frac{1}{N}\sum_{i=1}^{N} (D_i - \bar{D})^2 \qquad \text{(B-24)}$$

$$\bar{A} = \frac{1}{N}\sum_{i=1}^{N} A_i, \text{ and } \mathrm{Var}(A) = \frac{1}{N}\sum_{i=1}^{N} (A_i - \bar{A})^2 \qquad \text{(B-25)}$$

If N is small, $(N-1)$ should be used instead of N in the variance denominators.

Note: The present delay theory is correct only if the system is in statistical equilibrium, in which case $D = A$. In terms of D and A:

$$q = \frac{\bar{A}}{R+G} = \frac{\bar{D}}{R+G} \qquad \text{(B-26)}$$

$$I = \frac{\mathrm{Var}(D)}{\bar{D}} + \frac{\mathrm{Var}(A)}{\bar{A}} + \frac{\mathrm{Var}(D) + \mathrm{Var}(A)}{\bar{D}} \qquad \text{(B-27)}$$

It should be noted that I varies with q. If q is very small, $\mathrm{Var}(D)$ should be very near to $\mathrm{Var}(A)$ because a car can be served only once it arrives. However, $\mathrm{Var}(D)$ is always less than $\mathrm{Var}(A)$ because of the regularity imposed by the constant signal cycle. As the traffic increases, $\mathrm{Var}(D)$ decreases. If traffic is very heavy, and the system is near to saturation, $\mathrm{Var}(D)$ is likely to be quite small because the number of departures at each cycle will always be very near to the average maximum sG that can be handled.

It ought to be noted that, for Poisson arrivals, $\mathrm{Var}(A)/\bar{A} = 1$. If the arrival process is Poisson and if the intersection is nearly saturated, with very regular service, so $\mathrm{Var}(D) \doteq 0$, then $I = 1$. In view of the foregoing remarks, it is not quite correct to use the value I, based on one arrival rate, to predict delays for another arrival rate, particularly if the two rates differ dramatically. It would be useful to have on hand experimentally deduced correction factors for the variation of I with q at various basic types of intersections.

Measured $Q(R)$ values were used in Eqs. B-21 and B-23 to determine I for several intersection approaches. The resulting values of I were found to range from 1 to 2.

The service capability rate s presents some difficulty in observation because it is the average rate at which the intersection serves cars when there is a queue to keep supplying cars. Thus, among the N cycles observed, one must be able to select a number (say, K of them) with the property that, in each of the K selected cycles, enough cars are queued at the end of the red phase to keep the intersection constantly busy throughout the following green phase. If K such cycles exist, and if D_j is the number of departures during the jth of these K cycles, then

$$s = \frac{1}{GK}\sum_{j=1}^{K} D_j \qquad \text{(B-28)}$$

When an intersection approach is insufficiently saturated, the value must be estimated. Experimental data showed that 20 vehicles per minute green time per lane is a reasonable estimation for s.

Numerical Example

Suppose:

$G = 0.6$ min;
$R = 0.9$ min;
$q = 6.0$ vehicles per minute;
$s = 16$ vehicles per minute of green time; and
$I = 1.6$;

then Eq. B-21 gives $\bar{Q}(0) = 12.0$ as the average queue size at the beginning of the red phase. Eq. B-23 gives $\bar{Q}(R) = 17.4$ as the average queue size at the end of the red phase. Eq. B-22 gives $\bar{w} = 2.43$ minutes as the average delay.

Graphical Solution

Figures B-2 and B-3 may be useful for obtaining approximate solutions with somewhat reduced arithmetic.

Figure B-2 shows $\bar{w}_1$, the average delay for vehicles not held over to another cycle. $\bar{w}_1$ is a good approximation to the total average delay if the traffic is light. Figure B-3 shows $\bar{Q}(0)$, the average queue length at the beginning of the red phase.

To make it possible to exhibit the information compactly, it was necessary to use $F = \dfrac{G}{(R+G)}$ = fraction of the time the signal is green. Further, Figure B-2 gives the $\bar{w}_1$ component of delay in units of one signal cycle.

For the foregoing numerical example, $F = 0.4$. With $F = 0.4$, $q/s = 0.375$, Figure B-1 shows $\bar{w}_1 = 0.285$ signal cycles. Because one cycle is 1.5 min long, $\bar{w}_1 = (0.285)(1.5) = 0.43$ min. For $F = 0.4$, $q/s = 0.375$, and $I = 1.6$, Figure B-3 shows $\bar{Q}(0) = 12$.

VALIDATION AT TEST INTERSECTION

In considering which intersections would be appropriate for testing the model, those where traffic is shaped successfully by a green wave effect should be avoided, because then arrival moments are correlated with the signal phasing, not considered in this model. Intersections so saturated that the average vehicle waits more than one cycle interval should also be avoided, because the model does not consider delay beyond one extra cycle.

An appropriate and instructive intersection is at Central Avenue and West Market Street in Newark. This was observed extensively during the 7:30 to 8:30 AM peak, both before and after a change in the signal cycle. The measurements of $\overline{w}$, by test-car observations, were insufficient for a direct comparison of this quantity. However, the crucial component of the $\overline{w}$ formulation is $\overline{Q}(0)$; and because $\overline{Q}(R)$ [leading to $\overline{Q}(0)$] was observed quite accurately, reasonable tests of the model can be made by comparing the theoretical $\overline{Q}(R)$ to observed results.

Table B-1 summarizes the analysis of the Central Avenue-West Market Street intersection.

Central Avenue, Eastbound

The average queue size $\overline{Q}(R)$, at the end of the red phase, was observed to be 34.6 before and 16.7 after the change in signal timing. The model predicted corresponding values of 34.2 and 19.1. The errors are about 1 percent (fortuitously good) and 12 percent (probably more typical).

When the "before" data were used to predict the "after" $\overline{Q}(R)$, the predicted value was 16.8 versus an observed value of 16.7. The small 1-percent error is certainly fortuitous because both q and I changed between "before" and "after." The changes happened to compensate each other in such a way that the prediction error was minimized.

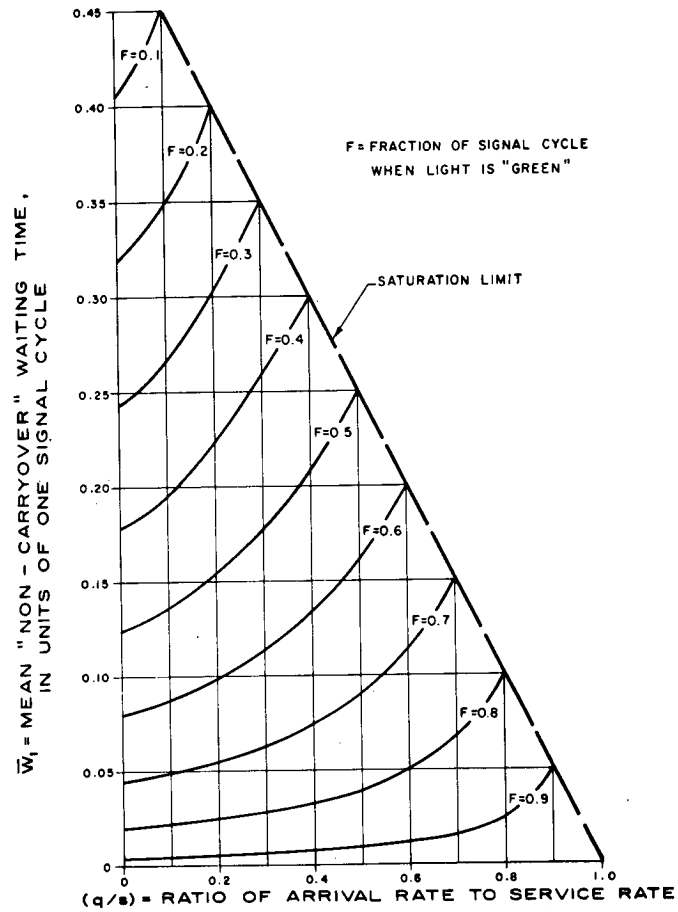


Figure B-2. Mean "non-carryover" waiting time ($\overline{W}_1$).

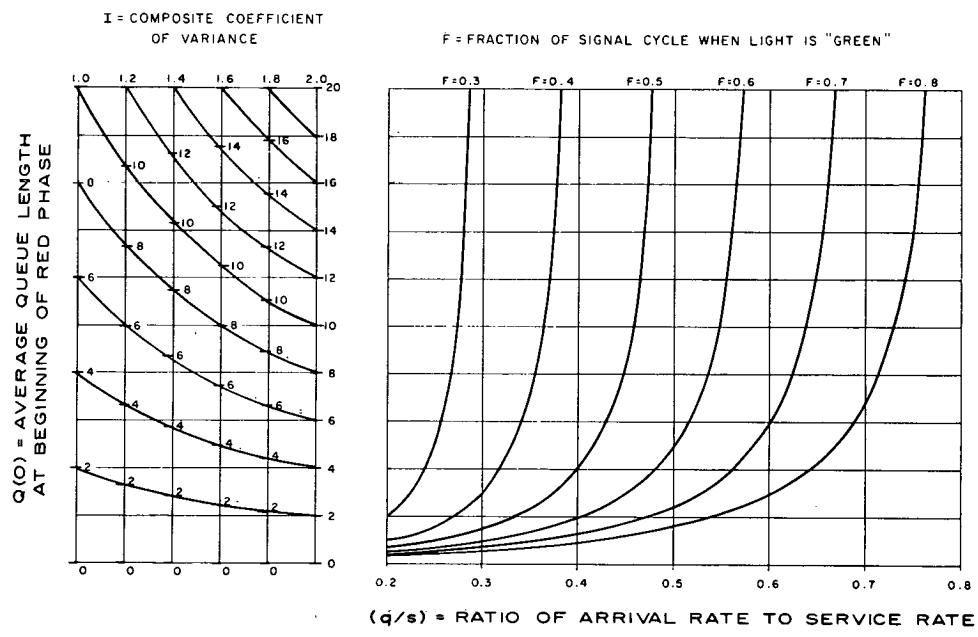


Figure B-3. Average queue length at beginning of red phase.

TABLE B-1

TRAFFIC AT THE CENTRAL AVENUE—WEST MARKET STREET INTERSECTION

ITEM	CENTRAL AVE. EB "BEFORE"	CENTRAL AVE. EB "AFTER"	WEST MARKET ST. SB "BEFORE"	WEST MARKET ST. SB "AFTER"
Observed variables for predicting congestion:				
G (min) [a]	0.72	0.99	0.72	0.45
R (min)	0.78	0.51	0.78	1.05
q (veh/min)	30.0	33.0	14.6	12.1
s (veh/min of green time)	66.8	66.8 [b]	36.1	40.2
Var $(D/\bar{D})$	0.44	0.99	0.7	0.4
I [c]	1.44	1.99	1.7	1.4
Observed congestion variable:				
$\bar{Q}(R)$	34.6	16.7	12.7	33.6
Congestion variables calculated from predictor variables:				
$\bar{w}$ (min)	0.73	0.24	0.65	— [d]
$\bar{Q}(0)$	10.8	2.3	4.5	— [d]
$\bar{Q}(R)$	34.2	19.1	15.9	— [d]
Prediction: congestion variables for "after," based on "before" values of q, s, and I:				
$\bar{w}$ (min)		0.21		∞
$\bar{Q}(0)$		1.53		∞
$\bar{Q}(R)$		16.8		∞

[a] G = green plus amber time (drivers are assumed to be aggressive).
[b] s is assumed to be the same as "before" because system was insufficiently saturated after change to deduce this quantity from the observed data.
[c] The shown values for I are based on assumption that $\mathrm{Var}(A)/A = 0.0$ (Poisson arrivals). Data were not available for $\mathrm{Var}(A)$.
[d] Too near to saturation for meaningful calculation.

Note that the improved "after" service, reducing average delay by about ½ min, handled about 10 percent more vehicles than did the "before" service. The increase in $\mathrm{Var}(D)$ from "before" to "after" is to be expected.

West Market Street, Southbound

The average queue size $\bar{Q}(R)$ at the end of the red phase was observed to be 12.7 and calculated to be 15.9 before the change in the signal cycle. The error is about 25 percent.

It was not possible to perform the corresponding "after" comparison because, with only about 18 vehicles served per cycle and an average queue size of 33.6, the average delay was on the order of about two signal cycles. The model is not correct for an average delay much beyond one cycle length.

When "before" data were used to predict "after" performance, the model indicated that "after" traffic would be too heavy for equilibrium and that the queue length would tend toward infinity. This did not happen, because the number of vehicles declined about 20 percent as a result of the reduction of green time for West Market Street. Again, $\mathrm{Var}(D)$ changed with the system saturation, as expected.

Additional Test Intersections

Procedures similar to the computations for the Central Avenue-West Market Street intersection were used to estimate $\bar{Q}(R)$ for approaches to the intersections of Central Avenue-High Street and Washington Street-Market Street. The results of these estimations are compared with observed measurements in Table B-2.

In all cases the estimated $\bar{Q}(R)$ agreed favorably with the measured value when observed "before" or "after" data were used as input to the Newell model. When the observed "before" values of q, s, and I were used as input to predict the "after" $\bar{Q}(R)$, the estimated $\bar{Q}(R)$ was less accurate.

Comparison of Delay Time

An additional test of the Newell model was performed using data acquired at the intersection of Washington Street and West Market Street during the PM peak hour for Experiment A33. This test compared the model estimated change in delay time at the Washington Street northbound approach (straight and right turns only) with test-car speed and delay data.

The theoretical delay "before" and "after" was found using Eq. B-29:

$$\bar{w} = \frac{R^2}{2(R+G)\left(1-\frac{q}{s}\right)} + \frac{\bar{Q}(R) - qR}{q} \qquad \text{(B-29)}$$

which is a combination of Eqs. B-22 and B-23. Because Eq. B-29 eliminates the composite arrival-departure coefficient of variability, I, the estimated change in delay was computed directly from observed "before" and "after" field counts of $\bar{Q}(R)$, q, and s. The measured delay was obtained

TABLE B-2

SUMMARY OF $\bar{Q}(R)$ ESTIMATIONS USING NEWELL'S FORMULA

INTERSECTION APPROACH	ITEM COMPARED	MEASURED VALUE (VEH/CYCLE)	NEWELL INTERSECTION MODEL: ITEMS INPUT	NEWELL INTERSECTION MODEL: EST. VALUE (VEH/CYCLE)
Central Ave. WB at High St.	$\bar{Q}(R)$ "before"	24.7	R, G, q, s, I ("before")	21.4
	$\bar{Q}(R)$ "after"	18.3	R, G, q, s, I ("after")	18.4
	$\bar{Q}(R)$ "after"	18.3	R, G ("after"); q, s, I ("before")	15.6
Washington St. NB at West Market St.	$\bar{Q}(R)$ "before"	16.6	R, G, q, s, I ("before")	16.6
	$\bar{Q}(R)$ "after"	12.0	R, G, q, s, I ("after")	13.7
	$\bar{Q}(R)$ "after"	12.0	R, G ("after"); q, s, I ("before")	14.9
Washington St. NB at West Market St.	$\bar{Q}(R)$ "before"	7.6	R, G, q, s, I ("before")	7.8
	$\bar{Q}(R)$ "after"	5.1	R, G, q, s, I ("after")	5.6
	$\bar{Q}(R)$ "after"	5.1	R, G ("after"); q, s, I ("before")	5.7

from the Marbelite graphs of test-car speed and delay runs on Washington Street northbound. There were 40 "before" runs and 17 "after" runs. The values computed and observed are given in Table B-3.

It should be noted that the 57 test-car runs are not necessarily an accurate measure of the average delay for the 1,000 vph at this approach. The samples form two different populations, depending on whether the test vehicle was stopped. Although the Newell model estimated delay for both "before" and "after" data is considerably lower than the test-car delay, the difference in delay (0.13 min per vehicle) is surprisingly close for both methods.

OPTIMIZING SIGNAL TIMING

The validation tests of the Newell intersection delay model are satisfactory. The prediction errors seem to be reasonable, particularly in view of the limited accuracy with which the congestion parameters can be measured experimentally.

If it could be anticipated that the changes in the arrival rate q and the service rate s would remain small when $F = G/(R + G)$ is changed, and that I would remain constant, the model would serve as a useful tool to optimize fixed-cycle signal timing at an intersection so as to minimize the sum of the delays for competing streams of traffic. Actually, there is no great need for I to be constant if reasonably accurate correction factors can be developed with experience.

With two competing streams of traffic, A and B, at a fixed-cycle signalized intersection, Eq. B-22 could be used to estimate delays for traffic at each approach as a function of F_A.

$$F_A = \frac{G_A}{R_A + G_A} = \frac{R_B}{R_B + G_B} \qquad \text{(B-30)}$$

in which $R_A = G_B$ and $G_A = R_B$. The result would look somewhat like Figure B-4. The over-all average delay would be

$$\bar{w} = \frac{\bar{w}_A\, q_A + \bar{w}_B\, q_B}{q_A + q_B} \qquad \text{(B-31)}$$

which would look like Figure B-5.

TABLE B-3

WASHINGTON STREET NORTHBOUND, PM PEAK HOUR

CONDITION	R (MIN)	$\bar{Q}(R)$ (VEH/CYCLE)	q (VEH/CYCLE)	s (VEH/CYCLE)	AVERAGE DELAY, $\bar{w}$ (MIN): TEST CAR	AVERAGE DELAY, $\bar{w}$ (MIN): NEWELL
"Before"	0.90	16.6	17.2	46.5	1.07	0.49
"After"	0.84	12.0	16.4	46.5	0.94	0.36
Difference	−0.06	−4.6	−0.8	0	−0.13	−0.13

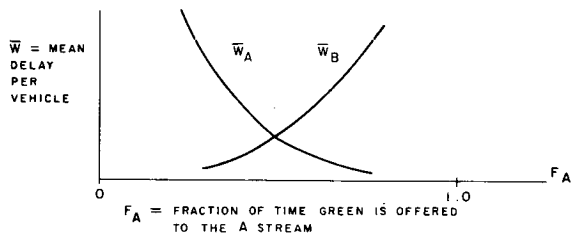


Figure B-4. Mean delay per vehicle versus fraction of green time.

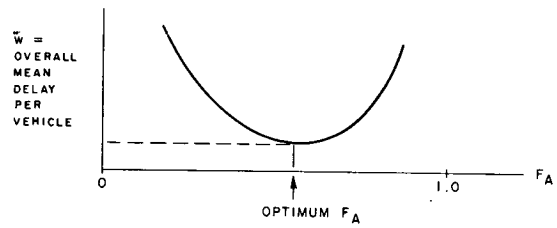


Figure B-5. Over-all mean delay per vehicle versus fraction of green time.

APPENDIX C

NETWORK ASSIGNMENT MODEL

The purpose of a downtown roadway network model is to evaluate changes of network traffic flow resulting from major operational improvements. Although it is possible to survey the entire system before and after each operational change to measure the network effects, this method becomes much too time-consuming when several major operational changes are contemplated. The objective of the Second Level Analysis was to assess the ability of an existing traffic model to estimate network changes of traffic flow using the measured local effects of an operational change as input.

Several existing traffic models were investigated to select a model that would estimate changes in network travel when an operational change produced a substantial diversion of traffic within the system. The traffic assignment process developed by the Bureau of Public Roads was selected as the most promising existing model to be tested for these purposes. The process is well-documented in the *Traffic Assignment Manual* (*8*) and the manual for *Calibrating and Testing a Gravity Model for Any Size Urban Area* (*10*). This system has been used extensively in comprehensive urban transportation studies throughout the United States, and its development for downtown areas would be a logical extension of the urban area transportation assignment package.

The following sections describe the procedures used to develop and test the network assignment model for the downtown area of Newark. The PM peak hour (4:30 to 5:30 PM) was selected as the time period for analysis. The general categories of study include gathering of input data, developing zonal trip tables, developing the network, calibrating and adjusting the network, and testing the model's ability to evaluate traffic operational changes. Figure C-1 outlines the general procedures of study.

GATHERING DATA

An extensive data-collection program was undertaken in preparation for the development of the assignment model. As Figure C-1 shows, a roadway inventory and travel time survey were required input for the network development. Volume counts and a parking inventory were used in the development of the zonal trip tables, and additional volume counts were required for the network calibration.

Roadway Inventories

A comprehensive roadway inventory of the downtown area was conducted in preparation for coding the network link map. Information collected included the length of roadway sections, width of pavement, number and direction of travel lanes, curbside regulations, and driveway locations for all streets within the study area. Traffic signal locations, stop signs, and turn prohibitions were recorded for all intersections.

Travel Time Surveys

The basic logic of the network assignment model is that all trips will select a minimum time path route of travel. Thus, the travel time input to the simulated network is critical to the success of the model.

The travel time along each section of roadway was surveyed by injecting a test car into the stream of traffic, traveling at a speed that, in the opinion of the driver, represented the average speed of all vehicles in the traffic stream. Runs taken between 4:00 and 6:00 PM were used to represent the evening peak-hour travel. A minimum of 16 runs in each direction of travel was obtained on various weekdays to develop the average travel on major roadways, and a minimum of six runs was averaged for the travel on minor streets.

The test-car travel survey was supplemented by intersection surveys to develop travel times for turning vehicles.

Time-lapse photography was used to record this information at complex intersections.

Parking Inventory

A complete reconnaissance of all off-street parking facilities within the downtown area was performed to develop the fine-grained zonal trip tables. The location, number of spaces available, and the general midday occupancy were recorded for each facility. The entrances and exits for each facility also were located. Where applicable, the number of curbside vehicles parked during the PM peak hour was recorded.

Volume Counts

A systematic volume count program was required to develop the zonal trip tables and to develop measurements to test the accuracy of the network assignment model. This program consisted of control station counts, cordon counts, screen-line counts, and various spot counts.

Road-tube detectors with 15-min-interval printed tape recorders were placed at several control stations to obtain a full 5-day-week count each month for each direction of travel at each station. A 2-day directional count, using road-tube detectors, was obtained along all roadways crossing the study-area cordon line. Additional 2-day road-tube counts and short-term manual spot counts were performed at various locations throughout the study area to develop screen-line counts and link volume counts, adjusted to represent an average weekday evening peak-hour directional volume.

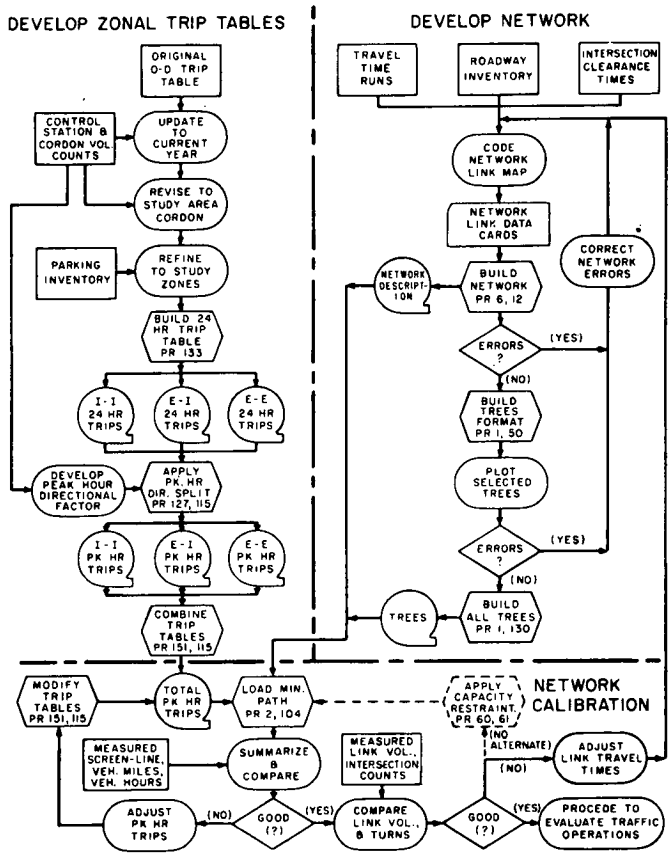


Figure C-1. Network assignment flow chart.

DEVELOPING ZONAL TRIP TABLES

An existing origin-destination trip table, derived for the 1960 Newark Transportation Study, was used as the source of information to develop a peak-hour trip table for the downtown assignment model. As Figure C-1 shows, the original transportation study trip table was updated to represent 1968 traffic within the downtown study area, refined to a fine-grained zone system and factored to represent directional peak-hour trips.

Newark Transportation Study

Trip data derived for the 1960 Newark Transportation Study were obtained from three separate surveys. An external cordon survey, conducted at 42 stations located near the city boundaries in June 1960, collected origin-destination information that represented approximately 80 percent of the total trip interchanges in Newark. A downtown parking survey, conducted concurrently with the external cordon survey, collected information for all trips beginning or ending in the central business district (CBD). An origin-destination survey, conducted by the New Jersey Highway Department in 1945, provided information on trip movements between internal Newark zones other than those of the CBD. This last element of trip information was of little interest to the current study, because these trips do not enter the CBD.

Analysis of the 1960 Newark Transportation Study indicated that 530,400 trips entered and left the Newark cordon area daily; of these, about 85,000 were external-to-external, or through trips. In addition, there were approximately 25,000 trips daily between the CBD and other zones; 7,600 intrazonal trips within the CBD; 45,600 trips between external zones, exclusive of the CBD; and 61,700 intrazonal trips in this same area.

Revising Origin-Destination Information to Study Area

The first step in the analysis of Newark Transportation Study trip data was to review the zoning to determine which areas were most suitable for development of the road simulation model. Obviously, the CBD, as described in the Newark Transportation Study, is the best area for this purpose. It not only contains most of the potential road improvements but also is the area in which parking surveys were conducted that can be used for control of trip distributions to a finer zonal base. The CBD, as defined in the Newark Transportation Study, conforms very closely to the downtown portion of the study area of this project. This downtown area is bounded on the north by the Erie-Lackawanna Railroad; on the east by the Passaic River and the Penn Central Railroad; on the south by Poinier Street; and on the west by Elizabeth Avenue, Clinton Avenue, and High Street. It includes 20 zones of the original transportation study (Fig. C-2).

The next step in data analysis was to combine zones out-

side this study area, reducing the number of components to be handled in the manipulation of trip data. Trip tables were examined in conjunction with route maps of the area to determine which trip components would cross the downtown portion of Newark. These portions of the trip tables were isolated, and the external zones contributing these trips were grouped into "drainage areas," which could be considered tributary to the major arterials. New trip tables were developed that contained only those trips of interest to the current study. Working with this simplified trip table, the various trip combinations were analyzed to allocate trips to logical points of entrance or departure from the study area.

Updating to Present Year

The number of cordon crossings from the foregoing assignment was compared and brought into balance with current average daily traffic volumes at these locations. Comparison with 1960 data indicates that trip volumes have changed very little in the interim period. Cordon-crossing trips numbered 428,000 in the current survey, as compared to 421,000 in 1960. Because the difference is so small, a single factor (1.066) was used to adjust the 1960 trip level to 1968. A trip table was developed that eliminated all reference to external zones, identifying the external trip ends by the point of cordon crossing.

Refining to Fine-Grained Study Zones

The foregoing adjustments isolated trip components of interest to the study area, based on the 20 zones of the original transportation study. These zones, however, were considered to be too large for the network simulation proposed for this analysis. The original 20 zones were divided into 136 zones for purposes of this study (Fig. C-3).

A parking inventory, in conjunction with parking data from the 1960 transportation study, furnished information for the development of factors for the distribution of trip ends within each of the 20 super-zones of the Newark Transportation Study. A review of the turnover rates and parking durations of the 1960 study indicated that facilities within each of the zones did not vary significantly in parking characteristics. Therefore, in all but two of the zones, distribution factors were based on a present count of available off-street parking spaces. Two zones (100 and 101) were primarily residential areas with a predominance of on-street parking. In these two zones, available curb parking was used in the development of zonal factors.

Three separate trip files were prepared; these, in total, represent almost 319,000 daily trips. The three trip files are: internal (I-I) trips, which begin and end within the downtown study area; external (E-I) trips, with one end inside the study area and the other end outside the study area; and through (E-E) trips, which begin and end outside the study area.

TABLE C-1

NEWARK DOWNTOWN AREA TRIP SUMMARY

TYPE OF TRIP INTERCHANGE	24-HR TRIPS ALL	24-HR TRIPS AVG/ INTER-CHANGE	PM PEAK-HOUR TRIPS ALL	PM PEAK-HOUR TRIPS % 24 HR	PM PEAK-HOUR TRIPS AVG/ INTER-CHANGE
Internal (I-I)	11,087	1.2	1,000	9.0	0.06
External (E-I)	193,826	100	15,766	8.1	4
Through (E-E)	114,052	1,077	9,646	8.5	45
All	318,965		26,412	8.3	

Factoring Trip Tables to Evening Peak Hour

The final step in developing trip information for assignment to the network was the derivation of directional peak-hour factors. Volume counts at the downtown area cordon stations were used to develop the peak-hour (4:30 to 5:30 PM) factors and the directional split of through trips and external trips. External-to-internal trip components of the 24-hr trip tables were factored to derive peak-hour trips through use of a factor developed at the point of cordon crossing or at an adjacent station. External-to-external trips were similarly factored using an average of the factors developed for the two cordon-crossing points; internal-to-internal trips were factored using an average value developed through a study of stations within the downtown area. The number of trips by trip file is given in Table C-1.

The 24-hr trip files were placed on magnetic tape, using the Bureau of Public Roads' Program PR133. The peak-hour directional trip tables were developed on magnetic tape suitable for assignment, using the Bureau of Public Roads' Programs PR127 and PR115.

NETWORK DEVELOPMENT

The general procedures followed in developing the simulated network included coding the network link map, preparing the network link data cards, building the network and correcting errors, and plotting selected trees and correcting errors (Fig. C-1). The process was repeated until all errors were removed and the network was ready for the trip assignment. The specific procedures are given in the *Traffic Assignment Manual* (*8*).

The special considerations required for the Newark downtown network are described in the following sections.

Network Link Map

A street map of the Newark downtown area (at a scale of 1 in. equals 100 ft) was used as a base for the network link map. All streets having any significance to trips of interest to this study were included in the network system. The trip-loading characteristics of each of the 136 internal zones were also the subject of considerable study. A survey of the study area was made to determine which streets afford access to the off-street parking facilities. Because on-street parking was generally banned during the evening peak hour, trip-loading at locations giving access to off-street parking facilities was considered appropriate.

The network was coded on an extremely fine-grained basis, defining each direction of travel along roadways and identifying separate turn movements at important inter-

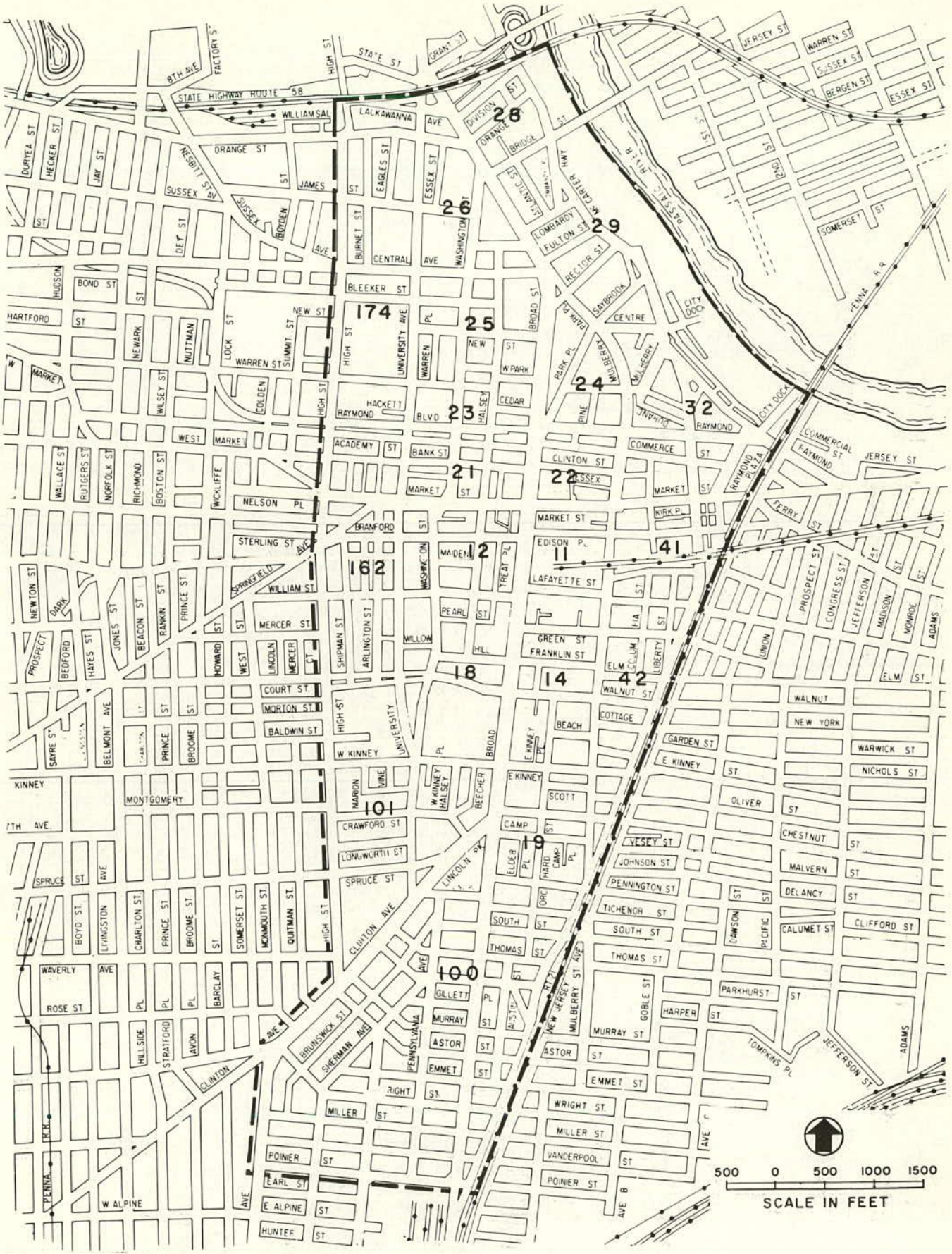


Figure C-2. Newark transportation study zones.

Figure C-3. Newark traffic assignment zones.

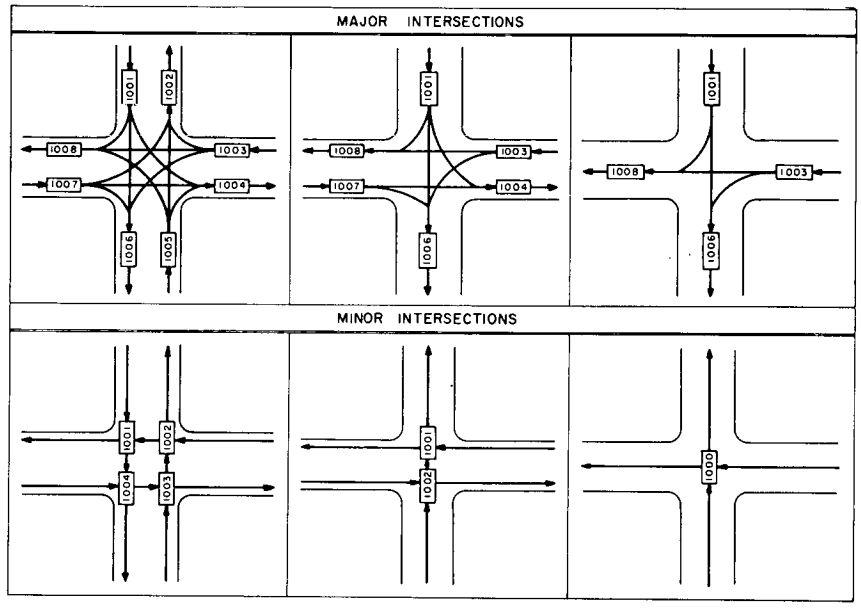


Figure C-4. Typical intersection coding.

sections. Less detail was used in coding minor intersections. Figure C-4 shows examples of the treatment in coding typical types of intersections.

A further refinement of the network coding involved the limitation of the maximum link distance and link travel time used. The link distances, scaled from the base map, were not allowed to exceed 0.63 mile. The link travel times, obtained from the travel time surveys, were not allowed to exceed 0.63 min. These restrictions eliminated the possibility of any distortion during the internal scaling process (63 being the largest number that may be described in six binary bits) of the Bureau of Public Roads' programs.

It was considered better to input distance and time information directly, rather than to input a speed and allow the computer to output time. The computer programs will accept values of link time measurements in hundredths of a minute, making possible a degree of refinement commensurate with the needs of the downtown network simulation. A sample portion of the network link map is shown in Figure C-5.

Network Link Data Cards

Preparation of the network link data followed the standard format described in the *Traffic Assignment Manual* (*8*, Chap. III). A jurisdiction code allows the assignment program to separately calculate the vehicle-miles, vehicle-hours, and average speed of trips assigned for each jurisdiction. The following jurisdiction codes were used for the Newark downtown network:

1—All major intersection turn links.
2—All Broad Street links.
3—All McCarter Highway links.
0—All other links.

The jurisdiction code, node number identifications, distance, travel time, and hourly directional volumes were coded into the appropriate card column for each network link. More than 2,000 link data cards were coded for the downtown network system.

Building the Network

After the cards had been listed and checked, the network and trees were built, using PR6, PR12, PR1, and PR50. Several selected trees were plotted to identify any network errors. After several corrections, the network was felt to be free of errors and ready for assignment to the trip files.

It should be noted that the computer programs for the Bureau of Public Roads' network assignment system had been designed for the high-speed binary IBM hardware such as the 7090 and 7094. Although some of the individual programs performed satisfactorily under emulation using the more recent IBM360 hardware, it was discovered that some programs (PR2, PR4) required a 7090/7094 computer.

NETWORK CALIBRATION

The network calibration consists of comparing the assigned results of the model with measured control values, adjusting the trip tables or link travel times, and comparing the new assigned results until the model results compare reasonably well with the measured controls.

Measured Control Values

The measured values used as controls for the network calibration were link volumes, screen-line crossing volumes, vehicle-miles of travel, and vehicle-hours of travel.

The *link volumes* were obtained from the control station counts, individual 48-hr counts, manual intersection hourly counts, and short-term manual spot counts. Each count

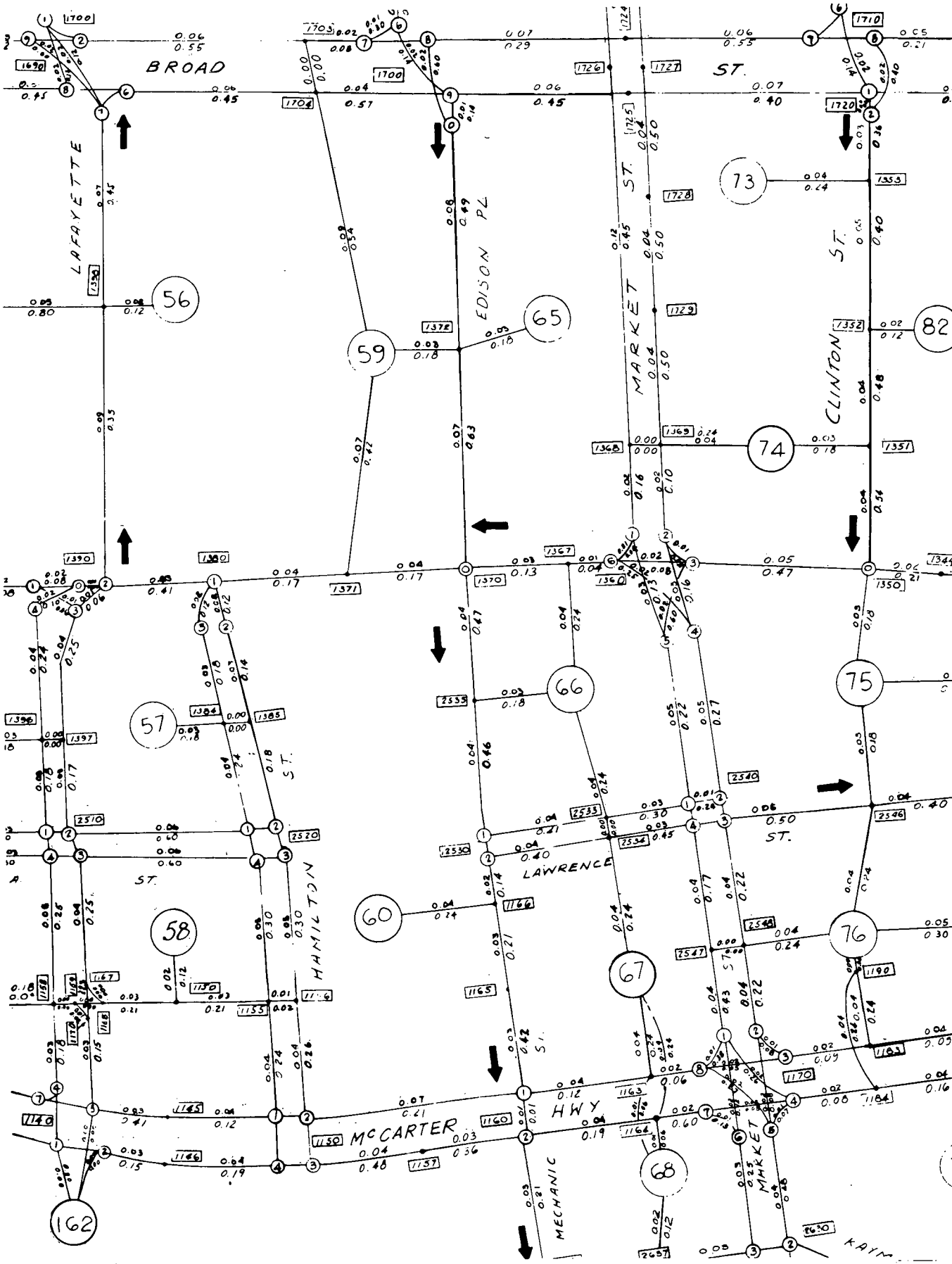
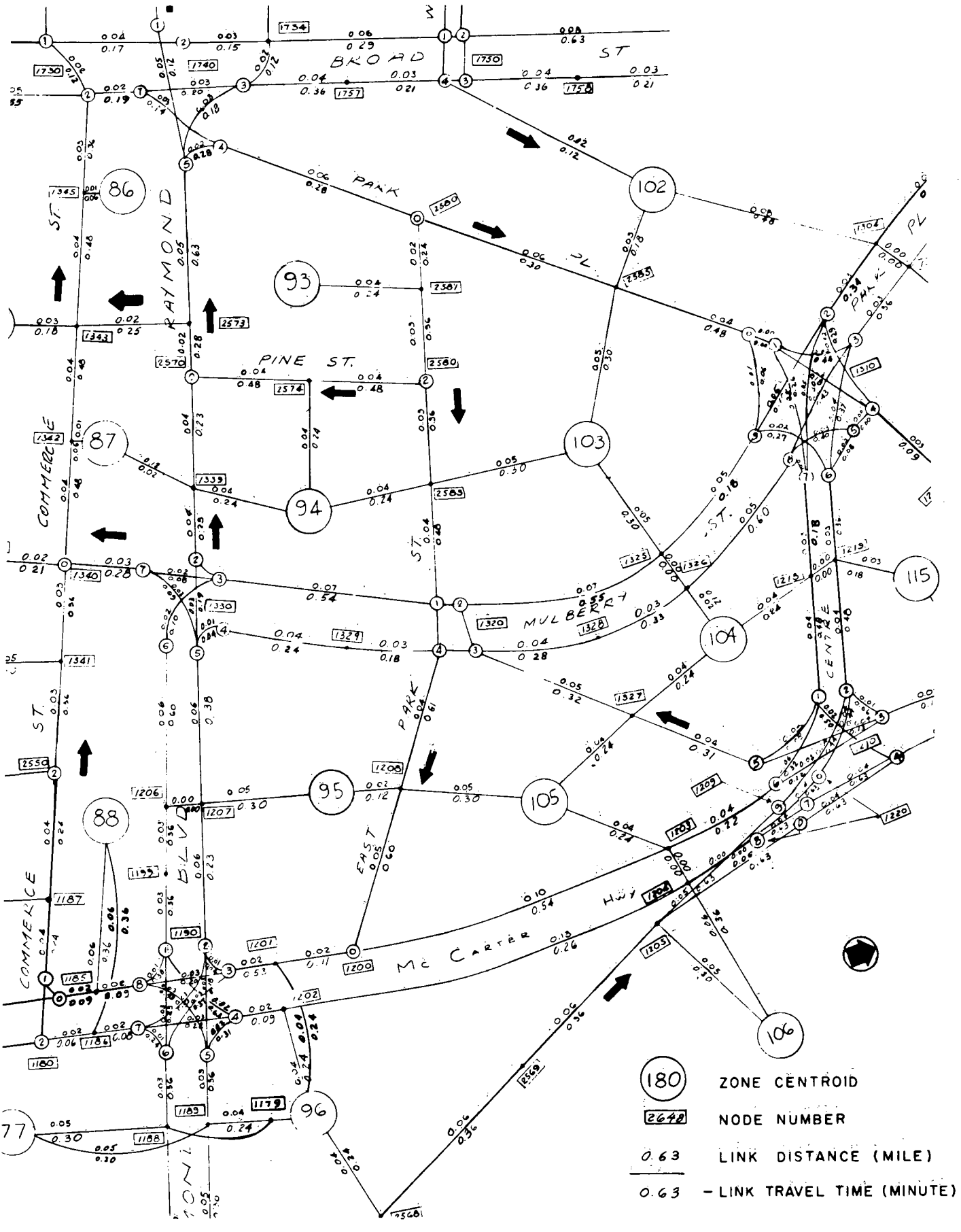


Figure C-5. Section of Newark, PM *peak-hour coded network.*


BROAD ST
PARK PL
RAYMOND ST.
PINE ST.
COMMERCE ST.
MULBERRY ST.
PARK BLVD
EAST
MC CARTER HWY
CENTRE
180 ZONE CENTROID
2642 NODE NUMBER
0.63 LINK DISTANCE (MILE)
0.63 – LINK TRAVEL TIME (MINUTE)

was adjusted to represent an average weekday evening peak-hour directional volume. The link volumes were coded on each network link data card.

The *screen-line crossings* for each of the four screen lines shown in Figure C-6 were obtained by listing the directional evening-peak-hour link volume for all roadways intercepted by the screen line. The volumes for minor streets not included in the assignment network were estimated and added to the link volume crossing.

Vehicle-miles and vehicle-hours traveled on all roadways in the system were obtained from a computer tabulation of the network link data cards. The product of the volume field and the distance field on each card produced the link vehicle-miles, and the volume multiplied by the travel time produced the vehicle-minutes for each link. The vehicle-miles and vehicle-minutes were tabulated by jurisdiction code for the entire network.

The use of the link data cards to summarize the vehicle-hours and vehicle-miles within the system introduces other potential uses of a coded network for storage of information. In addition to the distance, travel time (or speed), volume, capacity, and jurisdiction that are normally coded for the assignment procedure, other information (such as the number and type of accidents, number of lanes, pavement type, and parking regulations) could be recorded on cards similar to the link data cards for systematic reference and summarization.

Network Adjustments

Generally, the screen-line crossing comparisons serve to confirm the accuracy of the trip information, and the vehicle-mile and vehicle-hour comparisons are used to evaluate the network routings and travel times. However, because it is difficult to establish an ideal screen line in a downtown area (one with few roadway crossings, one that is straight and perpendicular to the roadway system to avoid possible "double crossings," and one that is located in an area with a minimum of turning and circulating traffic), four separate screen lines (Fig. C-6) were established. The trip table screen-line crossings, trips assigned crossing the screen lines, assigned vehicle-miles, and assigned vehicle-hours were all investigated to measure the accuracy of the trip data. The same information, together with link volumes, intersection turning volumes, minimum time paths, points of choice, specific trip interchanges, etc., was used to test and calibrate the network system.

Four separate trip assignments (three adjustments) were required to calibrate the network assignment model. A comparison of the original assigned trips with the measured screen-line crossings and vehicle-miles and vehicle-hours of travel indicated that an adjustment to the trip files was required to account for persons with a final destination in one super-zone (original Newark Transportation Study zone) who probably parked in an adjacent super-zone containing more available parking spaces. The logic of this adjustment is apparent by the comparison of zonal trip ends and number of parking spaces (Table C-2).

After comparing the link volumes of the second assignment with ground-count volumes, the network travel times on specific links were adjusted in an attempt to bring the assigned link volumes closer to the ground-count volumes.

An examination of the third assignment did not disclose any specific trip routing adjustments that would improve the assigned volumes on individual roadways. However, because the third assignment was generally high on major roadways and low on minor streets, the link travel times on some of the under-assigned minor streets were decreased for the fourth assignment. In addition, the through trips and external trips were revised for the fourth assignment. It had been assumed that the percentage of 24-hr trips moving during the evening peak hour (8.5 percent) would be the same for through trips and external trips. It is probably more logical to assume that the through trips would be more evenly distributed through the day and thus would have a lower peak-hour factor than the shorter external trips. Each cordon station was revised separately, resulting in the evening peak-hour trip files for Assignment 04 given in Table C-3.

The adjusted trip ends and the ground count for each external station are given in Table C-4.

Assignment Results

A summary of the comparisons between assigned values and measured control values is given in Table C-5. The results of Assignment 04 show a closer agreement than do the previous assignments for all comparisons with measured values. The comparison for each test is given in Tables C-6 through C-9.

The screen-line crossing comparisons (Table C-6) show that the assigned northbound trips were high for all assignments, whereas the assigned wesbound trips were generally low. The weighted percent root-mean-square error of the 102 individual links crossing the several screen lines (Table C-7) shows that Assignment 04 improved all volume groups over 100 vph and reduced the over-all error from 58 percent for Assignment 02 to 32 percent. It should be noted that the weighted root-mean-square error is not a measure of accuracy of the assignment but is a relative index of the ability to reduce the error of assignment. The vehicle-mile and vehicle-hour comparisons (Tables C-8 and C-9) show that Assignment 04 results in a better distribution of differences by jurisdiction when compared with measured values, although the assignment values for McCarter Highway (Jurisdiction 3) remained somewhat high.

Additional Adjustments

The link volumes output from Assignment 04 were recorded on a network link map and compared with ground-count directional hourly volumes. This comparison showed that the assigned volumes on major roadways such as McCarter Highway were generally higher than ground-count volumes, whereas the assigned volumes on minor roads were usually lower than ground-count volumes.

The "all-or-nothing" method of assignment can result in too many trips assigned to one roadway and too few trips assigned to a parallel route. Where drivers would actually distribute themselves over several routes between points of choice, the "all-or-nothing" method assigns all trips to the minimum path and none to the next best alternate. The

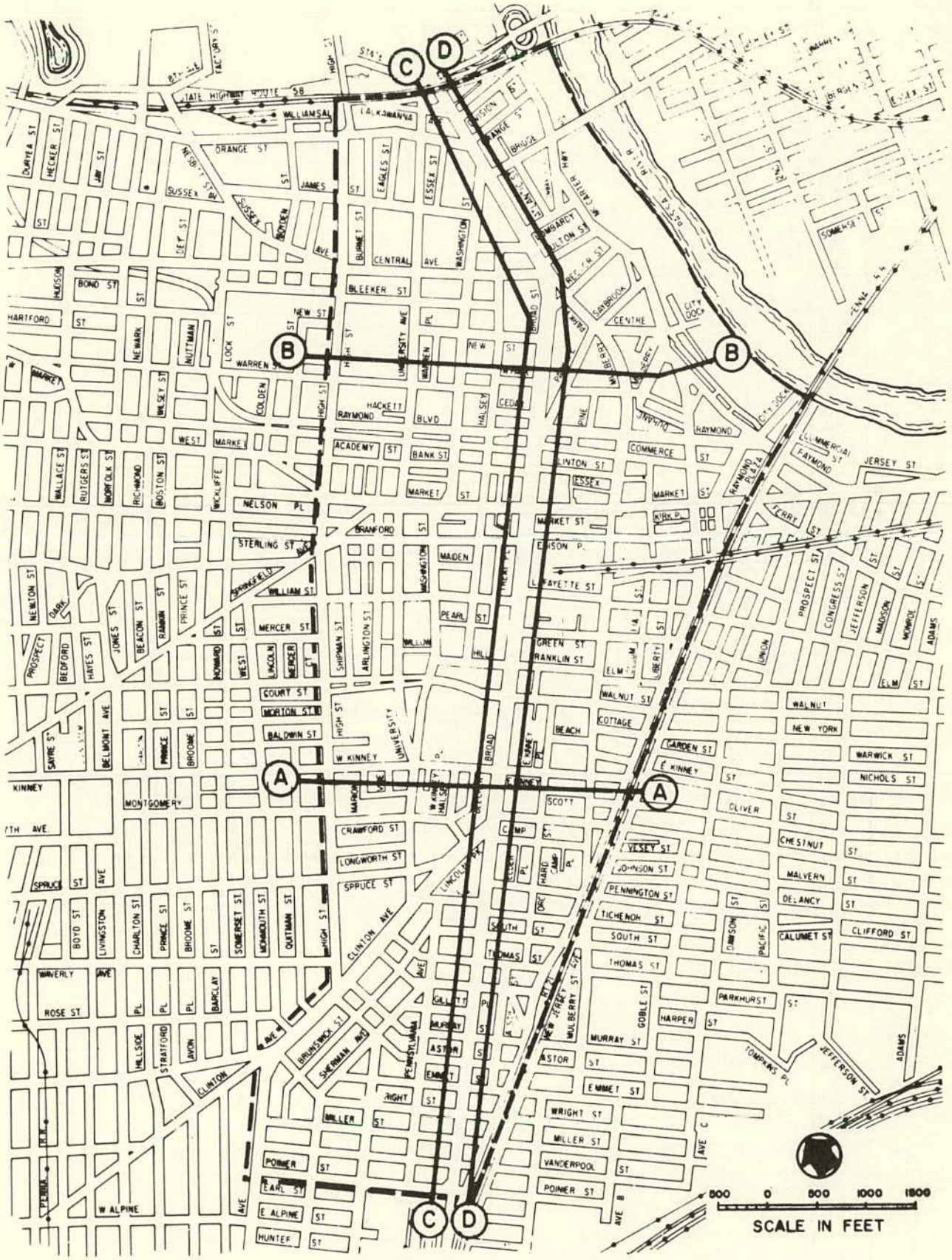


Figure C-6. Newark downtown study area, screen line locations.

TABLE C-2

NEWARK DOWNTOWN AREA, INTERNAL ZONES, NUMBER OF PARKING SPACES AND EVENING PEAK-HOUR TRIP ENDS

Zone	Parking Spaces	Trips into Zone		Trips out of Zone		Total Trip Ends		Parking Ratio
		Original	Adjusted	Original	Adjusted	Original	Adjusted	(Adjusted)
1	292	39	39	135	138	174	177	0.6
2	203	28	29	94	96	122	125	0.6
3	198	28	29	91	93	119	122	0.6
4	154	22	22	69	70	91	92	0.6
5	75	8	8	36	37	44	45	0.6
6	305	44	44	140	143	184	187	0.6
7	119	16	16	54	55	70	71	0.6
8	198	26	27	89	91	115	118	0.6
9	153	21	21	68	69	89	90	0.6
10	135	19	19	61	62	80	81	0.6
11	293	39	39	135	138	174	177	0.6
12	153	21	21	69	70	90	91	0.6
13	216	30	80	99	101	129	181	0.8
14	150	14	14	59	59	73	73	0.5
15	99	13	13	47	48	60	61	0.6
16	106	15	15	48	49	63	64	0.6
17	170	24	25	79	81	103	106	0.6
18	120	16	16	53	54	69	70	0.6
19	325	30	30	127	126	157	156	0.5
20	0	0	0	0	0	0	0	0.0
21	185	118	127	196	220	314	347	1.9
22	42	26	28	44	50	70	78	1.9
23	90	11	11	30	30	41	41	0.5
24	95	59	62	102	114	161	176	1.8
25	74	44	47	76	86	120	133	1.8
26	20	1	1	6	6	7	7	0.4
27	200	26	27	64	65	90	92	0.5
28	10	0	0	3	4	3	4	0.4
29	60	11	17	19	23	30	40	0.7
30	60	6	17	16	20	22	37	0.6
31	160	15	25	63	63	78	88	0.6
32	60	5	15	22	22	27	37	0.6
33	40	4	14	11	11	15	25	0.6
34	45	6	7	15	33	21	40	0.9
35	495	77	210	158	300	235	510	1.0
36	0	0	0	0	0	0	0	0.0
37	90	9	12	23	29	32	41	0.5
38	0	0	0	0	0	0	0	0.0
39	260	53	53	138	134	191	187	0.7
40	345	43	53	112	113	155	166	0.5
41	300	38	88	97	147	135	235	0.8
42	15	0	0	4	5	4	5	0.3
43	25	3	4	7	9	10	13	0.5
44	84	10	20	27	27	37	47	0.6
45	210	27	28	69	120	96	148	0.7
46	321	50	58	102	172	152	230	0.7
47	125	14	68	33	41	47	109	0.9
48	12	0	0	2	3	2	3	0.3
49	489	76	89	156	188	232	277	0.6
50	138	14	43	35	44	49	87	0.6
51	43	4	15	10	13	14	28	0.6
52	133	25	25	67	65	92	90	0.7
53	289	35	35	84	84	119	119	0.4
54	1055	134	330	303	303	437	633	0.6
55	193	25	25	55	105	80	130	0.7
56	405	64	79	149	238	213	317	0.8
57	0	0	0	0	0	0	0	0.0
58	110	13	16	29	36	42	52	0.5
59	288	46	57	106	184	152	241	0.8
60	304	40	48	79	97	119	145	0.5
61	70	14	14	37	36	51	50	0.7
62	207	26	81	58	58	84	139	0.7
63	40	5	5	10	25	15	30	0.8
64	0	0	0	0	0	0	0	0.0
65	105	18	22	41	52	59	74	0.7
66	170	22	26	43	54	65	80	0.5
67	26	3	4	7	9	10	13	0.5
68	0	0	0	0	0	0	0	0.0
69	638	127	126	333	323	460	449	0.7
70	150	74	78	151	157	225	235	1.6
71	0	0	0	0	0	0	0	0.0
72	45	24	25	46	47	70	72	1.6
73	45	320	23	544	49	864	72	1.6
74	0	0	0	0	0	0	0	0.0
75	285	38	89	59	168	97	257	0.9
76	375	49	119	76	217	125	336	0.9
77	350	46	110	72	197	118	307	0.9
78	200	26	62	41	110	67	172	0.9
79	550	278	291	551	571	829	862	1.6
80	0	0	0	0	0	0	0	0.0

Zone	Parking Spaces	Trips into Zone		Trips our of Zone		Total Trip Ends		Parking Ratio
		Original	Adjusted	Original	Adjusted	Original	Adjusted	(Adjusted)
81	0	0	0	0	0	0	0	0.0
82	97	691	95	1171	71	1862	166	1.7
83	90	9	29	28	27	37	56	0.6
84	16	0	0	4	4	4	4	0.3
85	0	0	0	0	0	0	0	0.0
86	0	0	0	0	0	0	0	0.0
87	500	64	153	127	296	191	449	0.9
88	275	35	86	57	152	92	238	0.9
89	392	75	73	202	198	277	271	0.7
90	256	29	78	81	79	110	157	0.6
91	812	92	91	266	259	358	350	0.4
92	0	0	0	0	0	0	0	0.0
93	0	0	0	0	0	0	0	0.0
94	370	46	142	96	227	142	369	1.0
95	475	62	151	98	262	160	413	0.9
96	315	42	99	64	173	106	272	0.9
97	40	5	14	8	22	13	36	0.9
98	210	38	37	108	106	146	143	0.7
99	190	49	48	119	116	168	164	0.9
100	100	25	24	62	61	87	85	0.9
101	230	61	60	145	152	206	212	0.9
102	1100	142	341	283	653	425	994	0.9
103	216	27	64	55	128	82	192	0.9
104	70	9	20	16	36	25	56	0.8
105	80	11	26	16	43	27	69	0.9
106	250	33	79	51	136	84	215	0.9
107	50	13	13	32	31	45	44	0.9
108	0	0	0	0	0	0	0	0.0
109	155	40	39	99	97	139	136	0.9
110	42	7	7	21	20	28	27	0.6
111	60	15	15	36	35	51	50	0.8
112	0	0	0	0	0	0	0	0.0
113	178	20	21	32	56	52	77	0.4
114	116	15	16	22	45	37	61	0.5
115	210	26	29	38	43	64	72	0.3
116	175	20	21	32	36	52	57	0.3
117	325	62	61	167	184	229	245	0.8
118	272	94	98	186	192	280	290	1.1
119	0	0	0	0	0	0	0	0.0
120	48	17	18	33	34	50	52	1.1

Zone	Parking Spaces	Trips into Zone		Trips out of Zone		Total Trip Ends		Parking Ratio
		Original	Adjusted	Original	Adjusted	Original	Adjusted	(Adjusted)
121	0	0	0	0	0	0	0	0.0
122	230	28	31	43	89	71	120	0.5
123	128	16	17	23	26	39	43	0.3
124	149	27	26	75	73	102	99	0.7
125	175	62	65	119	123	181	188	1.1
126	10	0	0	0	0	0	0	0.0
127	931	111	121	172	194	283	315	0.3
128	140	17	18	26	29	43	47	0.3
129	250	31	34	47	53	78	87	0.3
130	0	0	0	0	0	0	0	0.0
131	200	43	42	58	59	101	101	0.5
132	0	0	0	0	0	0	0	0.0
133	565	121	119	164	318	285	437	0.8
134	155	31	30	45	46	76	76	0.5
135	20	5	5	6	6	11	11	0.6
136	90	18	17	27	48	45	65	0.7
Total	24348	5039	5909	10694	11895	15733	17804	0.7

TABLE C-3

NEWARK DOWNTOWN AREA TRIP FILES

TYPE OF TRIP INTERCHANGE	24-HR TRIPS	ASSIGNMENT 03 TRIPS	ASSIGNMENT 03 % 24 HR	REVISED ASSIGNMENT 04 TRIPS	REVISED ASSIGNMENT 04 % 24 HR
Internal (I-I)	11,087	940	8.5	924	8.3
External (E-I)	193,826	16,376	8.5	17,804	9.2
Through (E-E)	114,052	9,760	8.6	9,040	7.9
All	318,965	27,076	8.5	27,768	8.7

capacity restraint computer program uses a balancing technique, where the output volumes of several iterative assignments are averaged to compensate for the limitations of the "all-or-nothing" assignment.

To confirm the assumption that the differences between assigned link volumes and ground-count volumes were due to the limitations of the "all-or-nothing" method of assignment, volume adjustments between alternate routes were performed manually, using a logic similar to the capacity restraint programs. The evening peak-hour ground count was considered to be the limiting volume, equivalent to the capacity of a roadway.

The general procedure was to identify sections of roadways overassigned and then to locate alternate routings or parallel roadways that were underassigned. The trip tables were investigated to ascertain that there were sufficient trips beween the points of choice to direct to the alternate route. The volume diverted to alternate routes was limited to the

TABLE C-4

NEWARK CORDON STATIONS, EVENING PEAK-HOUR VOLUMES

Station	Ground Count			ADJUSTED TRIP ENDS Inbound			Outbound			TOTAL		
	In	Out	Total	EE	EI	Total	EE	EI	Total	EE	EI	Total
137	783	523	1306	223	560	783	502	21	523	725	581	1306
138	1247	533	1780	385	862	1247	434	99	533	819	961	1780
139	180	71	251	27	153	180	12	59	71	39	212	251
140	307	263	570	134	173	307	263	0	263	397	173	570
141	818	399	1217	369	449	818	399	0	399	768	449	1217
142	242	216	458	131	111	242	216	0	216	347	111	458
143	109	45	154	52	57	109	45	0	45	97	57	154
144	311	0	311	210	101	311	0	0	0	210	101	311
145	1229	480	1709	195	1034	1229	88	392	480	283	1426	1709
146	460	285	745	84	376	460	104	181	255	188	557	745
147	0	202	202	0	0	0	156	46	202	156	46	202
148	165	0	165	57	108	165	0	0	0	57	108	165
149	1159	0	1159	614	545	1159	0	0	0	614	545	1159
150	246	164	410	44	202	246	148	16	164	192	218	410
151	131	92	223	0	131	131	0	92	92	0	223	223
152	990	632	1622	447	543	990	361	271	632	808	814	1622
153	102	0	102	0	102	102	0	0	0	0	102	102
154	549	366	915	218	331	549	165	201	366	383	532	915
155	502	450	952	61	441	502	242	208	450	303	649	952
156	2437	1004	3441	1288	1149	2437	409	595	1004	1697	1744	3441
157	1240	1116	2356	590	650	1240	650	466	1116	1240	1116	2356
158	618	479	1097	618	0	618	479	0	479	1097	0	1097
159	1042	1771	2813	548	494	1042	1360	411	1771	1908	905	2813
160	617	819	1436	305	312	617	271	548	819	576	860	1436
161	593	0	593	496	97	593	0	0	0	496	97	593
162	214	479	693	151	63	214	337	142	479	488	205	693
163	131	0	131	131	0	131	0	0	0	131	0	131
164	0	101	101	0	0	0	50	51	101	50	51	101
165	81	189	270	81	0	81	135	54	189	216	54	270
166	27	45	72	16	11	27	45	0	45	61	11	72
167	46	68	114	21	25	46	43	25	68	64	50	114
168	87	0	87	87	0	87	0	0	0	87	0	87
169	0	207	207	0	0	0	207	0	207	207	0	207
170	128	0	128	128	0	128	0	0	0	128	0	128
171	36	80	116	36	0	36	80	0	80	116	0	116
172	300	853	1153	259	41	300	787	66	853	1046	107	1153
173	0	120	120	0	0	0	120	0	120	120	0	120
174	141	0	141	141	0	141	0	0	0	141	0	141
175	50	72	122	50	0	50	72	0	72	122	0	122
176	29	66	95	29	0	29	66	0	66	95	0	95
177	36	62	98	36	0	36	62	0	62	98	0	98
178	2576	1726	4302	1415	1161	2576	1305	421	1726	2720	1582	4302
179	709	806	1515	86	623	709	144	662	806	230	1285	1515
180	267	165	432	5	262	267	3	162	165	8	424	432
Total	20935	14949	35884	9768	11167	20935	9760	5189	14949	19528	16356	35884

amount of overassignment, even when the alternate routes could have absorbed more traffic. In areas where all the logical alternate routes were also overassigned, no diversion or balancing was performed.

Most of the assigned volumes were brought closer to the ground-count volume as a result of the manual adjustment. Thus, it was concluded that the assignment model was calibrated within the limitations of an "all-or-nothing" assignment, and the trip tables and simulated network used for the fourth assignment were considered acceptable. A comparison of the percentage of error between assigned link volumes and ground-count volumes for the fourth assignment and after adjustment is given in Table C-10.

EVALUATION OF TRAFFIC OPERATIONAL CHANGES

Major operational changes that result in travel time changes over an extensive area can be expected to result in a substantial diversion of traffic. The effect on network flow,

TABLE C-5

SUMMARY OF ASSIGNMENT COMPARISON TESTS, NEWARK DOWNTOWN AREA, EVENING PEAK HOUR

ITEM	MEASURED VALUE	ASSIGNMENT RESULTS 01	02	03	04
Screen-line crossings:					
(I-I)	—	1,852[a]	1,852[a]	1,852[a]	1,852
(E-I) and (E-E)	—	48,538	49,858	48,650	48,008
All	49,650	50,390	51,710	50,502	49,860
Weighted % RMSE (not including I-I)	—	Not computed	58.2	48.8	31.7
Vehicle-miles:					
(I-I)	—	654[a]	654[a]	654[a]	654
(E-I) and (E-E)	—	26,886	28,236	27,853	27,101
All	26,327	27,540	28,890	28,507	27,755
Vehicle-hours:					
(I-I)	—	67[a]	67[a]	67[a]	67
(E-I) and (E-E)	—	2,636	2,731	2,770	2,699
All	2,748	2,703	2,798	2,837	2,766
Over-all speed (mph) (veh-mile ÷ veh-hour)	9.6	10.2	10.3	10.1	10.0

[a] Internal (I-I) file not assigned; value estimated from Assignment 04.

TABLE C-6

COMPARISONS OF SCREEN-LINE CROSSINGS, NEWARK DOWNTOWN AREA, EVENING PEAK HOUR

SCREEN LINE	GROUND COUNT	ASSIGNMENT 01 VOLUME	DIFFERENCE (%)	ASSIGNMENT 02 VOLUME	DIFFERENCE (%)	ASSIGNMENT 03 VOLUME	DIFFERENCE (%)	ASSIGNMENT 04 VOLUME	DIFFERENCE (%)
A—NB	4,140	4,156	+0.4	4,490	+8.5	4,310	+4.1	4,332	+4.6
A—SB	4,070	3,842	−5.6	4,386	+7.8	4,224	+3.8	4,312	+5.9
B—NB	5,760	5,800	+0.7	5,892	+2.3	5,921	+2.8	5,879	+2.1
B—SB	4,920	4,564	−7.2	4,882	−0.8	4,822	−2.0	4,805	−2.3
All NB	9,900	9,956	+0.6	10,382	+4.9	10,231	+3.3	10,211	+3.1
All SB	8,990	8,402	−6.5	9,268	−3.1	9,046	+0.6	9,117	+1.4
C—EB	5,910	6,156	+4.2	5,972	+1.0	5,725	−3.1	5,769	−2.4
C—WB	7,950	7,824	−1.6	7,524	−5.3	7,256	−8.7	7,506	−5.6
D—EB	6,370	6,460	+1.4	6,512	+3.1	6,281	−1.4	6,566	+3.1
D—WB	10,530	9,736	−7.5	10,200	−3.1	10,111	−4.0	10,605	+0.7
All EB	12,280	12,616	+2.7	12,484	+1.7	12,006	−2.2	12,335	+0.4
All WB	18,480	17,560	−5.0	17,724	−4.1	17,367	−6.0	18,111	−2.0
All crossings	49,650	48,538	−2.2	49,858	+0.4	48,650	−2.0	49,774	+0.2

TABLE C-7

WEIGHTED ROOT-MEAN-SQUARE ERROR OF ASSIGNED SCREEN-LINE CROSSINGS

VOLUME GROUP	NO. OF LINKS	AVERAGE COUNT	% OF TOTAL	ASSIGNMENT 02			ASSIGNMENT 03			ASSIGNMENT 04		
				RMSE	%RMSE	WEIGHTED	RMSE	%RMSE	WEIGHTED	RMSE	%RMSE	WEIGHTED
0–100	17	70	2.4	110	157	3.8	93	133	3.2	125	178	4.8
101–200	16	166	5.5	154	93	5.1	128	77	4.2	127	77	4.2
201–300	10	262	5.4	254	97	5.2	168	64	3.4	136	52	2.8
301–400	9	353	6.5	278	79	5.1	231	65	4.2	171	48	2.1
401–500	18	457	16.9	283	62	10.5	245	54	9.1	141	31	5.2
501–700	13	633	16.9	343	54	9.1	300	47	7.9	196	31	5.2
701–1100	9	938	17.5	424	46	8.1	278	29	5.1	186	20	3.5
1101–1800	10	1397	28.9	549	39	11.3	323	23	6.7	168	12	3.4
All	102	476	100.0	306	64	58.2	225	47	43.8	154	32	31.7

Total measured volume = 48,522.
Assignment 02 total volume = 48,783 (+0.5%).
Assignment 03 total volume = 47,415 (−2.3%).
Assignment 04 total volume = 46,096 (−5.0%).

TABLE C-8

COMPARISON OF VEHICLE-MILES ON NEWARK NETWORK, EVENING PEAK HOUR

JURISDICTION	MEASURED VALUE (VEH-MILES)	ASSIGNMENT 01		ASSIGNMENT 02		ASSIGNMENT 03		ASSIGNMENT 04	
		VEH-MILES	DIFFERENCE (%)	VEH-MILES	DIFFERENCE (%)	VEH-MILES	DIFFERENCE (%)	VEH-MILES	DIFFERENCE (%)
0—arterial roads	14,781	15,498	+4.7	14,938	+1.0	15,256	+3.1	15,579	+5.4
1—intersection turns	540	1,843	—[a]	2,091	—[a]	602	+11.4	568	+5.2
2—Broad St.	5,495	4,226	—[a]	4,426	—[a]	5,703	+3.7	5,439	−1.0
3—McCarter Hwy.	5,501	5,319	−3.3	6,781	+23.3	6,292	+14.4	6,169	+12.1
All	26,327	26,886	+2.1	28,236	+7.3	27,853	+5.7	27,755	+5.4

[a] The jurisdiction code used for intersections included through movements as well as turns on the first two assignments.

TABLE C-9

COMPARISON OF VEHICLE-HOURS ON NEWARK NETWORK, EVENING PEAK HOUR

JURISDICTION	MEASURED VALUE (VEH-HR)	ASSIGNMENT 01		ASSIGNMENT 02		ASSIGNMENT 03		ASSIGNMENT 04	
		VEH-HR	DIFFERENCE (%)	VEH-HR	DIFFERENCE (%)	VEH-HR	DIFFERENCE (%)	VEH-HR	DIFFERENCE (%)
0—arterial roads	1,635	1,589	−2.8	1,497	−8.5	1,583	−3.2	1,620	−0.9
1—intersection turns	86	200	—[a]	232	—[a]	86	0	83	−3.5
2—Broad St.	533	382	—[a]	417	—[a]	538	+0.9	522	−2.0
3—McCarter Hwy.	493	465	−5.7	585	+18.6	563	+14.2	541	+9.7
All	2,748	2,636	−4.1	2,731	−0.7	2,770	+0.8	2,766	+0.6

[a] The jurisdiction code used for intersections included through movements as well as turns on the first two assignments.

expressed in total vehicle-miles, vehicle-hours of travel, vehicle-hours of delay, and average travel speed resulting from major operational changes, could be substantial. The purpose of the network assignment model is to estimate the changes in network flow resulting from specific operational changes, using the local changes of travel time (measured in the immediate vicinity of the operational change) as input.

The model was tested by performing traffic assignments to a simulated network system representing "before" or existing conditions (Network 04) and a simulated network system representing the "after" conditions (Network 04 updated) for two experiments: Experiment B93, McCarter Highway Signal Progression; and Experiment B78, Broad Street Reversible Lanes.

TABLE C-10

LINK VOLUME AND GROUND-COUNT COMPARISONS

PERCENT ERROR (±)	PERCENT OF LINKS: ASSIGNMENT 04	PERCENT OF LINKS: MANUAL ADJUSTMENT
Within 10%	47	75
Within 20% (or ±100 vehicles)	69	88
Within 30% (or ±200 vehicles)	89	98

Experiment B93, McCarter Highway Signal Progression

Experiment B93 was used to evaluate the effect of preferential signal progression during the PM peak hour along 2.58 miles of McCarter Highway, varying in width from 50 to 87 ft, with 29 signalized intersections. For analysis, the roadway was divided into two sections, the dividing point being a central intersection from which the outbound progressions were implemented. Each section was studied by direction.

The "before" and "after" travel times for each section of roadway and direction of travel, measured from test-car travel time studies, were compiled, and the travel times to be used as input for the individual links were calculated, using the applicable link travel time factors (Table C-11).

This experiment required travel time changes for 140 link data cards. The total change in vehicle-hours of travel, assuming no diversion of trips, was computed from a tabulation of the link data cards. The vehicle-hours "before" (2,869) equal the sum of the products of the individual link volumes "before" and the link travel times "before." The vehicle-hours of travel "after" (2,775) equal the sum of the products of the individual link volumes "before" and the link travel times "after." Thus, the network change of vehicle-hours of travel, assuming no diversion of trips, was a reduction of 94 veh-hr (2,869 minus 2,775).

The change of network vehicle-hours of travel with an unlimited diversion of trips was obtained by updating the network with the new link data card travel times and performing a trip assignment. This assignment, allowing unlimited diversion, resulted in an output of 2,761 veh-hr of travel. With 2,869 veh-hr of travel before the change (from Assignment 04), the decrease of network travel time with unlimited diversion was 108 veh-hr.

Experiment data for typical links (Table C-12) indicate that the model overreacted to travel time changes when assigning link volumes. This shows the need for an assignment adjustment to obtain precise roadway volumes. The assigned volumes for the McCarter Highway change represent the maximum demand as a result of the progression change, with no regard to the capacity of McCarter Highway. The decrease of network travel time with unlimited diversion (108 veh-hr) represents the maximum benefit to be expected, provided adequate capacity exists, and the decrease with no diversion of trips (94 veh-hr) represents the minimum benefit.

Experiment B78, Broad Street Reversible Lanes

Experiment B78 was used to evaluate the effects of revising the direction of flow in specific lanes on Broad Sreet during the PM peak hour to provide additional lanes for the outbound movement. The study section is a 0.33-mile portion of Broad Street in Newark, varying in width from 62.5 to 90 ft. During the evening peak hour the center lanes were reversed in direction to provide four to six lanes northbound; the southbound direction received two lanes.

The experiment was performed in two major stages: (1) reversing the lanes only, and (2) revising the signal timing to reduce delays at congested areas and to provide a preferential progression for the outbound direction during PM peak hour. The results of the first stage were used to test the reaction of the network simulation model when both increases (in the outbound direction) and decreases (in the

TABLE C-11

McCARTER HIGHWAY SIGNAL PROGRESSION TRAVEL TIME

DIRECTION	SECTION	SPEED AND DELAY (MIN): "BEFORE"	SPEED AND DELAY (MIN): "AFTER"	MODEL (MIN): "BEFORE"	MODEL (MIN): "AFTER"	LINK TRAVEL TIME FACTORS
NB	1	5.16	5.97	6.70	7.77	1.1569
	2	7.75	3.45	7.15	3.23	0.4452
SB	1	6.59	5.05	6.80	5.21	0.7663
	2	4.48	3.98	5.87	5.21	0.8884

TABLE C-12

McCARTER HIGHWAY SIGNAL PROGRESSION

SEGMENT AND DIRECTION	LINK	VOLUME (VPH) ASSIGNMENT 04	VOLUME (VPH) MCCARTER CHANGE
A—NB	1556–1006	1730	1726
	1049–1067	1190	1459
	1153–1157	1300	1834
B—NB	1202–1204	1047	2797
	1257–1263	1270	2648
	1279–1284	1210	2282
A—SB	1283–1273	827	1146
	1259–1251	760	1244
	1203–1200	1220	1977
B—SB	1161–1252	1126	1681
	1062–1048	1250	1976
	1005–1555	1792	2425

inbound direction) of travel speed were observed. It should be noted that Experiment B78 extended beyond the network study area. Only the portion of Broad Street within the study area was used for the test.

The general procedure was to tabulate the "before" and "after" travel times for the individual link data cards (Table C-13).

The link data cards with the new "after" travel times were used to compute the saving in vehicle-hours that could be expected throughout the study area, assuming the volumes on the adjusted links remained unchanged; that is, if the change in travel time did not create any diversion of traffic. The total travel time for the "before" system was 2,869 veh-hr (Assignment 04). The total for the "after" system was 2,874 veh-hr—an increase of 5 veh-hr.

The link data cards with the "after" travel times were used to update the network; and the trips were assigned with no volume restraints, allowing an unlimited diversion of trips. The total vehicle-hours of travel output from the trip assignment to this updated network was 2,872 veh-hr, which represented an increase in total travel time of 3 veh-hr (2,872 minus 2,869).

TABLE C-13

BROAD STREET REVERSIBLE LANES

DIRECTION	MEASURED TRAVEL TIME (MIN) "BEFORE"	MEASURED TRAVEL TIME (MIN) "AFTER"	LINK TRAVEL TIME FACTOR	MODEL TRAVEL TIME (MIN) "BEFORE"	MODEL TRAVEL TIME (MIN) "AFTER"
NB	2.00	1.83	0.9176	1.81	1.74
SB	1.36	1.75	1.2862	1.95	2.51

TABLE C-14

BROAD STREET REVERSIBLE LANES, ASSIGNMENT COMPARISON

LINK	DISTANCE (MILES)	VOLUME (VPH) ASSIGNMENT 04	VOLUME (VPH) BROAD ST. CHANGES	TRAVEL TIME (MIN) ASSIGNMENT 04	TRAVEL TIME (MIN) BROAD ST. CHANGES	ΔT
1777–1774	0.05	1117	1195	0.42	0.40	−0.02
1774–1787	0.03	2022	2101	0.16	0.15	−0.01
1787–1786	0.03	2022	2101	0.16	0.15	−0.01
1786–1785	0.01	2014	2100	0.01	0.01	0
1785–1789	0.06	2039	2125	0.30	0.28	−0.02
1789–1796	0.05	2039	2125	0.24	0.22	−0.02
1796–1793	0.05	1975	2061	0.42	0.37	−0.05
1793–1806	0.04	2939	2941	0.25	0.23	−0.02
1806–1804	0.02	2365	2365	0.13	0.12	−0.01
1804–1807	0.02	724	521	0.12	0.14	+0.02
1807–1732	0.04	1003	551	0.32	0.41	+0.09
1732–1797	0.05	1003	551	0.14	0.18	+0.04
1797–1784	0.11	1380	634	0.53	0.60	+0.07
1784–1783	0.01	1346	513	0.12	0.23	+0.11
1783–1788	0.03	967	411	0.26	0.33	+0.07
1788–1773	0.04	1092	532	0.47	0.60	+0.13
1773–1778	0.03	631	356	0.11	0.14	+0.03

The model was found to overreact to travel time changes, especially when assigning volumes in the southbound direction, where the travel time increased (Table C-14). This indicates the necessity for an assignment adjustment when the model is to be used to predict actual roadway volumes resulting from specific operational changes.

Practical Uses

The previous two examples show that the network assignment model can be a useful tool to estimate the network effects of a major operational change. The assignment model will indicate the maximum demand for an improved facility and will provide an estimation of the maximum benefit to the network flow, in terms of vehicle-miles of travel, resulting from an operational improvement.

The model would be useful for estimating the total network effect resulting from a series of minor operational changes. Individually, each change may not significantly alter the network flow, but, when they are combined, could result in substantial changes of network flow. The model would also be most useful to predict network benefits when potential changes are considered, such as changes in the direction of travel or prohibiting turning movements within the network system.

The assignment model could be used as an aid in the functional classification and planning of a network system. A separate assignment of through trips or trips over a selected trip length would be useful when classifying the roadway system. Similar assignments could estimate the required travel speed for roadways to be improved as bypass routes.

The information required to build and test the network assignment model provides an invaluable insight into the traffic characteristics of an area. The coded network link map and network link data cards also provide a systematic method of recording and summarizing roadway information.

APPENDIX D

THE ANALOG TRAFFIC SIGNAL MODEL

The analog model is a tool to assist the traffic engineer with the task of network time-space diagramming for a densely signalized area such as the downtown business district of a medium-size city. Anyone who has faced this task knows the endless complications that arise in making adjustments to traffic signals in such an environment. This appendix describes the analog model used in the investigation and design of average, preferential inbound, and preferential outbound offset plans for the traffic signals of the Newark study area.

The existing Newark signal system includes 117 local intersection control units, operating traffic signals at 128 distinct intersections that are cable-interconnected to an analog computer-type master control located in City Hall.

The street system of downtown Newark includes one-way, two-way, and reversible streets, and also streets with reversible lanes in a network having both radial and grid characteristics. This results in a number of multilegged and high-angle intersections, with predominant traffic flows sometimes forced to travel indirect paths. The street system of downtown Louisville is in direct contrast, being composed primarily of evenly spaced streets forming a rectangular grid, with most of the streets operating one way.

To optimize traffic flow on a roadway having numerous signalized intersections, the traffic engineer normally will use time-space diagrams to design and plan signal-offset relationships. This optimum offset design is based on the relative importance placed on speed, volume, and delays.

Two intersecting roadways, each with a separate and distinct signal-offset plan, can easily be synchronized by this method at their common intersection without affecting either offset plan. However, where a number of roadways intersect, as in the densely signalized central business district of most cities, the limitations of this approach for network synchronization soon become apparent. A single adjustment in such an environment usually results in multiple changes to offset relationships of the network; this can be likened to the numerous outward-spreading waves resulting when a stone is thrown into a pond.

To comprehend the effects that changes in one area may have on adjacent areas often requires many hours of work constructing time-space diagrams to investigate the spreading effect of what may have initially appeared to be a minor adjustment, but which eventually was found to affect a large portion of the area.

The analog traffic signal model establishes a method by which this laborious, time-consuming task can be expedited. The main function of the model is to assist in the task of time-space diagramming by making these interacting influences visible to the traffic engineer. The influence of any local change on the network can be quickly determined. Signal analog models were built for use in both study areas. The Newark model was built first and is used as the basis for this description. The model concept probably can be best conveyed by saying that it affords the engineer a

three-dimensional medium with which to study time-space diagrams.

For the model to describe network signal offsets, the space portion of the time-space plan must include both length and depth. This was accomplished for the Newark model by gluing a map of the area on a single 4-ft by 10-ft piece of ¾-in. plywood. The size of this base was determined by the size of the study area and the scale (1 in. equals 100 ft) of the photogrammetric map used for this purpose.

It is important that the map be scale-worthy, because it provides the measurements of distance along the roadways. Both the map scale and the model base used proved to be ideal for the application intended in this experiment. The model base was mounted on three sturdy sawhorses. The base and its supports were designed to have sufficient rigidity for the construction and adjustment of the "time superstructure."

To be able to visualize the "time" portion of the model in a third dimension, a ¼-in. by 3-ft wooden dowel rod was erected at the center of each signalized intersection on the model base by drilling a ¼-in. hole through the model base and hammering the rod through this hole. It is important that the holes grip the dowels tightly enough to prevent them from sliding, and loosely enough so that they may be forced up or down for adjustment.

Before the dowels were placed in the model base, each rod was marked to represent cycles, beginning at a point a few inches from one end. Each cycle, in turn, was divided into two or more portions to represent the intervals in use. The portion of rod representing the green-signal interval for north-south traffic was colored green, and that for all east-west traffic was colored red. A third approach phase should be assigned a different color. In this model, lead-lag intervals were marked with black ink.

The scale used in marking the rods was 1 in. equals 20 percent of a cycle. By using percentage of cycle instead of seconds, a single setting of the "time" portion on these rods can represent a family of related cycle lengths and their associated offset speeds of progression. When the cycle length is changed while the distance and offsets remain constant, a change in the speed of progression will result. For instance, if, for a single setting that optimizes green-time use for the network, the distance between two signals is 400 ft, the cycle length 90 sec, and the offset 15 percent (13.5 sec), then the speed of progression is:

$$V = \frac{400 \text{ ft}}{13.5 \text{ sec}} \times \frac{30}{44} = 20.2 \text{ mph} \qquad \text{(D-1}a\text{)}$$

If the cycle length were changed to 80 sec, the 15-percent offset would equal 12 sec and the related speed would be:

$$V = \frac{400 \text{ ft}}{12 \text{ sec}} \times \frac{30}{44} = 22.8 \text{ mph} \qquad \text{(D-1}b\text{)}$$

It is possible, therefore, to develop data for a whole related group of cycle lengths and vehicle speeds without changing any setting on the model.

The trial settings are made by selecting a practical cycle length-speed combination. The speeds initially selected for various streets in the network should not be considered absolute, as minor variations in speed of progression sometimes can result in a large change in effective through-vehicle green band width.

In a similar manner, the speed and resulting green band width can often be optimized by minor cycle length changes, due to the direct relationship between cycle lengths and offset speeds. The optimum cycle length and vehicle speed combination very likely will not be the one initially assumed by the engineer but will be the single combination of this family of possible combinations that provides the best average setting, considering all network components.

The model affords the engineer a tool by which the resultant effects of such changes can be rapidly evaluated both for small cells and systemwide.

The task of coloring the dowel rods was simplified by the simple expedient of making a jig to hold the rod from a box having a hole in each side and an open end. The rod was inserted into the chuck of an electric drill in the jig and the drill was turned on. Colors were applied by holding a felt marker against the rotating rod.

The rods were then erected and strings were stretched along the green time bands in all directions. In constructing the Newark model, woolen knitting yarn was used for this purpose, both because it could be obtained in bright colors and sufficient diameter to be easily followed by the eye, and because it could be stretched slightly, permitting some adjustment of the settings without impairing the alignment of the strings. At the edges of the model, the yarn was secured to tacks driven into the base, thereby furnishing some support to maintain an erect position of the rods. Without this support the rods tend to draw inward as tension is applied to the strings. After the model is completely constructed, a certain amount of adjustment may be necessary to assure a truly vertical position of each rod. It is probably best to fasten the yarn securely at each end and make a single loop around each rod at the proper elevation, maintaining sufficient tension so that it will stay in place. The slope of the lines depicted by the yarn, therefore, has a distance abscissa and a time ordinate and becomes a measure of the speed at which the signal progression is operating.

It is probably best to begin the area study by developing the model to show the existing phasing and offsets, inasmuch as they are based on the traffic engineering experience of the past. The lower leading edge for green time on the rod representing the base signal of the system (main street green) is set in position flush with the tabletop, thereby establishing the elevation of the tabletop as the base time datum for the system. The offset relationship of every other signal in the system may then be established by measuring the appropriate time increment in percent of cycle as a distance above the tabletop.

As stated, offset plans being designed with the model use the parameters of maximum band width for a given offset speed and cycle length. Revisions to maximize band width where such changes are adaptable to present volumes should also be considered. The restraint of requiring

closely spaced intersections to operate in a basically simultaneous manner may be necessary to guard against excessive queues blocking one or more of these intersections.

The basic approach to establishing any offset plan with the model is to choose one or more major arterials for each basic direction (i.e., north-south and east-west) and synchronize the offsets between these "spines." All other less-important streets are then synchronized in the best possible way as "ribs." Such an approach reduces the time required to establish any basic offset plan on the model.

APPENDIX E

DESCRIPTION OF SURVEILLANCE EQUIPMENT

One of the objectives in the project statement is to "develop practical methods of measuring the effects of system changes." Part of the problem of developing this methodology is the selection of equipment capable of making the various flow measurements to be used to describe system changes. Description of traffic flow involves the basic measurements of volume and speed. Description of the quality of traffic movement requires, in addition, measurement of the number and duration of stops, the severity of accelerations and decelerations, delay time, speed range, and variability of travel time. In addition, special problems may require measurement of traffic queues, transit loading time, interference of turning vehicles, pedestrian interferences, etc.

Consideration of these requirements has led to the selection of various items of surveillance equipment that are described herein. In general, the experience of this project has indicated the need for versatile mobile equipment that may be used to cover the needs of many varying situations. In the initial stages of the project, consideration was given to the use of permanent counting stations, located in fixed positions on major arterials of the study areas. This concept was abandoned as it became apparent that the high cost of such installations would not be justified, considering the large number of stations required to develop sufficient information for study purposes. Instead, volume counting has been accomplished by use of automatic traffic recording equipment using the common road-tube-type installation which, while it requires considerable attention and manpower for maintenance, can be readily moved to give wider area coverage.

HUMAN OBSERVER

Probably the most versatile surveillance systems involve the human observer equipped with a stopwatch and appropriate data forms. Numerous situations require the exercise of judgment, the ability to cope with changing conditions, the measurement of items that occur infrequently, or the interaction of several flow elements. These situations require the judgment of a human observer.

Experience with several types of stopwatches indicates a need for careful consideration in the selection of this simple piece of equipment. First, the need for uniformity should be mentioned. When personnel are involved in a situation requiring very careful recording of observations taken in rapid sequence, a watch with distinctly marked, easily read intervals is necessary. A preference among survey personnel has been expressed for a stopwatch which reads 60, rather than 30, sec for each passage of the hands. It is even more important that all stopwatches used by the project should be similar in this respect to avoid the very likely error of misreading a 30-sec watch after having become accustomed to the 60-sec type. Two watches that are very efficient in situations requiring additive readings are the double timing stopwatch and the additive stopwatch. The double timing watch has two hands, one of which can be stopped to record a certain interval, while the other hand continues to advance. When the interval has been recorded, this hand may be advanced to the position of the continuously moving hand to be ready for the measurement of the next interval. This type of watch is extremely valuable in measuring such continuous operations as signal timing. The additive stopwatch has a push button that permits instantaneous return of the watch to the zero point, from which it will immediately begin the timing of a second interval. An observer can read one interval, press the button returning the watch to zero, and instantaneously begin the measurement of the second interval. In many situations, watches of this kind have eliminated the need for a second observer.

VEHICLES

Four 1968 Chevrolet Bel Air station wagons were procured for project use. Two of these vehicles were located in each study area. One was used for general project work and the other, equipped with a Marbelite Traffic Data Compiler, Model TD-1, was employed for obtaining speed and delay information. The two vehicles used for this latter purpose were Model 15635, having 200-hp V-8 engines, Powerglide transmissions, power steering, and air conditioning.

MARBELITE TRAFFIC DATA COMPILER, MODEL TD-1

The Traffic Data Compiler, designed by Jacob Greissman of the Department of Traffic of New York City, produces a graph of vehicular motion having a speed ordinate and a time abscissa (Fig. E-1). A flexible cable, coupled to the vehicle's speedometer, makes it possible to measure the vehicle's speed. This measurement is registered by a stylus moving in a vertical direction and calibrated to the scale of the special chart paper provided for this equipment. Horizontal movement is provided by a motor-drive assembly that advances the chart paper at any one of four speeds—¼ in., ½ in., 1 in., or 2 in. For uniformity, all speed and delay runs made for this project used the 1-in.-per-minute setting.

In addition to the graphic recording of vehicular speeds, individual counters record the seconds of total trip time, cumulative stop time, travel time greater than 10 mph, and travel time greater than 30 mph. An event marker button, when depressed by the operator, scribes a small dash at the bottom of the chart. In traversing a preplanned route, the operator uses this button to record predetermined checkpoints, which are usually the center lines of intersecting streets. Also, through the use of a predetermined code, the operator can quickly record the reason for delays in the space provided for such notes at the top of the chart. The exact position at which delays begin can be recorded by scribing a line along the edge of an event-recording bracket located opposite the stylus at the top of the chart in the area provided for notations. After a series of runs has been made, the chart paper is removed from the data compiler and completed in the office. This work includes scribing the center lines of streets as recorded by the event marker and filling in the street name. For project records, a Xerox copy of the chart was made on a form that provided space for appropriate headings, including the street on which the run was made, the direction of the run, the date, and time. Also, spaces were provided in which the delay time, travel time, time greater than 10 mph, time greater than 30 mph, acceleration noise, and number of delays were recorded. In addition to furnishing spaces for proper headings, the Xerox copy provided a more permanent form of the chart. This was essential, because the wax surface of the chart is easily marked by any object that touches it. Also, the wax will melt under high temperatures (90° F), destroying the chart entirely. Because of the vulnerability of these charts to high temperatures, it has been found desirable to have air conditioning in the vehicles used for this operation. It is necessary to provide sheltered parking spaces for these vehicles, because direct exposure to sunlight in an open parking area will quickly build up temperatures that will destroy the chart roll. Also, the melted wax can severely impair the functioning of delicate parts of this apparatus and affect the calibration of the machine.

The traffic data compiler has proven to be an extremely valuable device for analysis as well as surveillance purposes. Charts obtained from the data compiler are much more descriptive of actual conditions experienced in traversing the street system than are standard speed and delay runs,

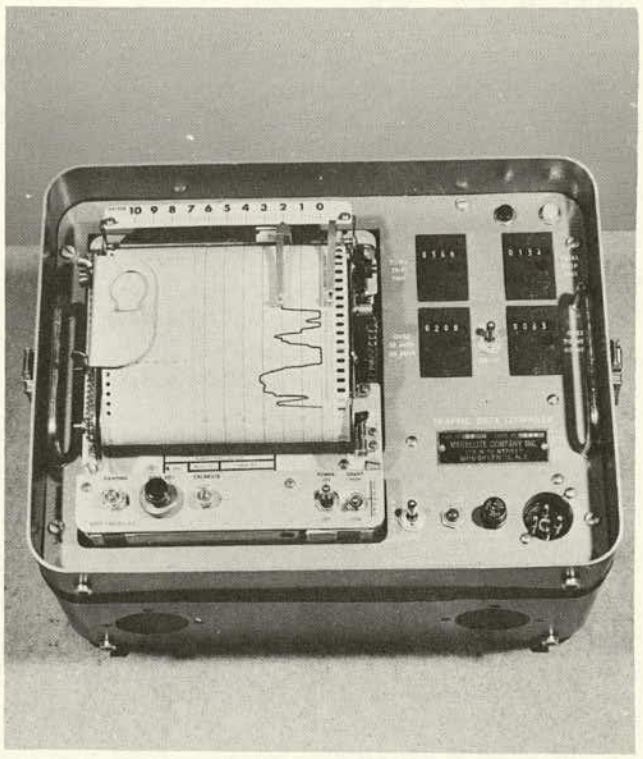


Figure E-1. Traffic data compiler.

which usually record nothing more than elapsed time between checkpoints and duration and cause of delays. In addition, the chart measures the severity of accelerations and decelerations and the smoothness of periods of sustained movement. Observation of several charts for the same run will often pinpoint deficiencies by depicting the repetitious nature of some delays.

AUTOMATIC TRAFFIC RECORDERS

Automatic traffic recording equipment purchased for use on this project was the Model RCT Traficounter, manufactured by Streeter-Amet. The RCT Traficounter prints at 15-min intervals and resets each hour. These counters were used with road-tube installations because this type of installation provided the needed flexibility and mobility. In addition, miscellaneous traffic-counting equipment was loaned to the project by Louisville, Newark, and the Kentucky State Highway Department. The traffic volume surveillance program consisted of two principal parts. First, the establishment and monitoring of a series of stations provided basic traffic flow information, data that measured daily and seasonal variations in traffic flow and developed control data for experimentation being conducted in adjacent areas. Second, "before" and "after" situations were monitored at the site of each experiment.

TIME-LAPSE PHOTOGRAPHIC EQUIPMENT

The camera used for time-lapse photography was a Beaulieu, R16ES, 16-mm movie camera with a 200-ft external magazine (Fig. E-2). This camera is capable of recording continuously for 2 hr 15 min, using 1-sec intervals. Frame speed may be varied from 2 to 64 frames per second. The camera is fully motor-driven, using 7.2 volts DC. The amount of current drawn by the camera depends on the speed and may vary from 300 milliamperes at 2 frames per second to 700 milliamperes at 64 frames per second.

Lenses used with the camera were the Angenieux 10-mm wide-angle lens, which has a horizontal lens angle of 64°, and the Angenieux 17- to 68-mm zoom lens, which has a variable horizontal lens angle of from 11° to 41°.

The DC supply was provided by six 1.25-volt Sonotone S-103, size D, rechargeable nickel cadmium batteries. These

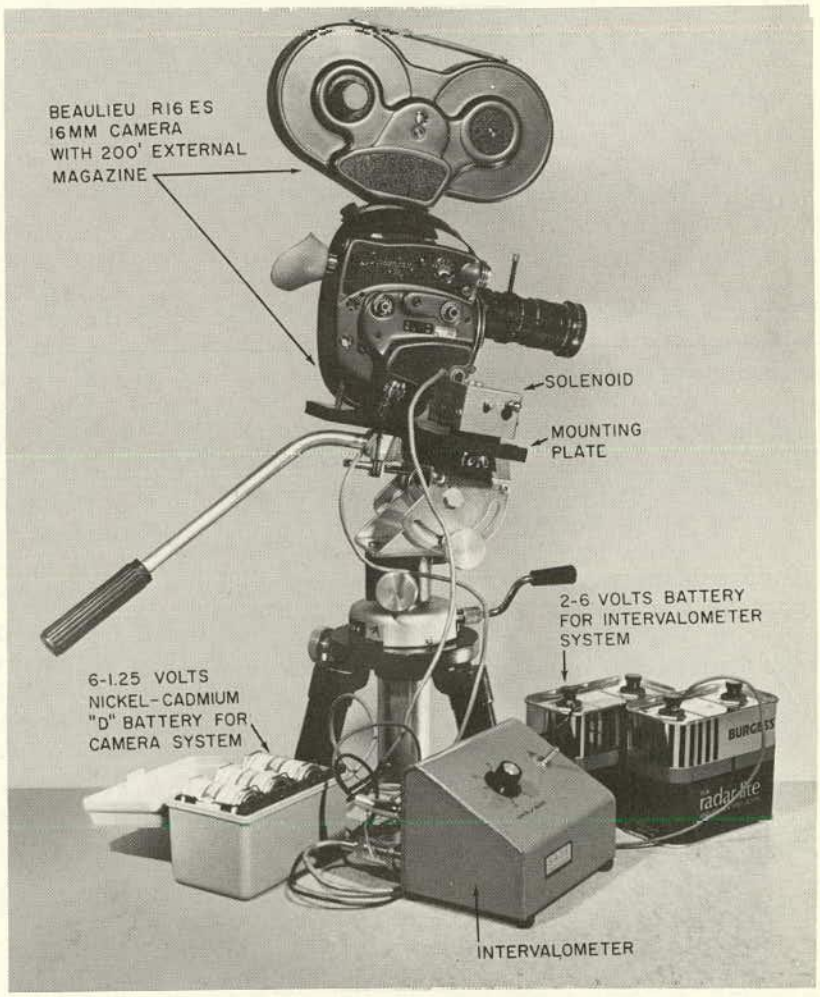


Figure E-2. Time-lapse photographic equipment.

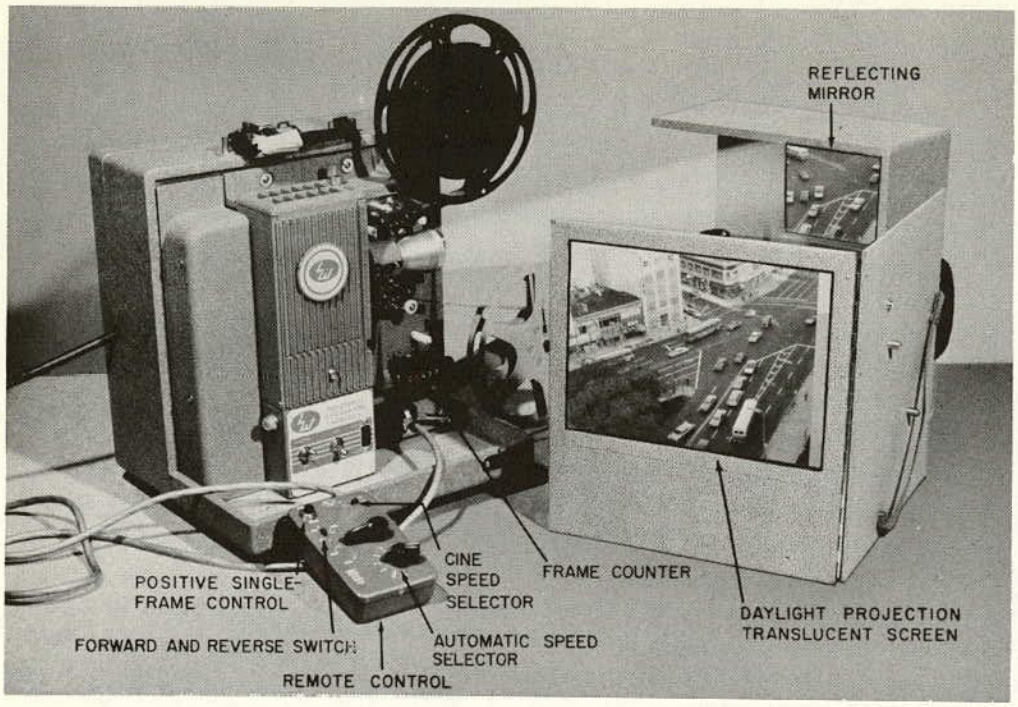


Figure E-3. Time-lapse data analysis equipment.

batteries may be completely recharged in 14 hr by use of a special battery charger.

The intervalometer used with this camera consists of two parts—a timer, custom-built for this project by Sage Instruments, Inc., 2 Spring Street, White Plains, N.Y.; and a Samenco Solenoid, built by Sample Engineering Company, 17 North Jefferson Street, Danville, Ill. The timer is battery-operated and provides a switch closure of $\frac{1}{125}$ sec at rates of from two intervals per second to one per 6 sec and is infinitely adjustable within this range. This very desirable feature makes possible field adjustment to compensate for temperature differences, level of battery charge, etc. The time accuracy of this equipment is ± 3 percent. The timer requires 400 milliamps at 12 volts DC.

Both the solenoid and camera were mounted on a plate of strong masonite die stock that could be attached to the tripod head or other mounting device from which the equipment was operated. The solenoid operates directly from the release socket, avoiding the use of cable releases which are not satisfactory for long-term continuous operation. Power supply used with this equipment was separate from the camera power supply and consisted of two 6-volt batteries.

The projector used in the analysis and data reduction phases was an L-W Model 224-A Photo Optical Data Analyzer (Fig. E-3). This equipment is a specially designed 16-mm film projector with flickerless operation and constant light intensity regardless of film speed or direction. The projector is capable of operation in both forward or reverse directions, either one frame at a time, controlled by the operator, or at various predetermined constant speeds. The built-in intervalometer provides selected speeds of 1, 2, 4, 8, 12, 16, or 24 frames per second. The projector has a five-digit frame counter that adds or subtracts, depending on the direction of film movement, and may be reset to zero by the operator. This equipment may be controlled remotely, making possible rapid manipulation of the film.

Time-lapse photographic equipment has been found to be very useful for analysis as well as surveillance purposes. In many instances wide-angle pictures covering a considerable area, taken from the top of a high building or other vantage point, provided the necessary information for analysis of particularly troublesome areas, where the interactions of many components were studied through repetitious viewing. Another use of this equipment was to present data to public officials, who were thus brought into more intimate contact with the problems of their area and were, therefore, more receptive to measures proposed for the relief of these situations.

APPENDIX F

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APPENDIX G

INDIVIDUAL EXPERIMENTS

This appendix contains reports on all experiments conducted on the project. Individual experiments are in the same order in which they are summarized in Chapter Two, and are separated into the following categories:

1. Directional Control and Lane Use.
2. Curb-Lane Controls.
3. Channelization.
4. Signal Controls.
5. Inclement Weather Effects.
6. Bus Operation.

The experiment number used for identification purposes carries a letter prefix that identifies the type and location of each experiment, as follows:

A. Local Improvement Studies, Newark.
B. Route and Network Improvement Studies, Newark.
C. Bus or Trucking Improvement Studies, Newark.
D. Local Improvement Studies, Louisville.
E. Route and Network Improvement Studies, Louisville.
F. Bus or Trucking Improvement Studies, Louisville.

DIRECTIONAL CONTROL AND LANE USE

One-Way Operation of Mellwood Avenue and Story Avenue—Experiment E30

Experiment E30 studies the effect of establishing a one-way pair of two formerly two-way roadways, Mellwood Avenue and Story Avenue, in Louisville, Ky. Both Mellwood and Story Avenues connect arterials of the northeast corridor of Louisville with those of the downtown area (Fig. G-1). Traffic in this corridor is heavy westbound in morning peak periods and heavy eastbound during the evening. This experiment was designed to measure the changes of travel time on Mellwood and Story Avenues during the morning and evening periods of heavy traffic after making Mellwood Avenue one-way eastbound and Story Avenue one-way westbound.

Experimental Area

The sections of these roadways between Baxter Avenue and Brownsboro Road, to be included as a one-way pair, are about 1 mile in length. Mellwood Avenue and Story Avenue join at Baxter Avenue and Main Street, diverge to a maximum separation of 1,200 ft, and are about 700 ft apart at Brownsboro Road. Story Avenue is 42 ft wide between Baxter Avenue and Frankfort Avenue, and 33 ft wide between Frankfort Avenue and Brownsboro Road. Mellwood Avenue is 60 ft wide from Baxter Avenue to the Louisville and Nashville Railroad crossing (about 2,000 ft east of Baxter Avenue), and 40 ft wide for the remainder of its length to Brownsboro Road.

In the "before" condition, Mellwood Avenue had two lanes operating eastbound and one lane westbound with parking permitted along the north curb. Story Avenue operated with two lanes westbound and one lane eastbound with parking permitted at the south curb except at the approach to Spring Street. Parking is also allowed on the south side of Mellwood Avenue in the 60-ft-wide section between Baxter Avenue and the railroad crossing. When operating as two-way roadways, both roadways carried a combined total of three lanes of moving traffic in each direction.

The locations of signal-controlled intersections for the "before" conditions are shown in Figure G-2.

Experimental Design

Figure G-2 shows the travel directions for both the "before" (or two-way) and the "after" (or one-way) conditions. The revised travel directions are:

1. Story Avenue—from two-way to one-way westbound between Brownsboro Road and Baxter Avenue.
2. Mellwood Avenue—from two-way to one-way eastbound between Baxter Avenue and Brownsboro Road.
3. Brownsboro Road—from two-way to one-way northbound between Mellwood Avenue and Story Avenue.

In addition to the travel pattern revisions, a traffic signal was installed at the intersection of Story Avenue and Frankfort Avenue to replace a four-way stop control.

For the "after" conditions, Story Avenue was marked for two lanes of westbound traffic between Brownsboro Road and Frankfort Avenue, and three westbound lanes between Frankfort and Baxter Avenues. Mellwood Avenue was marked for three eastbound lanes from Baxter Avenue to Brownsboro Road. Brownsboro Road, originally marked for one lane of traffic in each direction with parking permitted on both sides, was revised to two lanes northbound. The parking conditions for the one-way system were generally the same as those described for the two-way operation.

A plan for automatic traffic recorder counts and "average" car travel time and delay runs was designed to measure the difference between the two-way and one-way systems.

A water-main break on Story Avenue and malfunctions of some signal detectors on both Story and Mellwood Avenues resulted in a 9-month time difference between the "before" and "after" measurements. Unfortunately, I-64 (Fig. G-2) was opened to traffic during this period. The travel time runs were taken during the hours of 7:30 to 9:30 AM and 3:30 to 5:30 PM on the following dates:

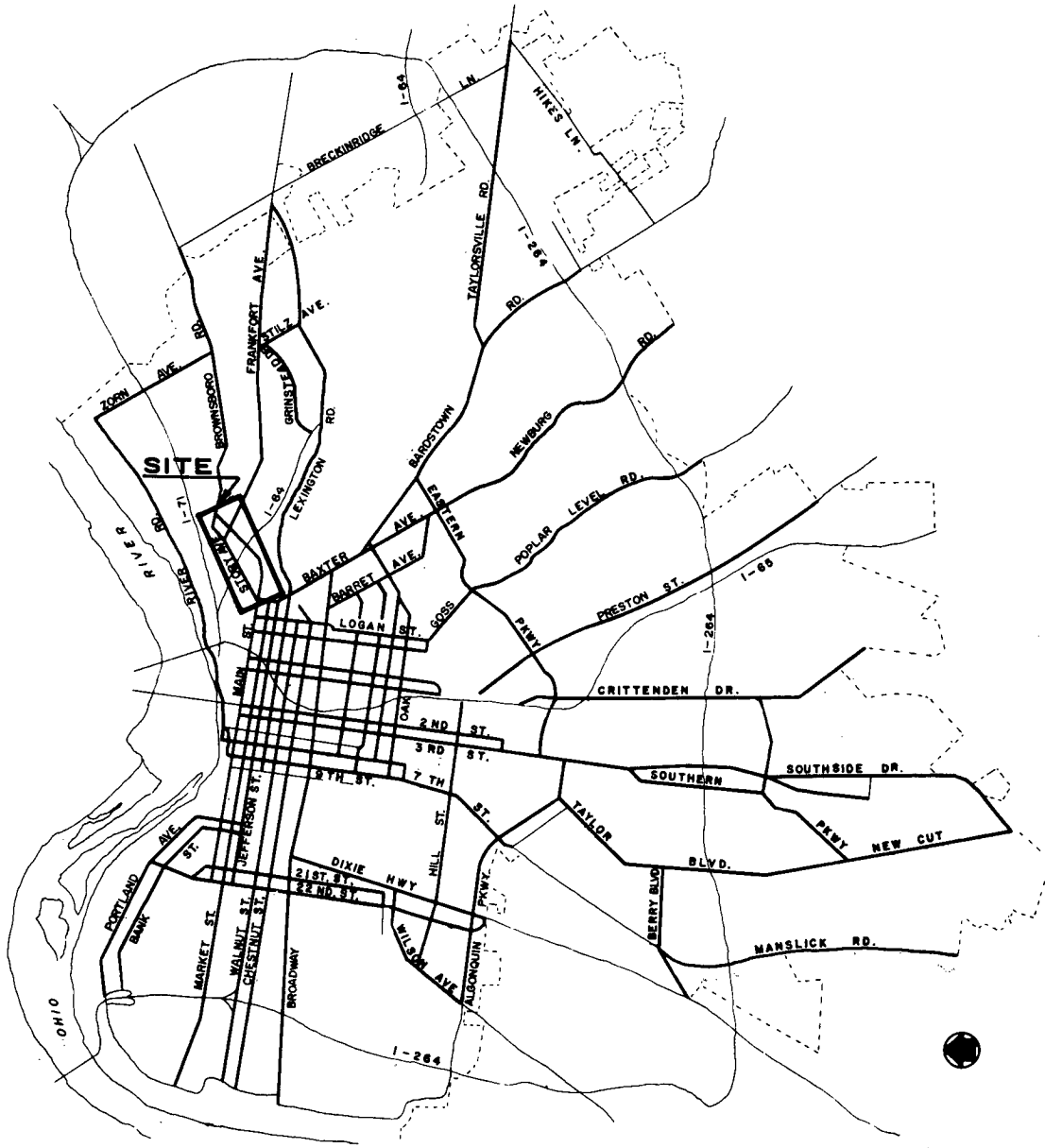


Figure G-1. Location map, Experiment E30.

DAY OF WEEK	DATES 7:30 TO 9:30 AM "BEFORE"	7:30 TO 9:30 AM "AFTER"	3:30 TO 5:30 PM "BEFORE"	3:30 TO 5:30 PM "AFTER"
Monday	5/27/68		5/27/68	
Tuesday	5/28/68		5/28/68	2/25/69
Wednesday	5/29/68		5/29/68	2/19/69
Thursday		2/20/69		2/20/69
Friday		2/21/69		2/21/69

Analysis and Conclusions

A statistical analysis of the measured "before and "after" travel time (or speeds) and volume counts was performed by time period and direction of travel. Any conclusions resulting from the analysis must consider the influence of I-64, opened during the period between measurements.

Travel Time Analysis.—The statistical analysis indicated a significant increase ($\alpha = 0.05$) of "after" travel speeds compared with "before" speeds for each time period on both Mellwood and Story Avenues (Table G-1). The "after" mean speed for the combined time periods and directions on both roadways of 23.8 mph was 28 percent greater than the "before" mean speed of 18.5 mph.

Figure G-3 shows the "before" and "after" mean speeds of each roadway for each time period and direction. Of particular interest, the "after" mean speeds for each roadway are similar for each time period, whereas there is a substantial difference by time period for the "before" mean speeds of each roadway.

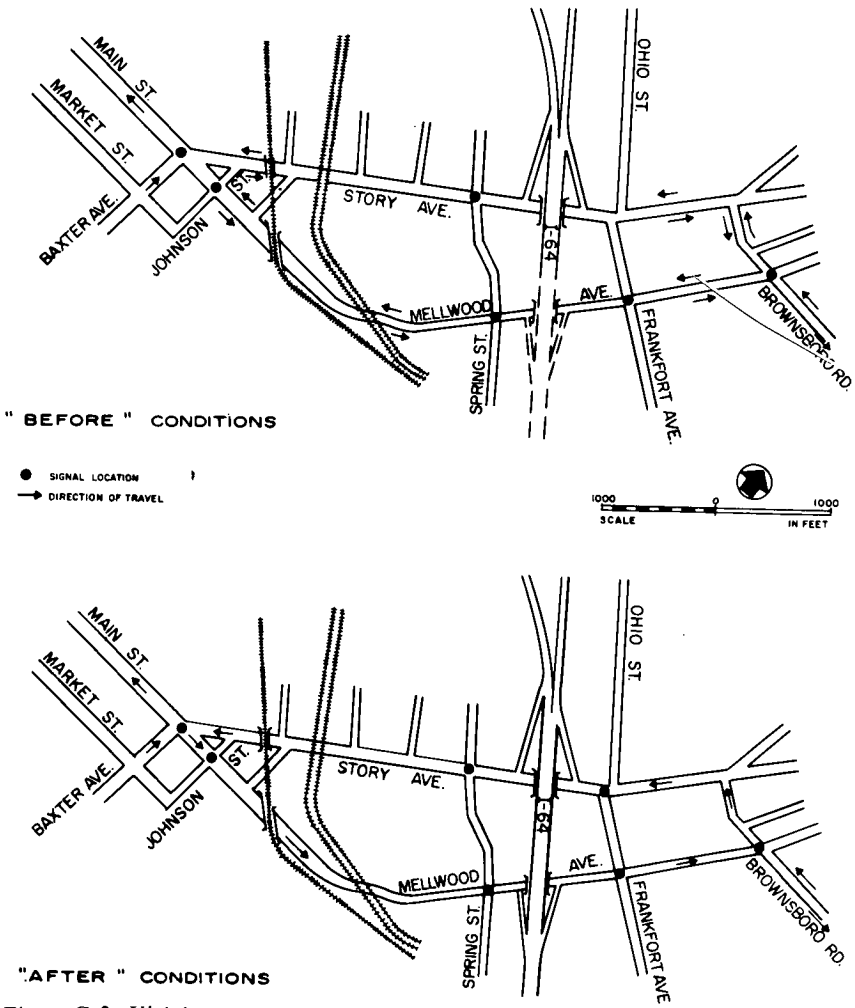


Figure G-2. Vicinity map.

TABLE G-1

SPEED AND DELAY RUN DATA

		"BEFORE"						"AFTER"					
		MELLWOOD AVE.			STORY AVE.			MELLWOOD AVE.			STORY AVE.		
DIRECTION	TIME PERIOD	NO. OF OBS.	MEAN SPEED (MPH)	VARI-ANCE (MPH)	NO. OF OBS.	MEAN SPEED (MPH)	VARI-ANCE (MPH)	NO. OF OBS.	MEAN SPEED (MPH)	VARI-ANCE (MPH)	NO. OF OBS.	VARI-ANCE (MPH)	VARI-ANCE (MPH)
EB	AM	12	16.6	7.7	10	20.9	10.5	26	20.2	9.1	—	—	—
	PM	4	17.3	20.8	3	18.1	7.7	46	20.6	9.6	—	—	—
	All	16	16.9	10.5	13	19.5	10.0	72	20.4	9.4	—	—	—
WB	AM	10	17.9	4.3	12	18.2	19.7	—	—	—	27	27.3	13.1
	PM	3	17.7	0.6	4	21.2	21.0	—	—	—	46	26.9	6.8
	All	13	17.8	3.6	16	19.7	19.9	—	—	—	73	27.1	9.1
All	AM	22	17.2	6.2	22	19.6	15.5	53[a]	2.38[a]	11.1[a]	—	—	—
	PM	7	17.5	12.7	7	19.6	15.7	92[a]	23.8[a]	8.2[a]	—	—	—
	All	29	17.4	7.5	29	19.6	15.6	145[a]	23.8[a]	9.3[a]	—	—	—

[a] Mellwood Avenue and Story Avenue combined.

TABLE G-2

AVERAGE PEAK-HOUR TRAFFIC VOLUMES

TIME PERIOD	"BEFORE" VOLUME		"AFTER" VOLUME	
	EB	WB	EB	WB
7–8 AM	742	1932	684	1336
8–9 AM	754	1668	623	1332
4–5 PM	1732	977	1318	978
5–6 PM	1509	671	1266	752

Volume Analysis.—The traffic volume counts were measured by direction on both Mellwood Avenue and Story Avenue west of Frankfort Avenue. Table G-2 gives the average peak-hour volumes by direction for both roadways combined.

The general reduction of volume for the "after" conditions must be attributed to a diversion of traffic to I-64. Figure G-4 shows the mean volumes for each direction by hour period with Tukey's limits for multiple comparison at the 95-percent level. The "before" and "after" volumes are significantly different for the 7 to 8 AM westbound volumes and the 4 to 5 PM eastbound volumes. However, the "before" and "after" volumes for the other six combinations of hour and direction are not significantly different.

Conclusions.—This experiment resulted in a significant increase of measured travel speed between the "before" and "after" conditions; however, it is possible that this increase could be due to a corresponding reduction of volume resulting from a diversion of traffic to I-64. However, Figure G-4 indicates that there is no difference between "before" and "after" volumes for the morning eastbound movements and the evening westbound movements, and Figure G-3 shows a significant increase between "before" and "after" travel speeds for these same time periods and directions. The volumes and speeds for "before" and "after" conditions are compared as follows:

TIME PERIOD	DIRECTION	VOLUME "BEFORE"	VOLUME "AFTER"	SPEED (MPH) "BEFORE"	SPEED (MPH) "AFTER"
7-9 AM	EB	1500	1310	19	20
7-9 AM	WB	3600	2670	18	27
4-6 PM	EB	3240	2580	18	21
4-6 PM	WB	1650	1730	20	27

Thus, it can be concluded that the revision of Mellwood Avenue and Story Avenue from a two-way system to a one-way pair did result in:

1. An increase of travel speed in the eastbound direction during the morning time period, from 19 to 20 mph.
2. An increase of travel speed in the westbound direction during the evening time period, from 20 to 27 mph.

The increased travel speed in the eastbound direction

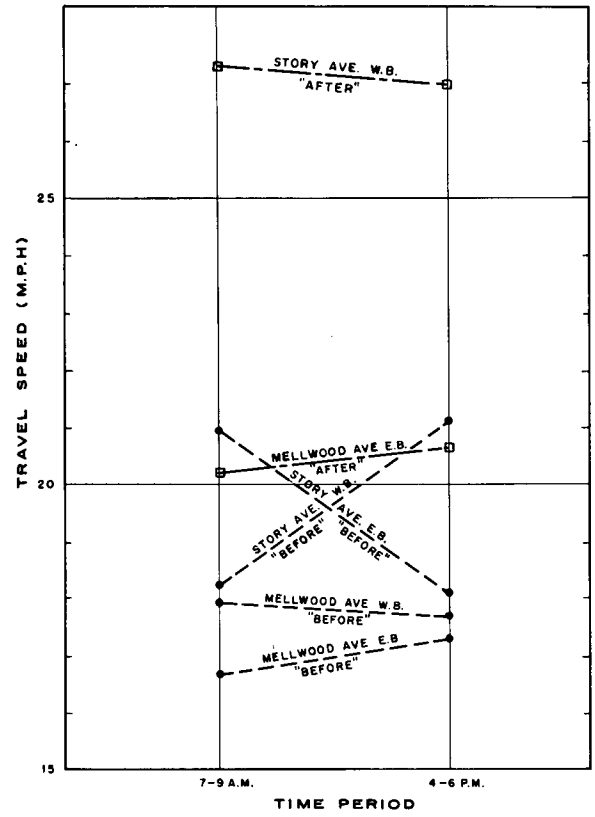


Figure G-3. Average travel speed comparison, Mellwood Avenue and Story Avenue.

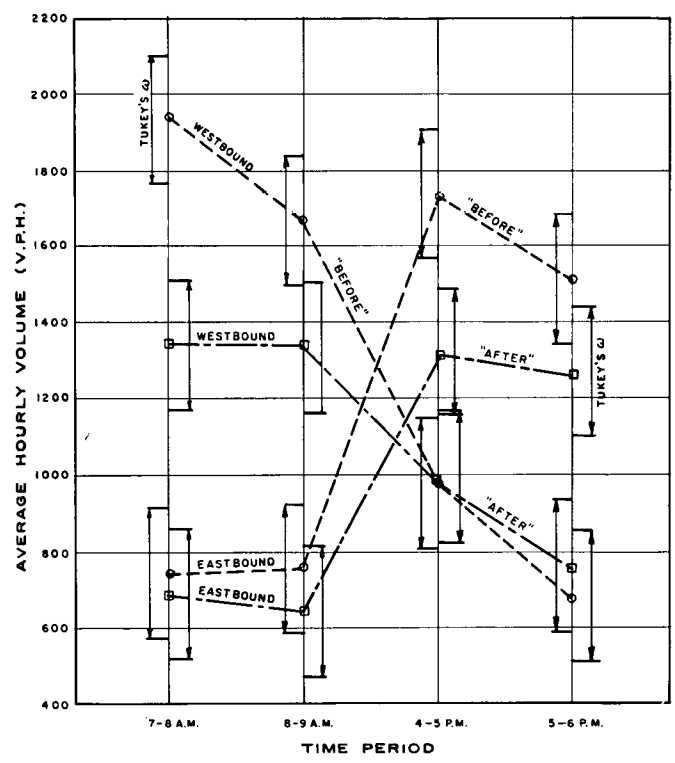


Figure G-4. Average hourly volume comparison.

during the evening time period (from 18 to 21 mph) and in the westbound direction during the morning time period (from 18 to 27 mph) is of approximately the same magnitude as that measured for the directions that displayed no volume change. This indicates that much of the increased travel speed for these elements can be attributed to the one-way operation.

Revision of One-Way Patterns on River Road and Fort Nelson Way—Experiment E31

Experiment E31 analyzes the effect of a revision to one-way roadways within a one-block area, in an attempt to reduce delays by eliminating traffic conflicts. The former one-way pattern of River Road and Fort Nelson Way in Louisville caused intersecting traffic flows at three locations. Experiment E31 proposed that the conflicts be eliminated by reversing the one-way direction of River Road and Fort Nelson Way between Fifth Street and Sixth Street. The revision produces a counterclockwise rotary movement at this location, with entrances to the rotary at River Road and at Sixth Street and exits at River Road, Fifth Street, and Fort Nelson Way (Fig. G-6). This rotary configuration is seen to have many applications in reducing delays at points where conflicting moderate flows may be separated through this expedient. The elimination of conflicts, however, requires the injection of more turns into routes through this area. The net effect of these adjustments is measured in this experiment.

Experimental Area

Experiment E31 was conducted in the area between Third and Seventh Streets, and between Main Street and River Road (Fig. G-5). Under the existing conditions, River Road was two-way between Third and Fifth Streets, and one-way eastbound from Sixth Street to Fifth Street. Fort Nelson Way was two-way between Fifth and Sixth Streets, and one-way westbound from Sixth Street to Seventh Street. Fifth Street was two-way between Fort Nelson Way and River Road, and one-way southbound from Fort Nelson Way to Main Street. Sixth Street was one-way northbound from Main Street to River Road, and Seventh Street was one-way southbound from Fort Nelson Way to Main Street.

Purpose and Scope

Experiment E31 measures the change in travel time and delays between the intersections of Main and Seventh Streets and Third Street and River Road resulting from a revision of one-way operations to reduce conflicting movements. Under existing conditions, traffic conflicts occurred at Sixth Street and Fort Nelson Way, at Fifth Street and Fort Nelson Way, and at Fifth Street and River Road. The most critical conflict occurred at Fifth Street and River Road, where the left turn from River Road westbound into Fifth Street southbound opposed the through movement on River Road eastbound, the through movement being controlled by a STOP sign.

The proposed revision makes River Road one-way westbound from Fifth Street to Sixth Street, Sixth Street one-way southbound from River Road to Fort Nelson Way, Fort Nelson Way one-way eastbound from Sixth Street to Fifth Street, and Fifth Street one-way northbound from Fort Nelson Way to River Road. The existing and proposed conditions are shown in Figure G-6.

Design of Experiment

The primary purpose of Experiment E31 was to reduce the delays at River Road and Fifth Street due to the conflict of left-turning vehicles from River Road into Fifth Street with the through traffic on River Road eastbound, which was controlled by a STOP sign. This conflict was removed by reversing the direction of River Road between Sixth Street and Fifth Street, as shown in Figure G-6. The proposed revision made possible the removal of signal controls at Fort Nelson Way and Fifth Street and eliminated the need for STOP-sign controls on River Road eastbound at Fifth Street and on Fort Nelson Way westbound at Sixth Street.

The improvement design involved some minor lane marking and painted channelization to organize the movements and the placing of four ONE WAY—DO NOT ENTER signs. In addition, signs routing all westbound and southbound trucks into Fourth Street from River Road were required due to a low-clearance bridge over Sixth Street south of River Road.

Surveillance System Design.—Speed and delay runs, using the Traffic Data Compiler to record each run taken before and after the revision, were the source of measured data for comparing the results of the revision. The measurements selected for comparison were total trip time through the area, number of stops, and total delay time en route during the morning (7:30 to 8:30 AM) and evening (4 to 5:30 PM) peak traffic periods. The patterns of travel were separated into northbound or southbound runs —the northbound runs to begin at Sixth Street and Main Street and end at Third Street and River Road, and the southbound runs to begin at Third Street and River Road and end at Seventh Street and Main Street. The different patterns to be run are shown in Figure G-7.

In addition to the speed and delay runs, 48-hr Automatic Traffic Recorder (ATR) counts were planned for six locations bordering the area; manual counts were scheduled for peak hours on all approaches to the intersection of River Road and Third Street.

Implementation of Improvement.—Advance notification of the revised one-way patterns was given to the public through the news media prior to April 22, 1968. The signing and lane marking were completed on Sunday, April 21, and police, along with personnel of the Louisville Traffic Department, were stationed at critical intersections to aid and direct motorists on Monday, April 22. There was some confusion on the first day of the revision, caused primarily by trucks that had proceeded on River Road west to Sixth Street where they were impeded by the low-clearance structure. Additional advance signs to route trucks south on Fourth Street were placed on the ramp at Third Street. By Wednesday, April 24, traffic appeared to flow smoothly through the area.

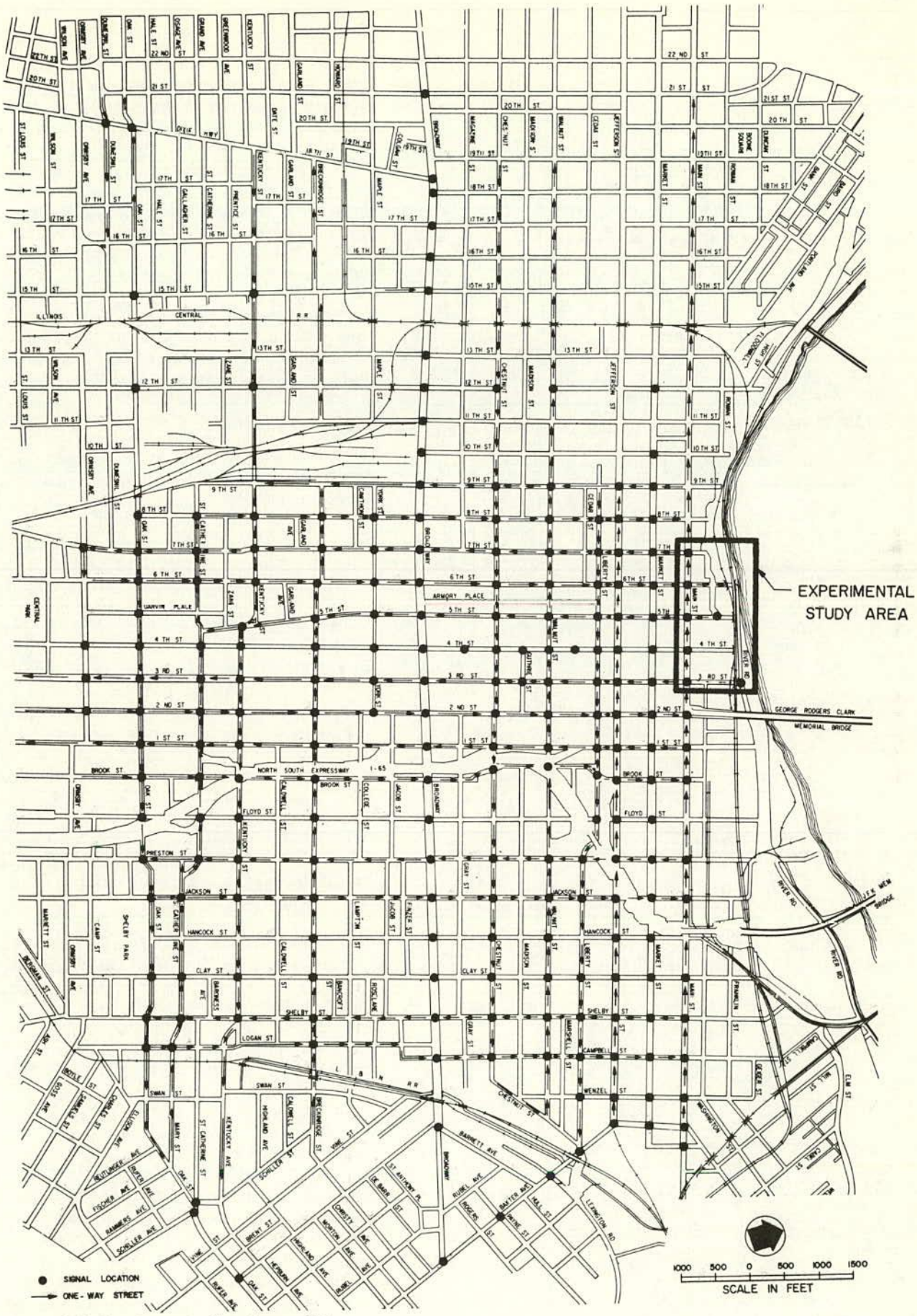


Figure G-5. Location map, Experiment E31.

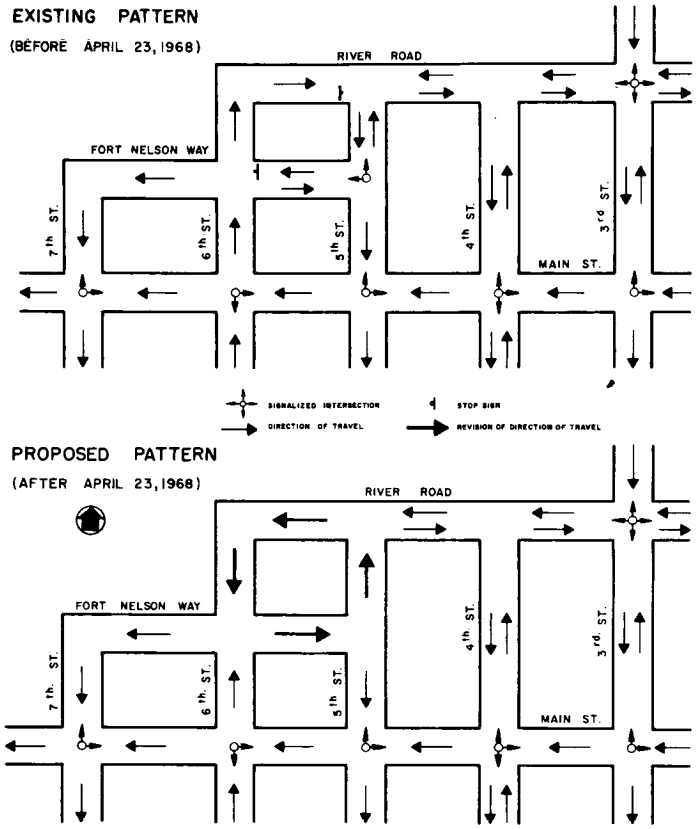


Figure G-6. Vicinity map.

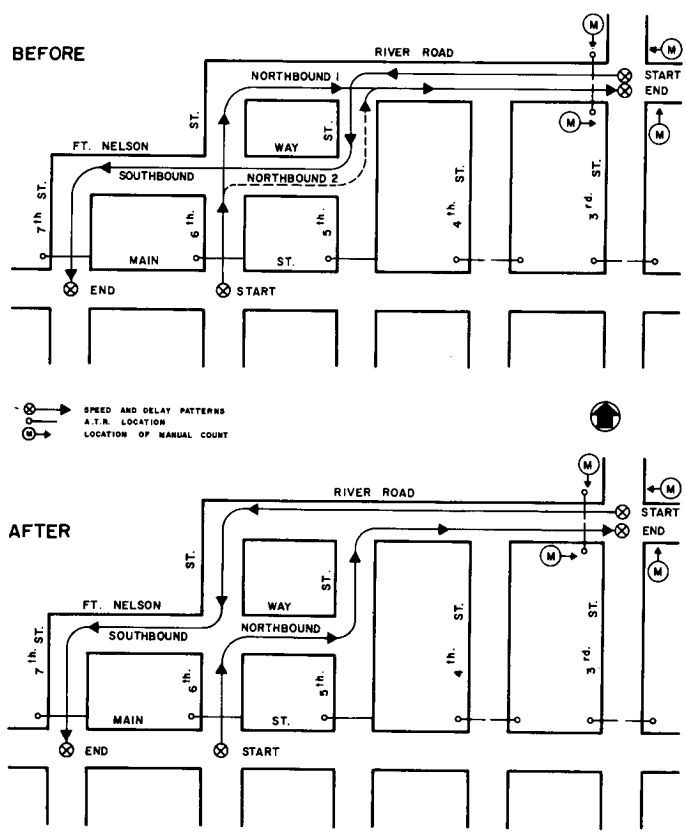


Figure G-7. Surveillance system.

Surveys.—Manual counts were taken on all approaches to the intersection of River Road and Third Street from 7:30 to 9:30 AM, and 4:30 to 6:30 PM, at 10-sec intervals, to measure volume changes that may have resulted from the proposed revision. The "before" counts were taken on April 17 and 18; "after" counts, on May 7 and 8. A complete record of "before" ATR volume counts was not obtained owing to installation failures. Speed and delay runs were performed on April 2, 4, 5, 16, 17, and 18, between 7:30 and 9 AM, and between 4 and 5:30 PM for "before" measurements. A total of 20 trips in each direction were recorded.

Seven "after" speed and delay runs in each direction were made on April 25 and 26, between the same hours as the "before" runs. All measurements were performed as scheduled, under similar good weather conditions.

First Level Analysis

The First Level Analysis for experiment E31 compared speed and delay runs obtained before and after the revision of one-way patterns. The manual volume counts at the intersection of River Road and Third Street during the morning and evening rush hours were also analyzed. A plot of the volume count by approach (Fig. G-8) shows very little difference between the different days counted and no noticeable difference between "before" and "after" counts.

Summary of Data.—A summary of the runs by time period and direction (Table G-3) shows the difference between the average speed, number of stops, and delay time before and after the revised one-way pattern was implemented. Speed, rather than travel time, was used for comparison because the distance varied slightly for northbound or southbound runs. The southbound runs, which had priority at the conflicting intersections during the "before" conditions, show a higher speed and less delays than the northbound movements during both time periods of "before" measurements. As expected, the "after" runs in the southbound direction are similar to the "before" runs. There is, however, a substantial improvement of speeds in the northbound direction.

Figures G-9 and G-10 (copied directly from Marbelite Data Compiler typical "before" and "after" charts) show the improvement resulting from the revised one-way patterns. The ordinate of the graph records speed at 1 in. equal to 20 mph; the abscissa measures time at 1 in. equal to 1 min. The graphs are similar for both "before" and "after" measurements in the southbound direction; however, the difference between "before" and "after" graphs for the northbound direction is substantial.

Comparison of Mean Speeds.—The speed and delay runs were combined into circuit round trips (combining north- and southbound directions) for analysis. Inasmuch as there was no marked difference in time periods, the AM and PM periods were also combined (Fig. G-11) in preparation for a two-sample t test. A t test requires that the samples be independent, the data be normally distributed, and the variances be equal. The random method of sample selection assures that the samples are independent; the plot of the data (Fig. G-11) indicates data consistent with a normal distribution.

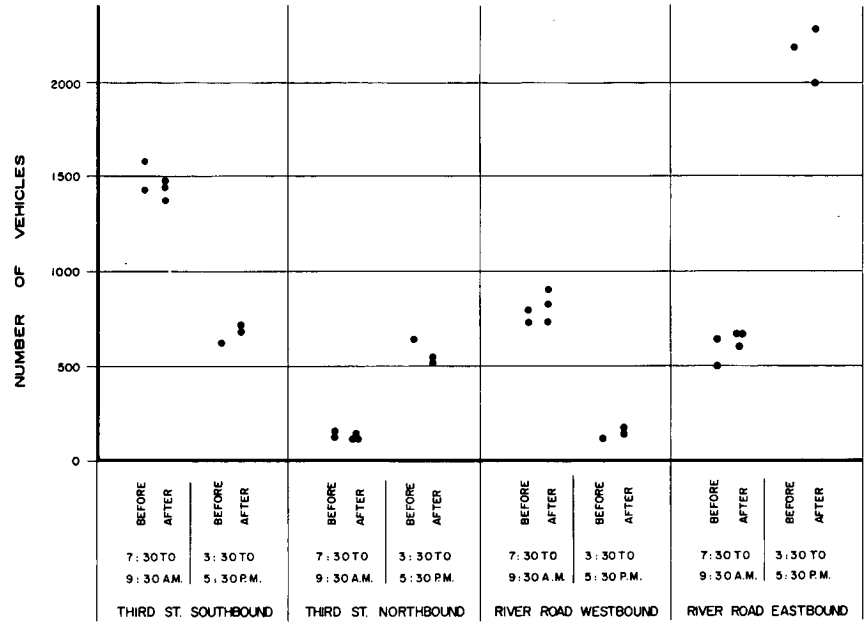


Figure G-8. Comparison of "before" and "after" volume counts, River Road and Third Street.

1. Homogeneity of variance was tested by use of an F ratio:

$$F = \frac{s^2 \text{ (larger)}}{s^2 \text{ (smaller)}} = \frac{(1.99)^2}{(1.63)^2} = 1.498$$

Because $F_{8,6}$ $(\alpha = 0.05) = 4.15$, the null hypothesis that two variances are equal *cannot be* rejected.

2. A two-sample t test was performed to test the null hypothesis that the "before" and "after" means are equal:

$$t = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{\frac{(n_1 - 1)\, s_1^2 + (n_2 - 1)\, s_2^2}{n_1 + n_2 - 2}\left[\frac{1}{n_1} + \frac{1}{n_2}\right]}} = \frac{17.16 - 13.82}{0.9305} = 3.59$$

Because $t(\alpha = 0.05) = 1.761$, the null hypothesis is rejected at the $\alpha = 0.05$ level.

3. A confidence interval $(\alpha = 0.05)$ was developed around the differences between the "before" and "after" means, as follows:

$$CI = (\bar{x}_a - \bar{x}_b) \pm t_{14}\ (\alpha = 0.05)\ \bar{x}_a - \bar{x}_b$$
$$CI = (17.16 - 13.82) \pm (1.761)\ (0.9305)$$
$$CI = 3.34 \pm 1.638$$

Or, 95 percent of the time, the difference between "before" and "after" mean speeds will be from 1.70 to 4.98 mph.

Summary and Conclusions

The results of this experiment show that a substantial saving in travel time, number of stops, and delay time was realized by the change of one-way patterns described previously.

Volume counts taken at each approach leg at a selected control intersection indicated that no noticeable volume changes occurred as a result of the improvement.

A comparison of speed and delay runs (Table G-4) shows a 25-percent increase in speed (from 13.8 to 17.2 mph), a 55-percent reduction in the number of stops (from 4.0 to 1.8 stops), and a 65-percent reduction in delay time (from 94 to 33 sec) for the average differences of all "before" and "after" measurements.

A two-sample t test rejected the null hypothesis at the $\alpha = 0.05$ level that the "before" and "after" mean speeds are equal. At a 95-percent confidence interval, the true difference between the "before" and "after" mean speed

TABLE G-3

SUMMARY OF SPEED AND DELAY RUNS BY TIME PERIOD AND DIRECTION

TIME PERIOD	DIRECTION	NO. OF RUNS	AVERAGE SPEED (MPH)	AVERAGE STOPS	AVERAGE DELAY TIME (SEC)
		(*a*) "Before" conditions			
AM	NB—1	5	10.4	1.8	87.2
	NB—2	4	11.6	2.3	46.0
	SB	9	16.6	1.7	20.3
PM	NB—1	7	9.9	2.9	83.6
	NB—2	6	11.7	2.0	51.3
	SB	11	15.3	1.7	32.0
		(*b*) "After" conditions			
AM	NB	5	19.4	0.6	12.4
	SB	4	16.6	1.0	24.5
PM	NB	3	17.9	0.7	5.7
	SB	3	16.8	1.3	23.3

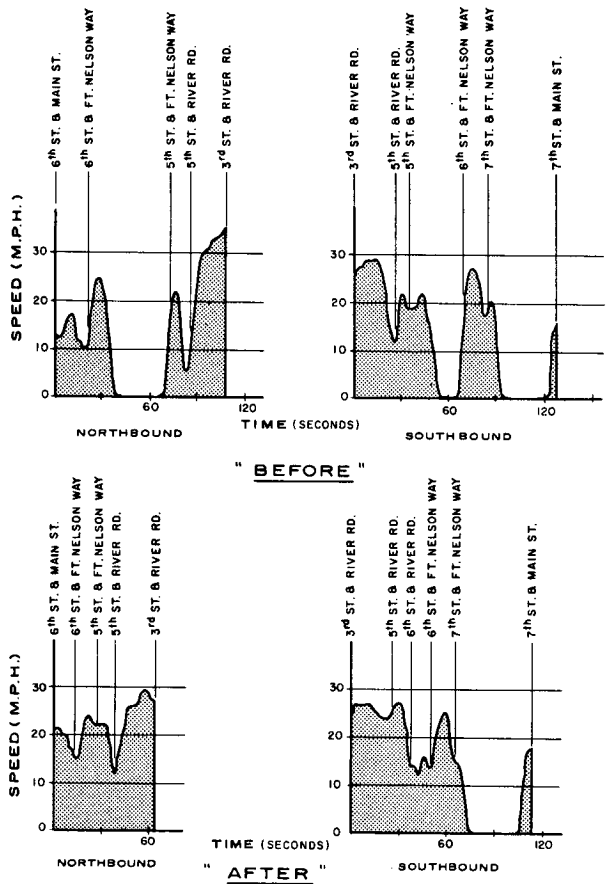


Figure G-9. Typical "before" and "after" speed and delay charts, peak hour.

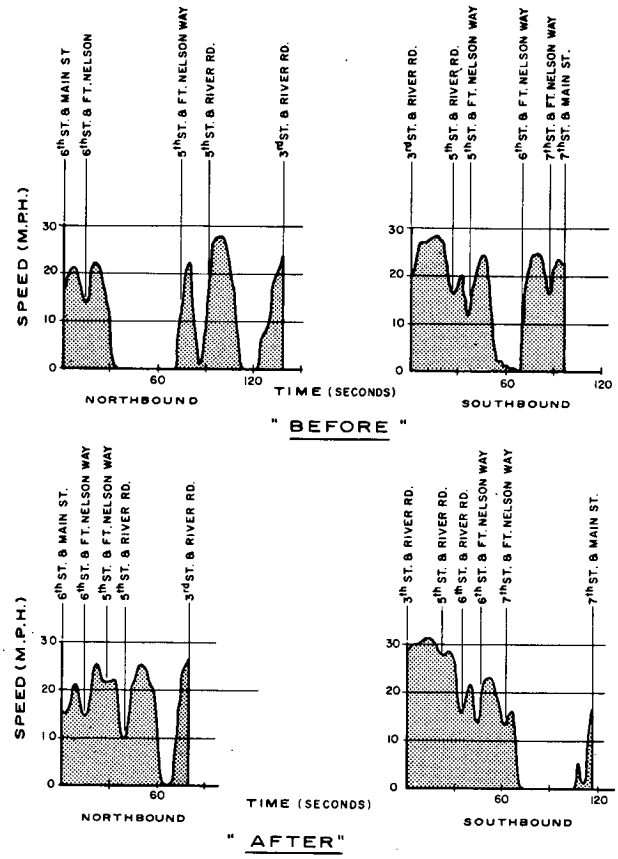


Figure G-10. Typical "before" and "after" speed and delay charts, evening peak hour.

was shown to be between 1.70 and 4.98 mph (3.34 ±1.64 mph).

The improved travel time of approximately 1 min per trip in the northbound direction was not sufficient to produce any measured volume changes beyond those attributed to the changed routing within the immediate experimental study area.

Convenience and Safety.—Although the experimental area is not a high accident area, according to police records of reported accidents, the elimination of two unsignalized conflicting movements at Sixth Street and Fort Nelson Way and at Fifth Street and River Road undoubtedly implies a reduction of accident potential.

The average trip through this area experiences two less stops and 1 min less of delay time. This is a substantial improvement in smoothness of flow and trip time, considering the shortness of the trip.

TABLE G-4

SUMMARY OF SPEED AND DELAY CIRCUIT RUNS

CONDITION	NO. OF CIRCUIT RUNS	MEAN SPEED (MPH)	STANDARD DEVIATION (MPH)	AVERAGE NO. OF STOPS	AVERAGE DELAY TIME (SEC)
"Before"	9	13.8	1.99	4.0	94.4
"After"	7	17.2	1.63	1.8	33.0
Difference	—	+3.4	−0.36	−2.2	−61.4
Percent difference	—	+25	−18	−55	−65

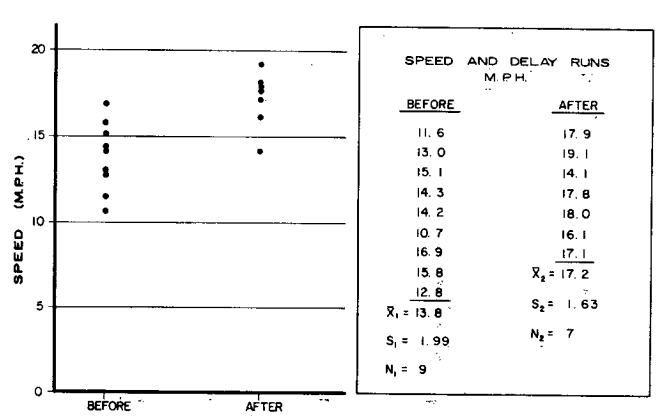


Figure G-11. Summary of speed and delay circuit runs.

Although the variances were not significantly different at the $\alpha = 0.05$ level, there is an indication that the decrease in the standard deviation in measured speeds (from 1.99 to 1.63 mph) could result in more predictable and consistent travel times.

Revision of Flow Directions in the Vicinity of Broad Street, Lincoln Park, and Pennington Street—Experiment A21

Experiment A21 considers the traffic congestion occurring in the vicinity of three closely spaced intersections on Broad Street in Newark (Fig. G-12). Broad Street, the major arterial through the downtown CBD, carries a large number of commuter buses. The street is heavily traveled, with a peak-hour volume of approximately 1,200 vehicles in each direction. Tichenor and Pennington Streets are east-west streets that terminate on the east side of Broad Street, and Lincoln Park is an east-west street terminating on he west side of Broad Street (Fig. G-13). Tichenor Street is one-way westbound, and Pennington is a two-way street. The intersections of Tichenor Street, Lincoln Park, and Pennington Street are signal-controlled. Prior to the experiment, long queues formed in the northbound approach of Broad Street at Pennington Street, often extending through the Lincoln Park intersection. This can be attributed to inadequate storage between Lincoln Park and Pennington Street for vehicles turning onto Broad Street. The storage problem is further aggravated by the presence of a bus stop on Broad Street immediately north of Pennington Street and, as a result, buses store in the curb lane almost exclusively. Also, because of the bus stop, cars northbound on Broad Street do not use the curb lane. A maximum of about 15 vehicles can store efficiently on Broad Street between Lincoln Park and Pennington Street.

Experiment A21 studies the improvement in traffic flow resulting from making Pennington Street one-way eastbound and altering the signal at Pennington and Broad Street so that it is pedestrian-actuated only. These modifications were designed to increase the storage length available to vehicles turning left from Lincoln Park to Broad Street by allowing vehicles turning into Broad Street northbound to move up to the Camp Street signals whenever the pedestrian signals at Pennington Street were not actuated.

It would have been better to have eliminated the signals at Pennington altogether. However, owing to the location of bus stops and office buildings in this area, city officials preferred to maintain the crosswalk at Pennington Street.

The problem at the Lincoln Park-Broad Street intersection, of course, would not affect Broad Street traffic if drivers would not force their way into this intersection from Lincoln Park when storage space clear of the crossing was not available. However, under prevailing conditions in the Newark study area, drivers queue across the intersection and crosswalk in several lines, preempting signal time from Broad Street to clear this queue when the signal changes. The experiment was planned to compare the situation in three ways. The effect on traffic flow was measured as follows:

1. Before any of the stated changes were made.
2. After the changes were made, for signal cycles when the Pennington pedestrian signal was actuated.
3. After the changes were made, for cycles when the Pennington pedestrian signal was not actuated.

Experimental Area

Broad Street is 86 ft wide north of Pennington and is marked for three travel lanes in each direction. The street is 64 ft wide south of Tichenor Street and is marked for two travel lanes in each direction. Tichenor Street is 33 ft wide, permitting travel one-way westbound. Pennington Street is 33 ft wide, is marked with a center line, and operated as a two-way street in the "before" condition. Lincoln Park is 46 ft wide and is one-way eastbound to Broad Street. The lane markings, signal locations, and curb restrictions are shown in Figure G-14. Typical traffic volumes on Broad Street and Lincoln Park are given in Table G-5.

Design of Experiment

The signing on Pennington Street was changed to permit operation one-way eastbound only, and standard one-way DO NOT ENTER signs were installed at Pennington Street and Orchard Street.

The signal controls at Broad Street and Pennington Street were modified so that the signal was pedestrian-actuated only. The signal timing at Broad and Pennington, when pedestrian-actuated, holds Broad Street traffic for 29 sec (Fig. G-14), allowing 13 sec for WALK and 16 sec for clearance.

"Before" and "after" measurements included a count of the vehicles clearing each cycle and the number of vehicles stopped each cycle on the Broad Street northbound approach at Tichenor, the Broad Street southbound approach at Camp Street, and the Lincoln Park eastbound approach at Broad Street. The vehicles clearing the Pennington Street westbound approach were counted during the "before" conditions, and the number of vehicles stopped in each lane on the Broad Street northbound approach at Pennington Street was recorded. All cycles actuated by pedestrians at Pennington Street for the "after" conditions were identified.

The measurements were obtained on two weekdays for the "before" conditions and three weekdays for the "after" conditions. The "before" measurements were taken on Wednesday and Thursday, June 19 and 20, 1968. Pennington Street was made one-way on September 18, 1968, and the "after" measurements were taken on Wednesday, October 23, Thursday, October 24, and Wednesday, October 30, 1968. A summary of vehicles counted during these periods is given in Table G-5.

During the morning of the first day of "before" counts the progressive-responsive (PR) system was operating on a standby 80-sec cycle, and on the second day of "after" counts a car broke down on Broad Street at the Lincoln Park intersection at 8:20 AM.

The number of vehicles westbound on Pennington Street was found to be negligible. Only 6 vehicles westbound on Pennington Street were counted during a morning peak hour, and 22 vehicles westbound on Pennington Street were

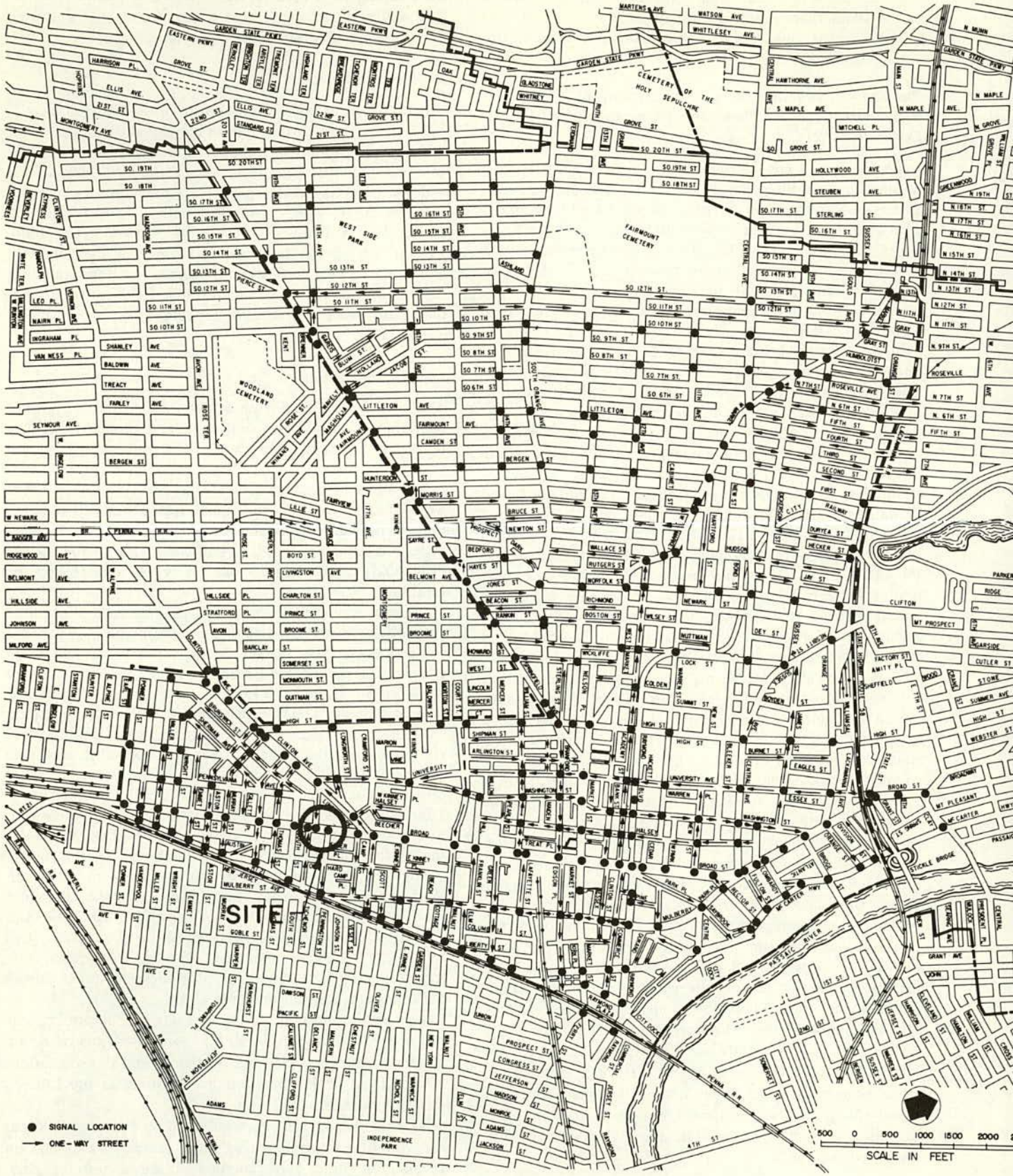


Figure G-12. Location map, Experiment A21.

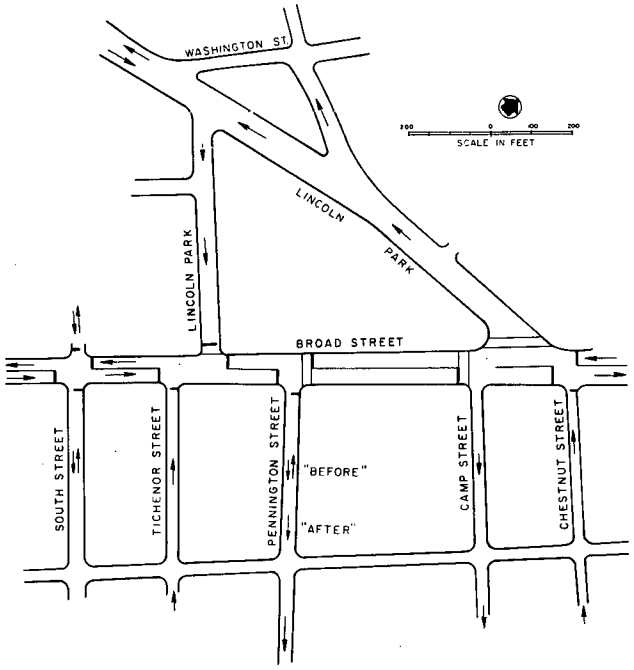


Figure G-13. Vicinity map.

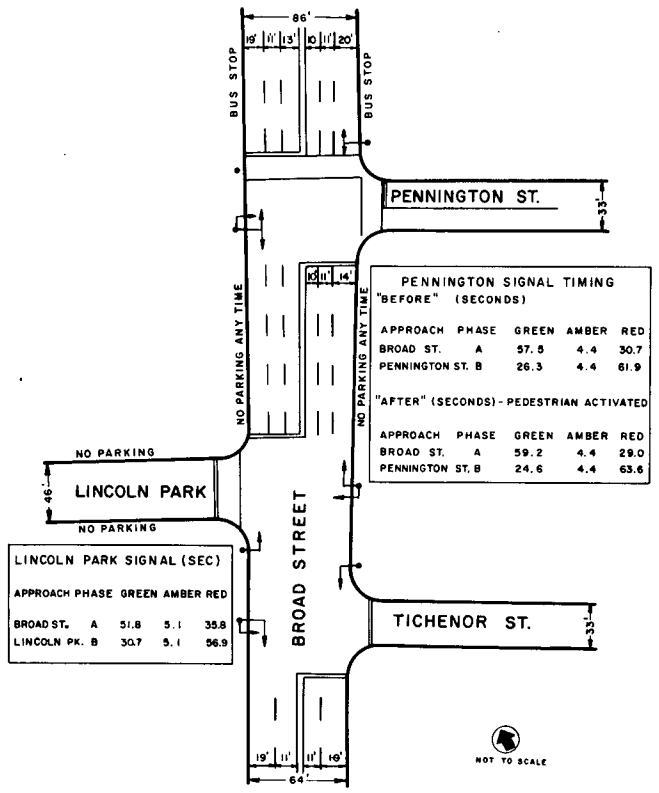


Figure G-14. Lane markings and signal locations.

counted during an evening peak hour. This small westbound volume was diverted to other westbound streets with no measurable effect after making Pennington Street one-way eastbound.

Convenience and Safety

Field observations indicate that, during "after" conditions, the number of occasions in which vehicles turning left from Lincoln Park onto Broad Street exceeded the storage was substantially reduced in comparison to the "before" conditions. Consequently, a lesser number of vehicles attempted to force their way through the intersection rather than wait through another red signal. The flow of vehicles on Broad

TABLE G-5

COMPARISON OF VEHICLES CLEARING INTERSECTION BY 15-MIN PERIODS AND VEHICLES STOPPED AT LINCOLN PARK EASTBOUND AT BROAD STREET

CONDITION	7:30 TO 7:45	7:45 TO 8:00	8:00 TO 8:15	8:15 TO 8:30	ALL
(*a*) Lincoln Park EB at Broad St.—vehicles clearing intersection					
"Before"—Thurs. 6/20	160	183	189	190	722
"After"—Wed. 10/23	174	199	170	151	694
"After"—Thurs. 10/24	189	190	162	173	714
"After"—Wed. 10/30	181	163	154	144	642
(*b*) Broad St. SB at Camp St.—vehicles clearing intersection					
"Before"—Thurs. 6/20	285	236	194	193	908
"After"—Wed. 10/23	288	267	197	195	947
"After"—Thurs. 10/24	294	263	192	221	970
"After"—Wed. 10/30	269	227	160	219	875
(*c*) Lincoln Park EB at Broad St.—vehicles stopped					
"Before"—Thurs. 6/20	70	85	105	89	349
"After"—Wed. 10/23	89	87	76	54	306
"After"—Thurs. 10/24	95	72	73	70	310
"After"—Wed. 10/30	82	60	58	51	251

Street near Lincoln Park was improved, as drivers were less often subjected to the annoyance of momentary delays waiting for the intersection to clear.

Analysis and Conclusions

Table G-5 summarizes the number of vehicles clearing, by 15-min periods during the morning peak hour, on the Lincoln Park eastbound approach and on the Broad Street southbound approach "before" and "after" the Pennington Street revision. The comparison shows no noticeable measured increase of vehicles clearing either approach during the morning peak hour. The maximum number clearing one cycle did not increase on either approach.

At the Lincoln Park approach, the number of vehicles clearing each cycle and the number of vehicles stored each cycle during the morning peak hour and the evening peak hour were recorded. The number of vehicles stored on red always cleared during the following green interval. After the changes involved in this experiment were implemented, there were still no cycles, either when the Pennington Street signal was actuated or not actuated, when the vehicles stored on the Lincoln Park approach did not clear during the following green interval. Therefore, there is no noticeable effect on vehicles clearing or vehicles stored as a result of the signal being actuated. However, when the pedestrian signal at Pennington was not actuated and vehicles turning into northbound Broad Street could move up to Camp Street instead of being stopped at Pennington, the storage was adequate and vehicles did not queue across the southbound lanes and the crosswalk.

The number of vehicles stored in each lane of Broad Street during the two days of morning peak-hour measurements for the "after" condition is given in Table G-6. A total of 457 turning vehicles were stopped due to interference by the pedestrian signal during 43 cycles when the signal was actuated—an average of 10.6 vehicles per cycle. Of course, no vehicles were stopped for this reason during the 37 cycles when the signal was not actuated. Based on the foregoing average of 10.6 vehicles per cycle, and that during an average hour 18.5 cycles were not actauted, the improvement resulted in a reduction of approximately 195 vehicles stopped at Pennington Street during the morning peak hour. However, under present conditions these vehicles are stopped at the adjacent intersection of Broad Street and Camp Street, so that the improvement of traffic flow from this experiment has little significance on a network basis. However, such local interference must be eliminated before a successful signal progression can be accomplished.

TABLE G-6

NUMBER OF VEHICLES STOPPED ON BROAD STREET NORTHBOUND AT PENNINGTON STREET
7:30 TO 8:30 AM

NO. OF VEHICLES STOPPED, BY LANE							
THURSDAY, OCTOBER 24				WEDNESDAY, OCTOBER 30			
LEFT	MIDDLE	CURB	ALL	LEFT	MIDDLE	CURB	ALL
0	2	0	2	1	4	1	6
2	3	0	5	0	3	2	5
2	3	0	5	0	4	5	9
2	4	5	11	0	5	1	6
3	4	3	10	5	7	4	16
4	4	3	11	0	6	2	8
7	8	6	21	0	5	5	10
6	6	4	16	3	5	6	14
2	3	4	9	3	4	7	14
5	6	2	13	1	5	3	9
3	3	3	9	1	1	4	6
4	3	4	11	1	4	5	10
5	5	4	14	0	2	1	3
4	3	5	12	1	3	6	10
4	5	6	15	5	5	6	16
2	5	7	14	0	5	2	7
2	6	2	10	0	4	7	5
4	5	3	12	0	2	3	5
8	8	5	21	2	6	5	13
5	4	2	11	0	1	3	4
4	5	2	11				
8	9	8	25				
4	5	4	13				
90	109	82	281	23	81	72	176

Total stopped (43 cycles out of 80 total cycles) = 457.
Average per cycle = 10.63.

Broad Street Reversible Lanes—Experiment B78

Experiment B78 was designed to evaluate the effects of reversing the direction of flow in specific travel lanes during periods of peak volumes to favor the preferential inbound and outbound movements. The work included placing rubber cones along the line dividing opposing directions of traffic flow and revising existing lane and center-line markings to appropriately designate the reversible lanes. Regulatory signs notifying the public of the required lane operations also were erected.

Experimental Area

Experiment B78 was conducted on Broad Street in Newark, between Clay Street on the north and Central Avenue on the south. This portion of Broad Street is 3,180 ft (0.6 mile) long (Fig. G-15).

Broad Street is a major north-south arterial street serving downtown Newark. Daily, approximately 37,000 vehicles use this facility in the experimental area, with peak-hour, peak-direction volumes of more than 1,750 vehicles southbound in the AM and 2,700 vehicles northbound in the PM. Office buildings housing some of Newark's largest employers are located on the east side of Broad Street between Central Avenue and Orange Street. Other land use in the experimental area includes mixed types of commercial businesses, smaller office buildings, parks, and the city library. The Erie-Lackawanna Railroad commuter station serving Newark is located adjacent to Broad Street between State Street and Lackawanna Plaza. As a result of these environmental conditions, peak traffic activity includes high-volume commuter bus operations and concentrations of pedestrians that are primarily bus and rail commuters.

Twelve of the 14 streets providing ingress to Broad Street

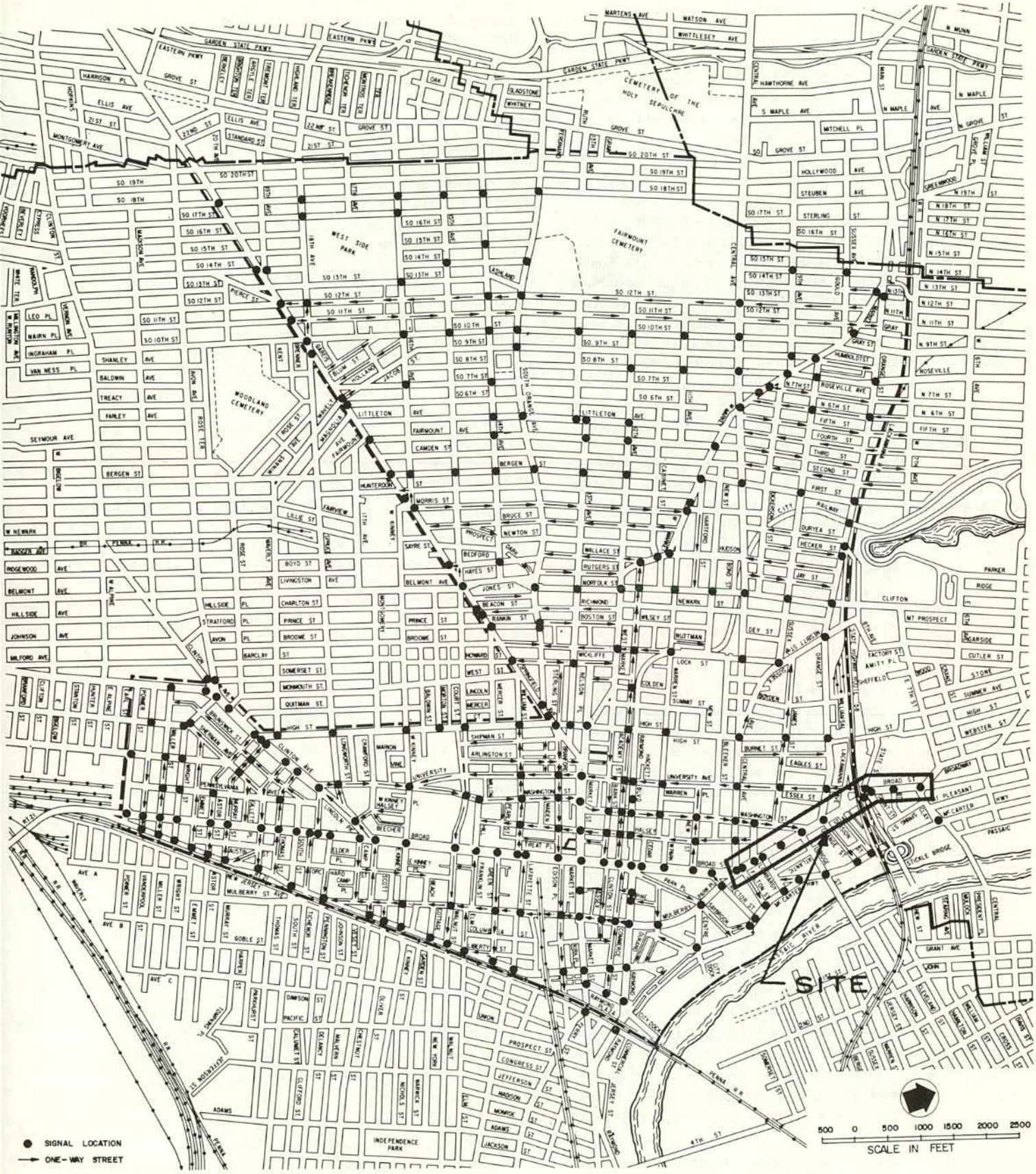


Figure G-15. Location map, Experiment B78.

in the experimental area have traffic signal control, as does one mid-block pedestrian crossing. Five additional streets operate one-way to permit only egress from Broad Street (Fig. G-16). The signals are a part of a master-controlled, interconnected system. During this experiment these signals operated on a fixed 90-sec cycle, with a single offset plan providing a basically simultaneous operation (Fig. G-17). No revisions in signal operation were made during this experiment.

Several of the intersecting streets in the experimental area are of major importance. Central Avenue and Orange Street are two of the primary arterial routes to the west; Park Place provides access to the east for both Central Avenue and Broad Street. Washington Street in a one-way northbound arterial street through the heart of the CBD. Bridge and Clay Streets provide most of the access across the Passaic River for traffic between Newark and Kearny and Harrison. Clay Street, together with Broadway, provides access to Broad Street for the majority of traffic between Newark's CBD and communities to the north.

As Figure G-16 shows, the clear curb-to-curb width on Broad Street within the experimental area varies from 62.5 to 90 ft. Pavement markings in the "before" condition were designed to provide for eight lanes of moving traffic (without parking) from Central Avenue to Washington Street and, similarly, for six lanes between Washington and Clay Streets. With parking permitted during nonpeak periods, these were reduced to six and four lanes, respectively.

During the AM peak traffic period in the "before" condition, parking was prohibited, and the lanes were evenly divided for opposing directions of traffic movement. During the PM peak period in the "before" condition, parking also was prohibited. A reversible lane provided additional capacity between Washington and Clay Streets. This lane was delineated by markings (Fig. G-16), supplemented by traffic cones. To provide public notice of this lane reversal, large regulatory signs bearing the legend 4 LANES THIS DIRECTION, 4-6 PM, MON. THRU FRI. were posted on streetlight standards on the east side of Broad Street between Washington and Clay Streets. Therefore, in the "before" condition, four lanes were designated for northbound flow between Central Avenue and Clay Street, with only two lanes remaining for southbound flow between Clay and Washington Streets.

Convenience and Safety

Because city officials did not want to proceed with operational changes on a permanent basis, temporary traffic control devices were designed. In the absence of overhead signs and signals, which are normally used to control reversible lanes on multilane facilities of this width, specially designed pavement markings, portable signs, and traffic cones were used to bring notice of the revised operations to the motorists' attention. Details of these devices and their application in this experiment are shown in Figure G-18.

The effectiveness of these measures is attested to in part by the absence of reported accidents due to the reversible lane operation, although physical evidence indicated that some of the portable signs were struck during hours of darkness. All of the traffic cones and pedestal bases required to remain in the traveled way during periods of darkness were painted with highly reflective white paint.

Immediately prior to implementation, information and details of the proposed operations were published in local newspapers through the efforts of city officials, in an effort to achieve public awareness. As the experiment progressed, the improvements realized were acknowledged in published news articles and in many verbal comments. As a measure of the operational improvements realized from this experiment, the city is presently planning to make the revised operations permanent, with control by means of overhead signs and signals.

Experimental Design

During the AM peak traffic period in the "before" condition, southbound traffic entering Broad Street at Clay Street from both Broadway and Clay Street was observed to experience considerable delay and congestion. Additional southbound traffic entering Broad Street from Grant, State, Orange, Washington, and Bridge Streets, together with the high-volume southbound left-turn movement from Broad Street into Park Place, increased congestion and delay for southbound traffic in the entire experimental area. The left-turn movement from Broad Street into Park Place was observed to be 523 vehicles (greater than 27 percent of the total approach volume) during the 7:30 to 8:30 AM period. The design provided for an additional southbound lane from Clay Street to Central Avenue for the AM time period only by reversing the direction of flow in the adjacent northbound lane.

During the PM peak traffic period in the "before" condition, severe congestion was prevalent for northbound traffic from Central Avenue to Orange Street. This resulted from the merging of northbound traffic from Broad Street and Park Place at Central Avenue, a high concentration of commuter bus activity between Lombardy and Bridge Streets, and the substantial volume of northbound traffic turning left at Orange Street. Before the experiment, one survey indicated that 1,137 vehicles traveled from Broad Street and 654 vehicles traveled from Park Place northward at the Central Avenue intersection between 4:30 and 5:30 PM. These vehicles are awarded the right-of-way simultaneously by the signal control. Four moving lanes of traffic from Broad Street are merged directly with the two lanes of moving traffic from Park Place into the four available lanes on Broad Street north of Fulton Street.

The design provided for an additional northbound lane from Central Avenue to Lackawanna Plaza for the PM time period only by reversing the direction of flow in the adjacent southbound lane. The only conditions different between the "before" and "after" operations between Lackawanna Plaza and Clay Street were the use of yellow pavement markings and the regulatory signs, as shown in Figure G-18. The "after" PM peak period operation is shown in Figure G-19.

The lane markings between Washington Street and Lackawanna Plaza were revised to provide for seven lanes

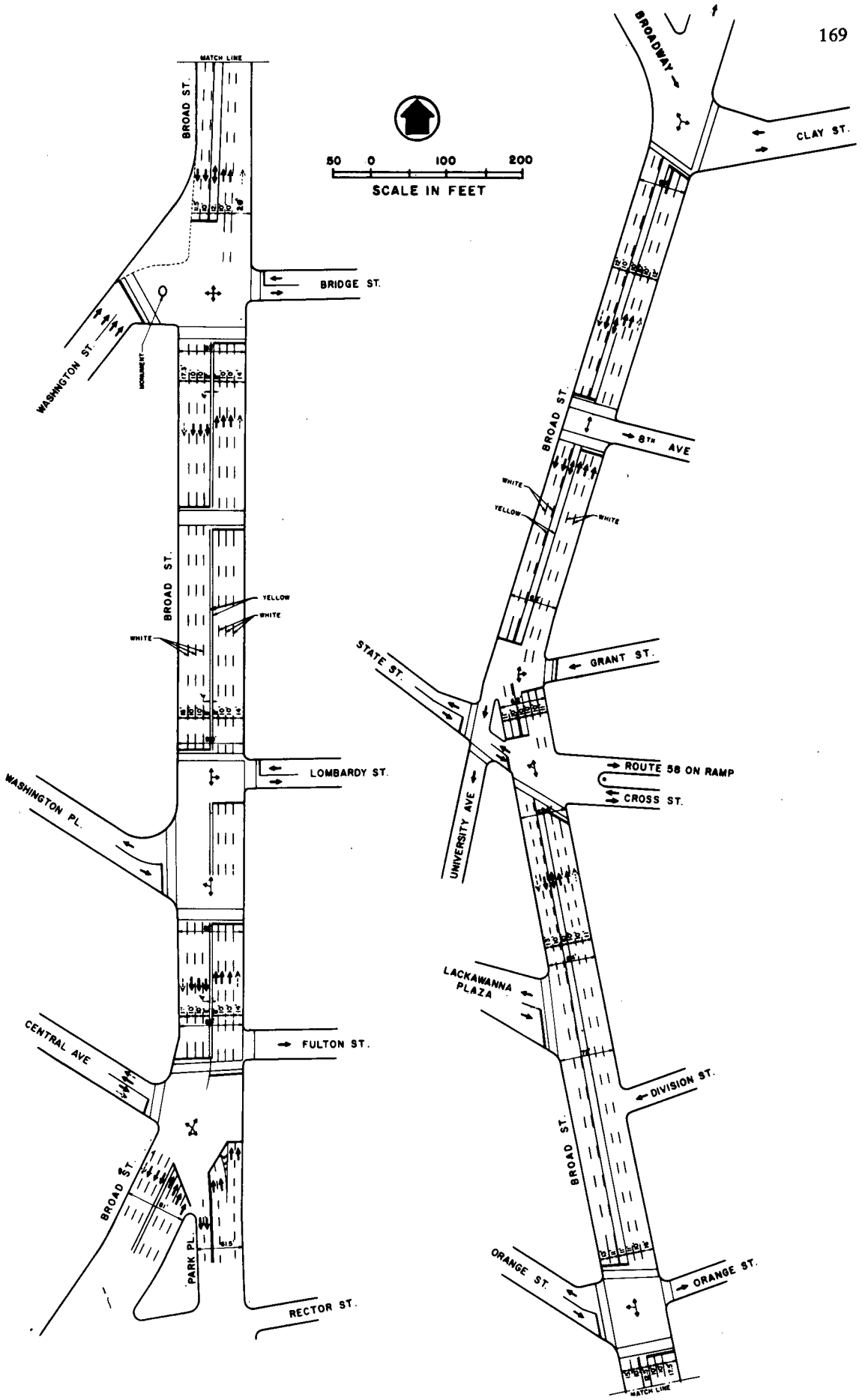


Figure G-16. Vicinity map, "before."

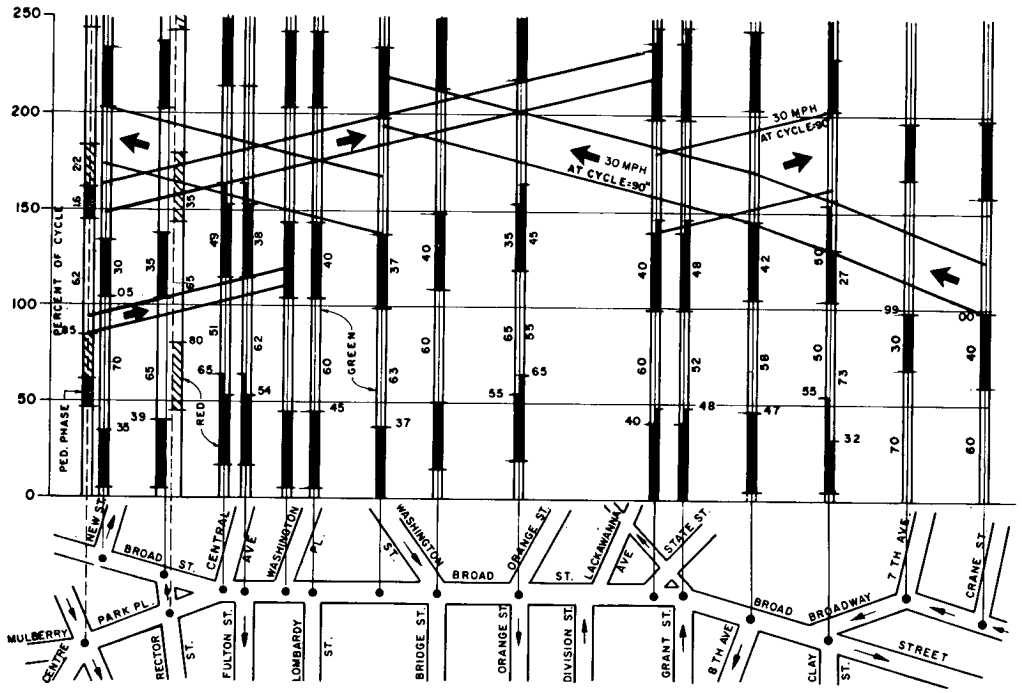


Figure G-17. Signal offset plan.

(Fig. G-18). These were reduced to six lanes at Lackawanna Plaza owing to the transition in street width from 72 to 64 ft, requiring motorists in one northbound lane to turn left. The AM and PM reversible lane operations required that the mandatory left-turn movement be made from different lanes, depending on the time of day. For purposes of safety, it was critical that both the southbound traffic directly opposing the mandatory turn lane and the vehicles in the northbound left-turn lane be physically diverted. In the absence of overhead signs and signals this was accomplished through the use of traffic cones and portable signs (Fig. G-20). In effect, these traffic cones and the portable signs became a movable channelizing island requiring manual relocation four times each day. Figure G-21 shows this location during AM peak operation. The sign in the lower left corner was used to provide advance warning of these conditions.

Rubber traffic cones were manually placed along the line dividing opposing traffic flows at 7 AM and again at 4 PM. They were then removed at 9 AM and 6 PM, respectively. During the off-peak traffic periods parking was permitted at both curbs. The total effect of the revised pavement markings during this period was to add one additional northbound traffic lane between Washington Street and Lackawanna Plaza. All other conditions remained identical to "before" conditions. "Before" and "after" lane direction operations are compared in Table G-7.

The effects of the designed and implemented reversible lane operation on this facility were determined by "before" and "after" comparisons of:

1. The number of vehicles stopped during the red signal phase.
2. The number of vehicles through during the following green signal phase.
3. The time required to travel the facility (trip time).
4. The time stopped during trips along this facility (delay time).
5. The number of stops occurring during each trip along the facility.

The actual number of vehicles stopped during the red signal phase and the number of vehicles through on the following green signal phase were manually counted for northbound and southbound movements on Broad Street at Central Avenue and Orange Street. Speed and delay runs were made between Clay Street and Central Avenue, a distance of 3,180 ft (0.6 mile), between 7 and 9 AM, and between 1 and 3 PM. Between 4 and 6 PM, runs were made only between Central Avenue and Orange Street, a distance of 1,550 ft (0.3 mile). These limits were selected to include the major changes in operations. A summary of all surveillance activity is given in Table G-8.

The PM speed and delay data were gathered in four groups—one during the "before" condition and three during the "after" condition. The latter were taken during three successive weeks to determine if greater improvement would be made as drivers became more familiar with the revised operating condition.

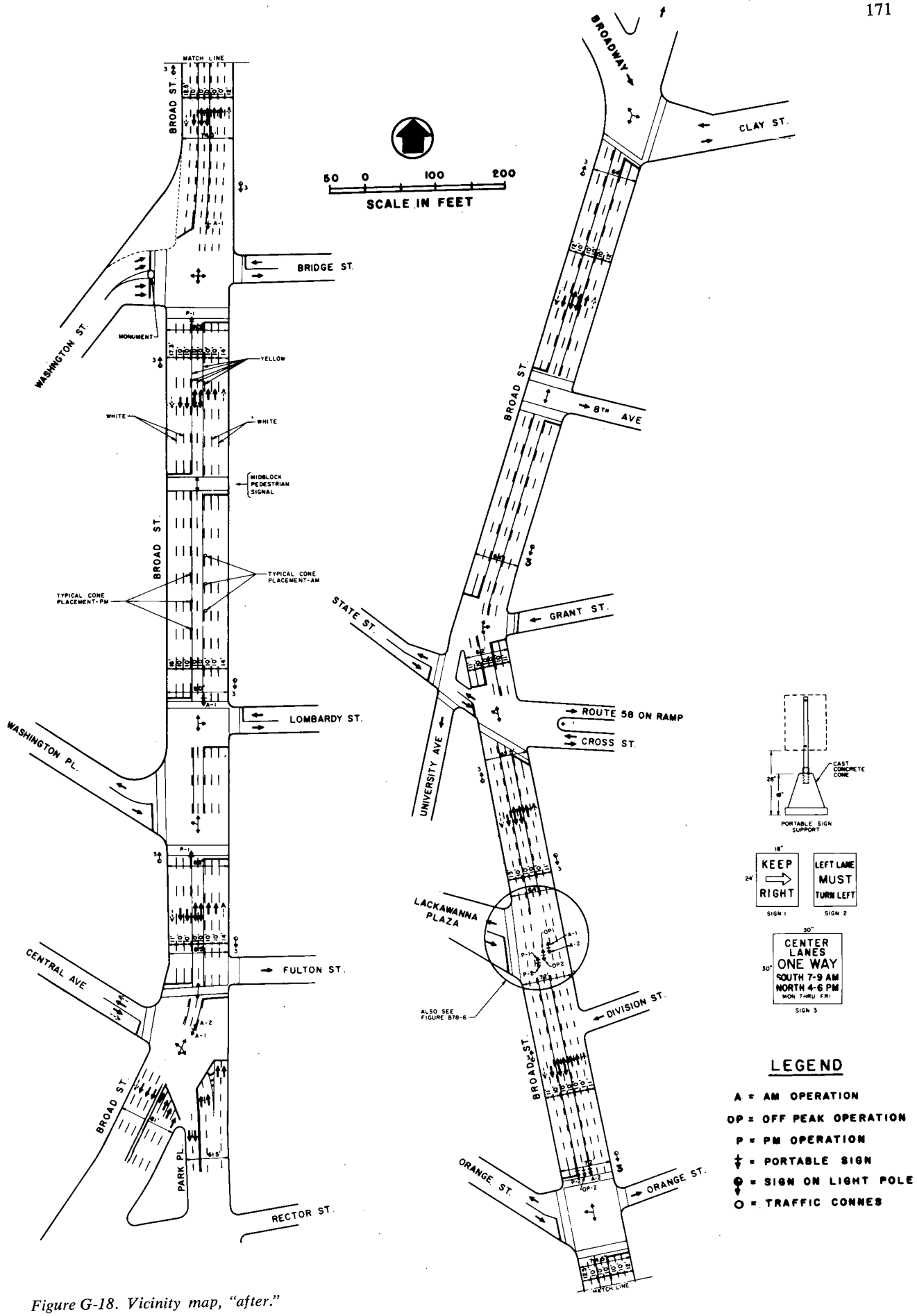


Figure G-18. Vicinity map, "after."

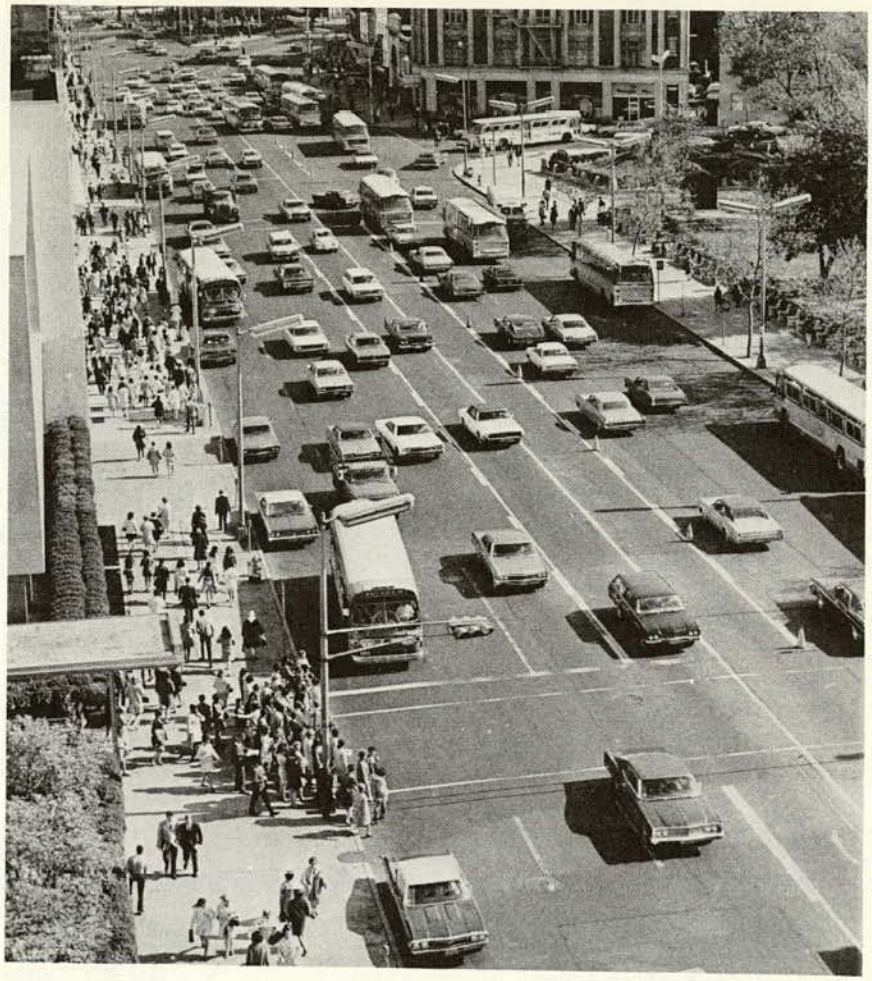


Figure G-19. View south from mid-block signal, PM *operation.*

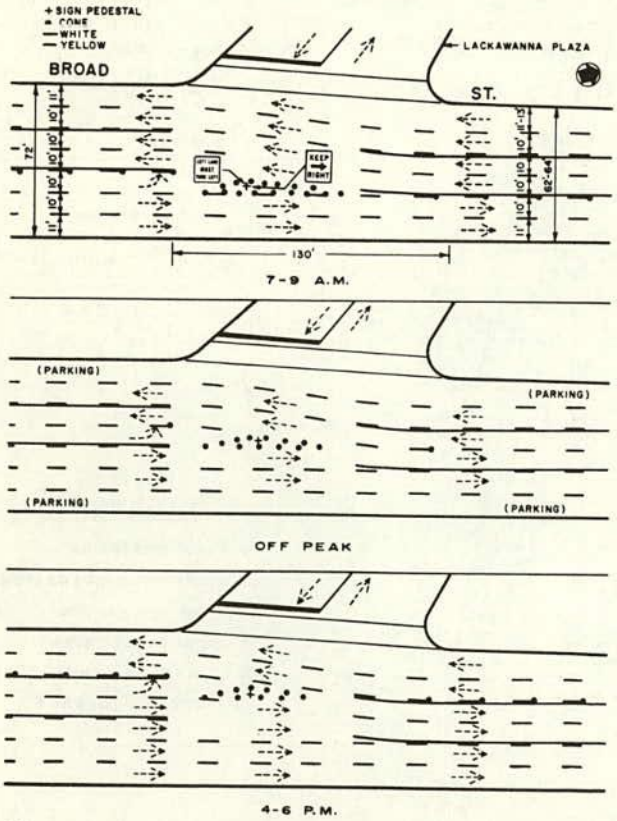


Figure G-20. Lane reduction operations at Lackawanna Plaza.

Analysis

Tables G-9 and G-10 summarize the statistical analyses of the comparisons of "before" and "after" measurements and list the mean values observed for each factor.

For the AM time period, a comparison of the "before" and "after" conditions indicated the following to be statistically significant:

1. Decreases for southbound traffic in total trip time of 109.2 sec (40.8 percent), in delay time of 73.2 sec (57.6 percent), and in the number of stops of 2.3 stops (56.1 percent).

2. A decrease in the number of southbound vehicles stopped of 8.2 (24.5 percent) and an increase in the number through of 4.6 (10.5 percent) at Orange Street per cycle.

3. Increases for northbound traffic in trip time of 46.7 sec (34.0 percent), in delay time of 23.8 sec (51.2 percent), and in the number of stops of 0.7 (43.8 percent).

4. A decrease in the number of northbound vehicles through per cycle at Park Place of 1.0 (15.2 percent).

In terms of miles of travel, the changes for southbound flow are reductions of 3.0 min per mile in trip time, 2.0 min per mile in delay time, and 3.8 stops per mile. Similarly, the changes for northbound flow are increases of 1.3 min per mile in trip time, 0.7 min per mile in delay time, and

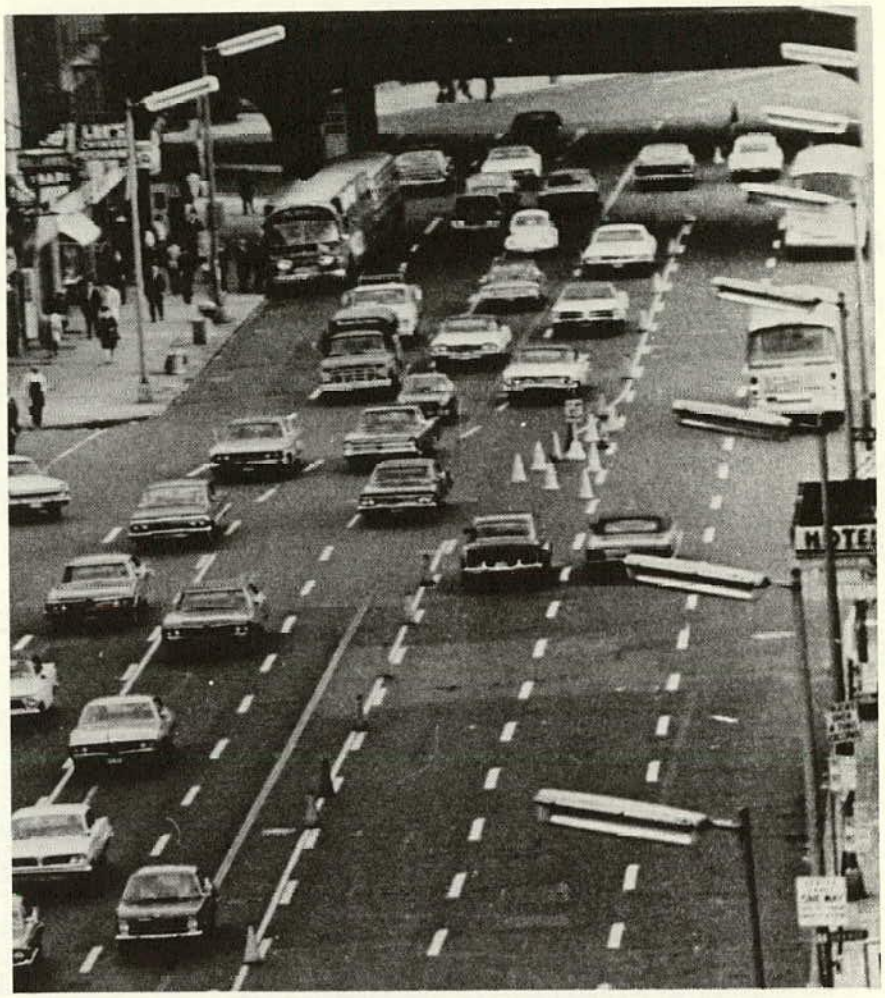


Figure G-21. View north from Orange Street, lane reduction, AM peak operation.

TABLE G-7

NUMBER OF LANES FOR MOVING TRAFFIC

TIME PERIOD	DIRECTION OF FLOW	NO. OF LANES: CENTRAL TO WASHINGTON [a] "BEFORE"	CENTRAL TO WASHINGTON [a] "AFTER"	WASHINGTON TO LACKAWANNA [b] "BEFORE"	WASHINGTON TO LACKAWANNA [b] "AFTER"	LACKAWANNA TO CLAY [c] "BEFORE"	LACKAWANNA TO CLAY [c] "AFTER"
AM peak	N	4	3 [d]	3	3	3	2 [d]
	S	4	5 [e]	3	4 [e]	3	4 [e]
	All	8	8	6	7 [e]	6	6
Off-peak	N	3	3	2	3 [e]	2	2
	S	3	3	2	2	2	2
	All	6	6	4	5 [e]	4	4
PM peak	N	4	5 [e]	4	5 [e]	4	4
	S	4	3 [d]	2	2	2	2
	All	8	8	6	7 [e]	6	6

[a] Curb-to-curb width, 85 to 90 ft.
[b] Curb-to-curb width, 74.5 to 72 ft.
[c] Curb-to-curb width, 62.5 to 64 ft.
[d] Lane reduction.
[e] Added lane.

TABLE G-8

SUMMARY OF SURVEILLANCE

CONDITION	SPEED AND DELAY RUNS: NO. OF RUNS NB	SB	DATES OF RUNS	DATES OF MANUAL VOLUME SURVEYS: AT ORANGE ST. DAY 1	DAY 2	DAY 3	AT CENTRAL AVE. DAY 1	DAY 2	DAY 3
(*a*) AM time period									
				Tues.	Thurs.	Fri.	Tues.	Thurs.	Fri.
"Before"	24	21	4/14-4/18, incl.	4/15	4/17	4/18	4/15	4/17	4/18
"After" [a]	26	21	4/28-5/5, incl.	4/29	5/15	5/2	4/29	5/1	5/9
(*b*) Midday time period									
"Before"	6	8	1967–1968						
"After"	6	7	6/10						
(*c*) PM time period									
				Mon.	Tues.	Wed.	Mon.	Tues.	Wed.
"Before"	26	20	1967, 1968, 4/14	4/14	4/15	4/16	4/14	4/15	4/16
"After" (1st week)	28	25	4/28-5/2, incl.	—	4/29 [b]	—	—	4/29 [b]	—
"After" (2nd week)	19	19	5/5–5/9, incl.	5/5	—	—	5/5	—	—
"After" (3rd week)	27	25	5/12–5/15, incl.	—	—	5/14	—	—	5/14

Note: All dates 1969, unless noted otherwise.
[a] Experiment implemented 4/28.
[b] Conditions abnormal.

1.2 stops per mile. While the percentage improvement for southbound traffic appears to be almost equal to the percent of detriment to northbound flow, the mean values are different. A comparison of these values weighted by the number of vehicles affected reveals considerable differences (Table G-11). Using the total vehicle-hours of trip time for the 7:30 to 8:30 AM time period as represented by mean times recorded for speed and delay runs and mean volumes observed at Orange Street, the increase of 12.81 veh-hr of trip time for northbound traffic obviously does not offset the decrease of 45.17 veh-hr of trip time for southbound traffic (i.e., for each vehicle-hour increase in trip time suffered by northbound traffic, a decrease of 3.5 veh-hr was realized by southbound traffic). In a similar manner, the change in delay time of +6.62 veh-hr and −32.99 veh-hr for northbound and southbound traffic, respectively, indicated that, for each vehicle-hour increase in delay time suffered by northbound traffic, a decrease of 5.0 veh-hr was realized by southbound traffic. These statistics become more impressive when the 186.7-vehicle (10.6 percent) increase in southbound volume is compared to the 13.3-vehicle (1.3 percent) decrease in northbound volume.

The fact that only one of the factors analyzed for the vehicles observed at Central Avenue changed significantly, and that being for a nondominant flow direction, was not unexpected. The signal phasing provided for the high-volume southbound left turn into Park Place was recognized as inadequate, and this situation is reflected by the analysis. Although not significant, the increases in southbound vehicles through and stopped at Central Avenue could be indicative of the increased volumes reaching this location through the improvements realized by the revised operations.

For the PM time period, a comparison of the "before" and "after" conditions indicated the following to be statistically significant:

1. An increase in the number of stops experienced by southbound traffic of 0.6 stop (66.7 percent).
2. Increases in the number of southbound vehicles stopped per cycle at Orange Street of 3.4 vehicles (28.6 percent) and at Central Avenue of 2.7 vehicles (34.6 percent).
3. An increase in the number of northbound vehicles stopped per cycle at Orange Street of 5.4 vehicles (18.5 percent).
4. A decrease in the number of northbound vehicles stopped per cycle at Central Avenue of 4.3 vehicles (35.8 percent).

The increases in the number of stops and in the number of vehicles stopped for southbound flow were expected and are probably due to the reduced number of lanes from Washington Street to Central Avenue. The increase in the number of vehicles stopped at Orange Street was unexpected. Because instances of police officers overriding the signal controls at Bridge and Orange Streets and illegal parking were observed more frequently in the "after" surveillance period than in the "before" period, a comparison of speed and delay runs with and without these factors was made. The effects and the frequency of occurrence of these factors are evident from Table G-12. Only the PM period is given, because the police officers directed traffic solely at that time.

Although no significant decreases were found in trip time for northbound traffic, the trend toward less time improved with each successive week (Table G-9). This

TABLE G-9

SUMMARY OF ANALYSIS, BROAD STREET NORTHBOUND

CONDITION	SPEED AND DELAY: TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	VEHICLES PER CYCLE: AT ORANGE ST. THROUGH	AT ORANGE ST. STOP	AT CENTRAL AVE. THROUGH	AT CENTRAL AVE. STOP	AT PARK PLACE THROUGH	AT PARK PLACE STOP
(a) AM time period									
"Before"	137.3	46.5	1.6	26.0	12.0	16.1	3.7	6.6	4.7
"After"	184.0[a]	70.3	2.3	25.7	12.2	15.9	4.5	5.6	4.3
Net change	+46.7	+23.8	+0.7	−0.3	+0.2	−0.2	+0.8	−1.0	−0.4
Percent change	+34.0	+51.2	+43.8	−1.1	+1.7	−1.2	+21.6	−15.2	−8.5
Sig. level	0.0005	0.005	0.005	NS	NS	NS	NS	0.05	NS
(b) Midday time period									
"Before"	76.8	29.7	1.0						
"After"	88.2	32.5	1.3						
Net change	+11.4	+2.8	+0.3						
Percent change	+14.8	+9.4	+30.0						
(c) PM time period									
"Before"	120.4	50.3	1.8	68.2	29.2	29.2	12.0	19.5	—
"After" Wk. 1	118.5	53.4	1.9						
Wk. 2	113.2	51.1	1.8	68.8	34.6	31.1	7.7	21.0	—
Wk. 3	110.1	53.1	2.0						
Net change (Wk. 3)	−10.3	+2.8	+0.2	+0.6	+5.4	+1.9	−4.3	+1.5	—
Percent change	−8.6	+5.6	+11.1	+0.9	+18.5	+6.5	−35.8	+7.7	—
Sig. level	NS	NS	NS	NS	0.05	NS	0.05	NS	—

[a] Variability increased ($\alpha = 0.05$).
NS = not significant ($\alpha = 0.10$).

TABLE G-10

SUMMARY OF ANALYSIS, BROAD STREET SOUTHBOUND

CONDITION	SPEED AND DELAY: TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	VEHICLES PER CYCLE: AT ORANGE ST. THROUGH	AT ORANGE ST. STOP	AT CENTRAL AVE. THROUGH	AT CENTRAL AVE. STOP
(a) AM time period							
"Before"	267.8	127.1	4.1	44.0	33.5	44.2	10.2
"After"	158.6[a]	53.9[a]	1.8[a]	48.6	25.3	44.4	12.0
Net change	−109.2	−73.2	−2.3	+4.6	−8.2	+0.2	+1.8
Percent change	−40.8	−57.6	−56.1	+10.5	−24.5	+0.5	+17.6
Sig. level	0.0005	0.0005	0.0005	0.05	0.05	NS	NS
(b) Midday time period							
"Before"	67.5	21.3	0.9				
"After"	79.6	29.3	1.1				
Net change	+12.1	+8.0	+0.2				
Percent change	+17.9	+37.6	+22.2				
(c) PM time period							
"Before"	81.9	27.5	0.9	18.7	11.9	28.1	7.8
"After" Wk. 1	106.9	46.3	1.4				
Wk. 2	95.7	36.9	1.2	18.9	15.3	30.7	10.5
Wk. 3	105.3	45.2	1.5				
Net change (Wk. 3)	+23.4	+17.7	+0.6	+0.2	+3.4	+2.6	+2.7
Percent change	+28.6	+64.4	+66.7	+1.1	+28.6	+9.3	+34.6
Sig. level	NS	NS	0.05	NS	0.05	NS	0.05

[a] Variability reduced ($\alpha = 0.05$).
NS = not significant at $\alpha \leq 0.10$.

TABLE G-11

COMPARISON OF VEHICLE-HOURS[a] FOR TRIP TIME AND DELAY TIME

CONDITION	NB (VEH-HR) TRIP TIME	NB (VEH-HR) DELAY TIME	SB (VEH-HR) TRIP TIME	SB (VEH-HR) DELAY TIME	BOTH (VEH-HR) TRIP TIME	BOTH (VEH-HR) DELAY TIME
(*a*) AM time period—Clay to Central						
"Before"	39.67	13.43	130.93	62.14	170.59	75.57
"After"	52.48[b]	20.05	85.76[c]	29.15	138.24	49.20
Net change	+12.81	+6.62	−45.17	−32.99	−32.35	−26.37
Percent change	+32.3	+49.3	−34.5	−53.1	−19.0	−34.9
(*b*) PM time period—Orange to Central						
"Before"	91.24	38.12	17.02	5.71	108.25	43.83
"After"[d]	84.20[c]	40.61	22.08[c]	9.48	106.28	50.09
Net change	−7.04	+2.49	+5.06	+3.77	−1.97	+6.26
Percent change	−7.7	+6.5	+29.7	+66.0	−1.8	+14.3

[a] Volumes measured at Orange St.
[b] Volume decrease.
[c] Volume increase.
[d] Third-week speed and delay measurements.

TABLE G-12

COMPARISON OF SPEED AND DELAY RUNS WITH AND WITHOUT INTERFERENCE (NORTHBOUND, PM TIME PERIOD)

CONDITION	NO. OF RUNS	MEAN TRIP TIME (SEC)	MEAN SPEED (MPH)	MEAN STOP TIME (SEC)	MEAN NO. OF STOPS
No interference	36	83.6	12.6	29.5	1.2
Police interference	28	149.5	7.0	80.3	2.6
Net change		+65.9	−5.6	+50.8	+1.4
Percent change		+78.8	−44.4	+172.2	+116.7
Parking interference	10	124.5	8.5	59.2	2.3
Net change		+40.9	−4.1	+29.7	+1.1
Percent change		+48.9	−32.5	+100.1	+91.7

trend is also evident from the reduction of 7.04 veh-hr for northbound trip time (Table G-11).

The decrease in the number of northbound vehicles stopped per cycle on Broad Street at Central Avenue is indicative of the greater capacity provided by the added northbound lane for traffic leaving Central Avenue. It is of interest to note that, while the increases in northbound vehicles through per cycle from Broad Street (1.9 vehicles) and Park Place (1.5 vehicles) at Central Avenue are not separately significant, the total increase of 3.4 vehicles northbound per cycle represents an increase of 7.0 percent. This increase reflects the additional capacity provided by the added lane.

During the midday time period, a number of speed and delay runs were made (Table G-8). Tables G-9 and G-10 indicate increases in trip time, delay time, and the number of stops per run from the "before" condition for both northbound and southbound traffic. Because Table G-7 indicates no revisions to operations for southbound traffic and only the addition of a lane for northbound traffic between Washington Street and Lackawanna Plaza, these increases, although small, were not expected and may have been due to either a lack of sufficient data or operational conditions that were not readily apparent.

Conclusions

Experiment B78 resulted in greatly improved quality of flow for southbound traffic in the AM peak hour, including savings of 3.0 min per mile in trip time, 2.0 min per mile in delay time, and 3.8 stops per mile, and an increase from 8.09 to 13.67 mph in average speed, with approximately 10 percent more vehicles observed proceeding through the experimental area. This increase in volume was unable to clear the Central Avenue intersection owing to the limitations imposed by signal operation. Northbound traffic in the PM peak hour did not realize comparable improvements, primarily due to flow interference resulting from police officers and illegal parking.

The effects of revising the signal operations to further improve flow is examined in Experiment B100, Broad Street Signal Progression.

Revision of Lane Markings on Central Avenue—Experiment B86

Many factors, such as width of streets, pavement surfaces, turning movements, types of vehicles, and driver habits, affect the distribution of traffic to available lanes of a roadway. Unequal lane distribution can result in a reduction of the capacity of a roadway. Balanced flow generally would be more efficient, especially at signalized intersections. Proper lane markings should encourage more efficient use of the entire width of a roadway, especially

on those with varying widths, where lane markings can organize traffic and eliminate confusion.

Experiment B86 is a study to determine the effect that revised lane markings for a roadway of varying widths have on lane distribution, travel time, and volume of flow. The change of approach width, from two to three lanes, westbound at West Market Street, is treated as a separate study (Experiment A47).

Experiment Area

Experiment B86 was conducted on that portion of Central Avenue, in the city of Newark, that is an Essex County highway between High Street and the East Orange city line at South 17th Street. Central Avenue is a major arterial used by commuters between downtown Newark and the western suburbs. The roadway varies in width from 48 to 68 ft. It is 48 ft wide from High Street to Fourth Street, 50 ft from Fourth Street to the eastern side of West Market Street, tapers in width from 68 ft west of West Market Street to 60 ft at South Seventh Street, and continues at 60 ft in width from South Seventh Street to the city line. The location of Central Avenue and the various street widths are shown in Figure G-22. Parking is prohibited on the south side of Central Avenue from 7 to 9 AM and on the north side from 4 to 6 PM on weekdays.

The existing pavement marking plan consisted of a broken white center line only, in the 48- and 50-ft sections, from High Street to West Market Street. In the 68- and 60-ft sections, from West Market Street to the city line, the street was marked with a broken white center line plus one lane line in each direction.

Purpose and Scope

The purpose of this study is to evaluate any changes in volume and travel time due to the presence of lane markings delineating four lanes in the 48- and 50-ft sections, and six lanes in the 60-ft section. The effect of lane markings on speed and volume was measured in the morning peak hour (7:30 to 8:30 AM), in the evening peak hour (4:30 to 5:30 PM), and during midday hour periods. In addition, during the evening peak hour, "before" and "after" lane distributions were compared at four locations.

The curb lanes of Central Avenue are often obstructed by illegally parked vehicles. Bus stops and right-turning vehicles also incline drivers to a preference for use of the other lanes. In addition, repaving without raising the frames and gratings at catch basins, as well as frequent accumulations of debris in the gutters, impedes the use of the curb lanes.

In the "before" situation, drivers who occupied curb lanes (not delineated) avoided these impediments by crowding or encroaching on the adjacent lanes, maintaining sufficient clearance from the curb to avoid the obstacles. In the experiment no attempt was made to change the foregoing conditions but only to mark the lanes. Thus, drivers would be forced, if they accepted the lane markings, to reduce this clearance, bringing more pavement into use and reducing the crowding of vehicles in the center lanes. Better distribution of traffic should be reflected by an improvement of average speed and volume measurements during peak periods.

Of course, in the "before" situation, when the curb-lane striping was nonexistent, some drivers occupied a position neither clearly in one lane nor in the other, as later defined. However, because volume and speed rather than lane distribution are the variables of major interest, these drivers were assigned to the lane in which the major portion of their vehicle was located.

Design of Experiment

Observation of lane distribution at various locations along Central Avenue disclosed that the 48-ft section often operated as four lanes, with the center lane in each direction carrying more vehicles than the curb lanes. In the 60-ft section the width of 19 ft between the lane line and the curb often operated as one wide lane.

It was proposed to mark four lanes from High Street to Fourth Street, five lanes (three westbound) from Fourth Street to West Market Street, and six lanes from South Seventh Street to South Eighteenth Street. In the 50- and 60-ft sections, the lane widths were fixed at 10 ft. The center line was to be painted solid white, 8 in. in width, which is the county standard. Lane lines were printed intermittently, 4 in. in width, using the ratio of 15 ft of line to 25 ft not painted. The lane widths, both existing and proposed, are shown in Figure G-23.

Surveillance System Design.—The variables to be measured for comparison of the "before" and "after" conditions are the number of vehicles in each lane in each direction during the evening peak hour and the travel times in each direction during the morning and evening peak hours.

Manual volume counts, by lanes, at four locations along Central Avenue (Fig. G-24), were designated as the source data. Speed and delay runs, six in each direction for each time period, were scheduled as the source of travel time measurements. Control Station 6, located west of University Avenue, and Station 20, located east of South Seventh Street, were used to monitor volume changes on Central Avenue during the "before" and "after" surveys.

Implementation of Experiment.—The center line and the first lane line in the 60-ft section were applied by Essex County personnel with assistance of the researchers' personnel. During weekdays a large number of parking and unparking vehicles prevented the successful application of the remaining lane lines. The researchers, with the assistance of the city of Newark personnel, applied these lane lines on a Sunday, between 2 and 8 AM.

Surveys.—The "before" manual counts were made on Thursday, June 27, 1968, at four locations on Central Avenue (Fig. G-24). Vehicles straddling a lane line were assigned to the lane that was occupied by the major part of the vehicle. The eastbound direction was surveyed from 4 to 4:30 PM, and the westbound direction was surveyed from 4:30 to 5:30 PM. Data were summarized by 5-min intervals.

Speed and delay measurements were made by the average speed method, using the Marbelite Traffic Data Compiler. A summary of 85 runs performed prior to the lane striping

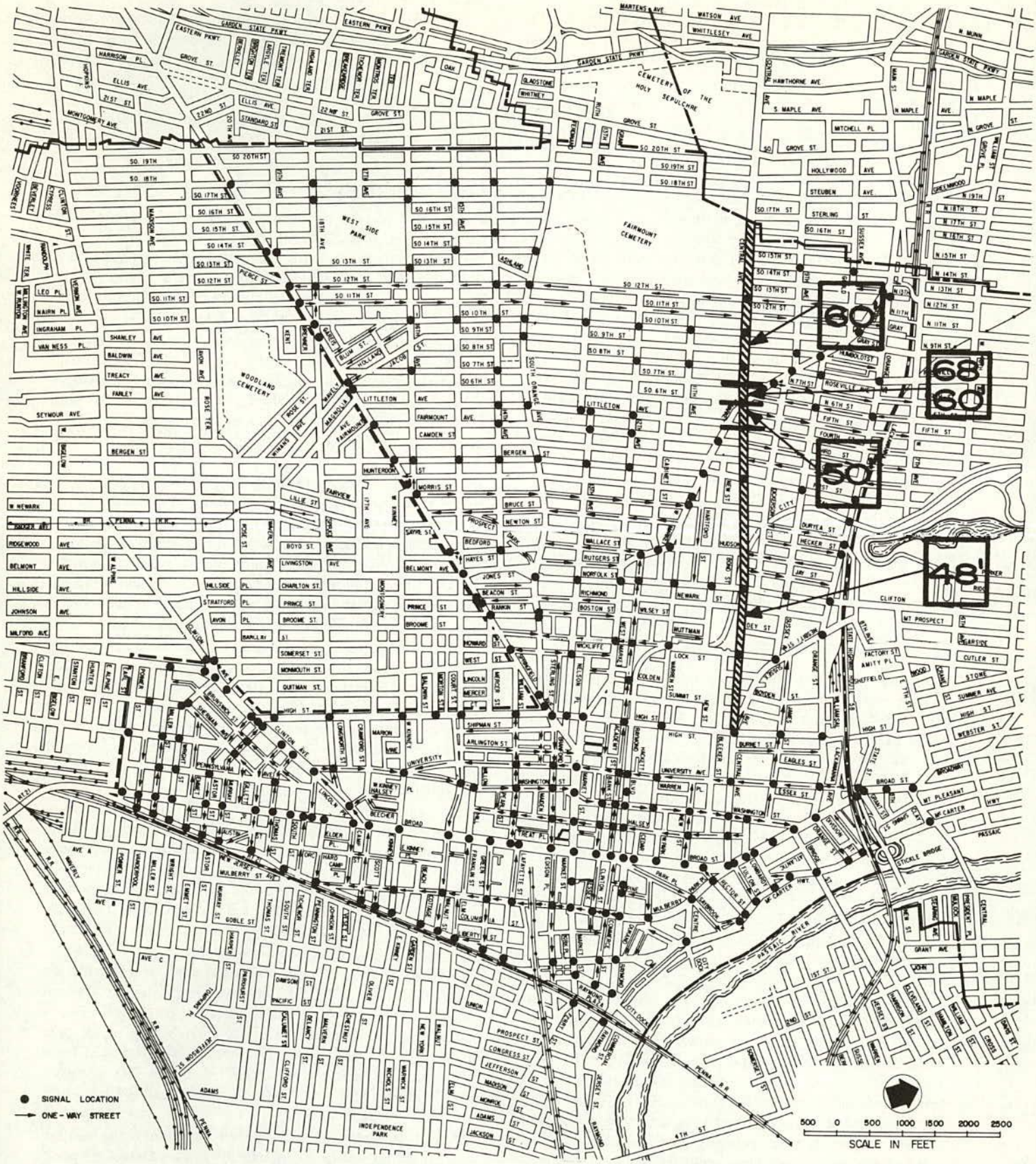


Figure G-22. Location map, Experiment B86.

indicated that a minimum of 25 runs would be required after the lane marking to detect a 1-mph difference in average vehicle speed.

First Level Analysis

The purpose of the First Level Analysis is to evaluate the significance of local changes in traffic flow caused by the revision of lane markings. This objective was accomplished through the statistical comparison of "before" and "after" measurements of certain quantities, average speed, hourly volume, and lane distribution, which were judged to be indicators of traffic flow improvement.

The analysis was performed on data representing three 2-hr time periods: 7 to 9 AM, 10 to 11 AM and 2 to 3 PM, and 4 to 6 PM. The "before" measurements were recorded between June 4 and July 19, 1968, and "after" measurements, between August 1 and August 10, 1968.

Analysis of Speed and Delay Runs.—A three-factor approximate analysis of variance (ANOVA) was performed on the "before" data (Fig. G-25 and G-26). The analysis indicated that direction, time, and the interaction effect of direction and time were significant at the $\alpha = 0.05$ level.

The westbound "before" directional mean is 15.03 mph, as compared to the eastbound directional mean of 13.11 mph. The analysis showed that speeds in the morning are different from speeds during the midday and evening periods, and the differences between direction are not constant for each period of the day. Only the average morning speeds are different at the $\alpha = 0.05$ level, as indicated by Tukey's limits. The average westbound morning speed of 17.45 mph is 28.7 percent higher than the average eastbound morning speed of 13.56 mph.

Because days of the week were not significantly different at the $\alpha = 0.05$ level, the factor of days was eliminated and the "before" data were regrouped into factors (Table G-13).

The size of the "after" sample was derived by using the formula:

$$N_2 = \left[\frac{N_1}{N_1\Delta^2 - Z^2s^2}\right]Z^2s^2 \qquad \text{(G-1)}$$

in which

N_1 = the size of the "before" sample = 87;
s = the pooled estimate of the standard deviation = 2.225; and
Z = the standard normal deviate = 1.96.

It was determined that an "after" sample (N_2) of 25 observations would be needed to detect a difference of at least $\Delta = 1$ mph between the "before" and "after" conditions. Because it was desirable to have the same number of observations in each cell, at least six runs were made during each time period by direction. The "after" runs are shown in Figure G-27.

Table G-14 gives the means and variances for each cell. A three-factor ANOVA indicated that "before" and "after," direction, time, an interaction effect of "before" and "after" and time, and an interaction effect of direction and time

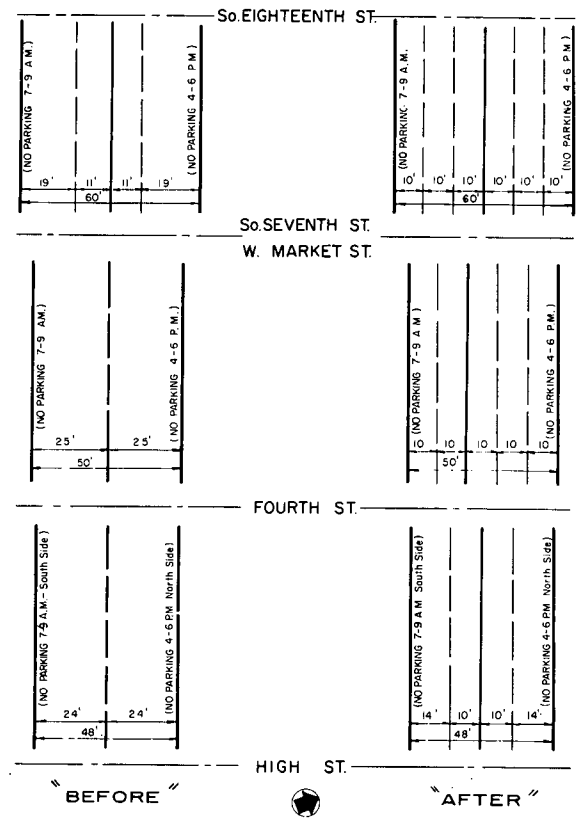


Figure G-23. Lane markings.

were significantly different at the $\alpha = 0.05$ level (Fig. G-28).

The average "before" speed was 14.12 mph as compared to an average "after" speed of 15.33 mph. This change represents an increase of 1.21 mph, or a decrease of about 37 sec in travel time over the entire length of 9,640 ft. Figure G-29 shows the interaction effect between "before" and "after" and time period, which indicates that the change in speed between the "before" and "after" conditions at the different times of the day is not constant. Tukey's limits for multiple comparison have been computed for each value and are shown in Figure G-29. These limits overlap for

TABLE G-13

CENTRAL AVENUE, MEAN SPEED "BEFORE" DATA

DIRECTION	MEAN SPEED "BEFORE" DATA (MPH) AM PEAK	MIDDAY	PM PEAK	MARGINAL MEAN (MPH)
EB	$n=15$	$n=13$	$n=11$	$n=39$
	13.67	12.42	13.45	13.18
WB	$n=20$	$n=14$	$n=14$	$n=48$
	17.64	13.52	13.98	15.04
Marginal mean	$n=35$	$n=27$	$n=25$	$n=87$
	15.66	12.97	13.71	14.11

n = number of observations.

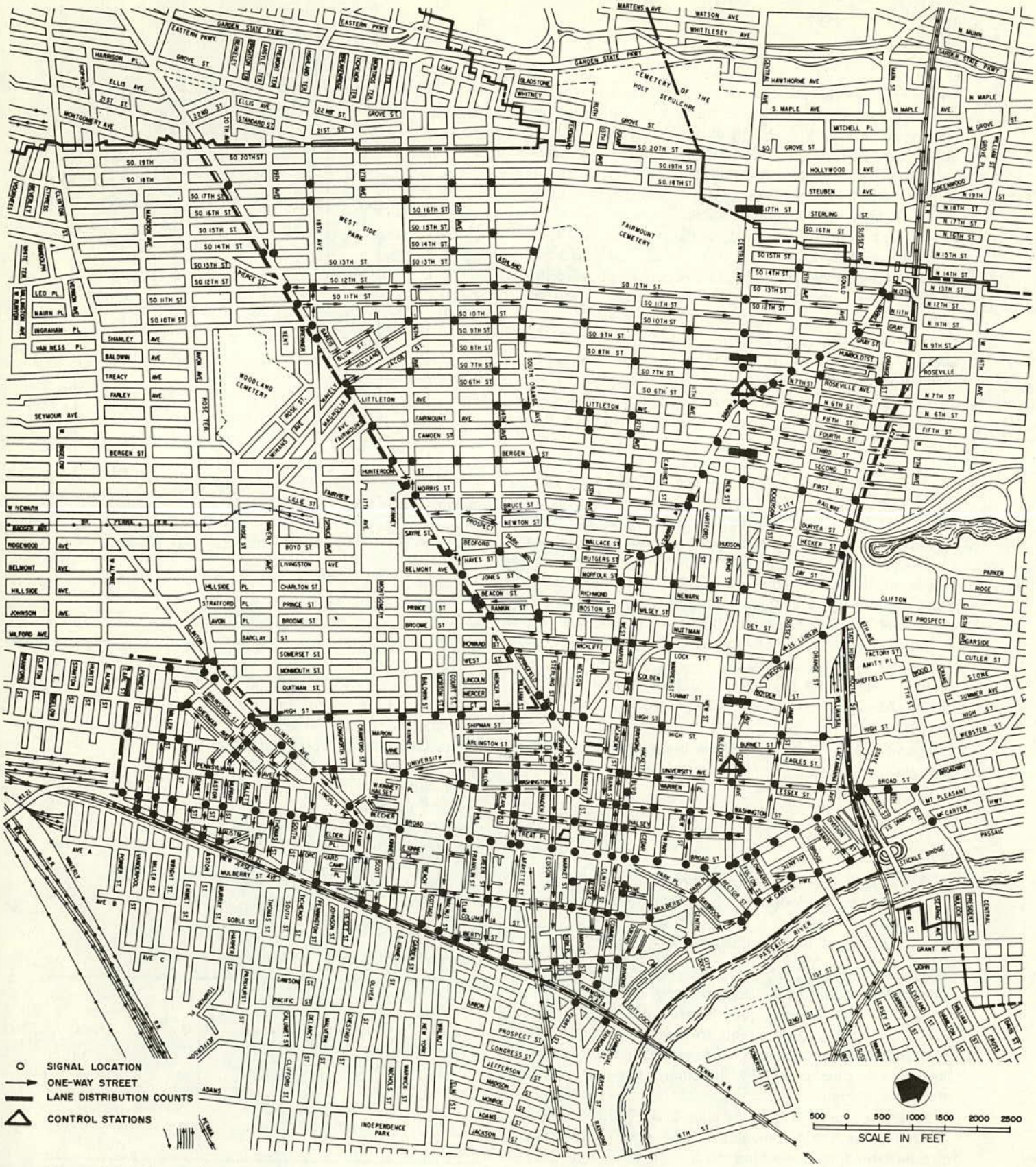


Figure G-24. Survey locations.

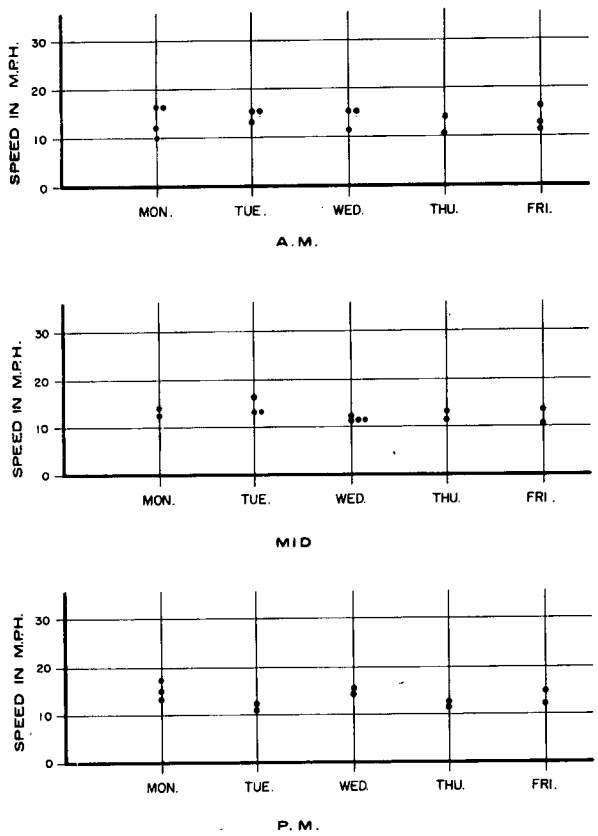


Figure G-25. Speed and delay runs, "before" data, Central Avenue eastbound.

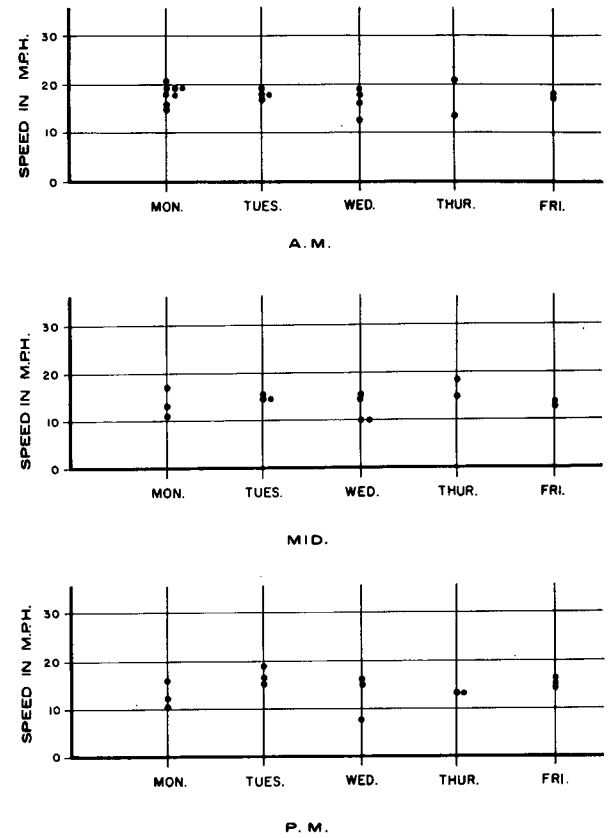


Figure G-26. Speed and delay runs, "before" data, Central Avenue westbound.

the AM and PM conditions, indicating no significant change for these time periods. However, the limts for the midday means do not overlap and indicate that on the average the "after" speed of 15.45 mph is 19.1 percent greater than the "before" speed of 12.97 mph.

Analysis of Volume Counts.—Periodic volume counts were recorded by automatic traffic recording machines on Central Avenue between West Market Street and South Seventh Street from March 1968 to July 1968. Plots of the individual values are shown in Figures G-30 and G-31.

These were investigated by a three-factor ANOVA. The factors of direction, time, and direction by time were found to be significant at the $\alpha = 0.05$ level. Because days were not significantly different, the data were regrouped, eliminating the day factor, and only those counts taken between June 4 and July 19, 1968, were used as shown in Figure G-32.

The size of the "after" sample was determined by using the formula:

$$N_2 = \left[\frac{N_1}{N_1\Delta^2 - Z^2s^2}\right]Z^2s^2 \qquad \text{(G-1)}$$

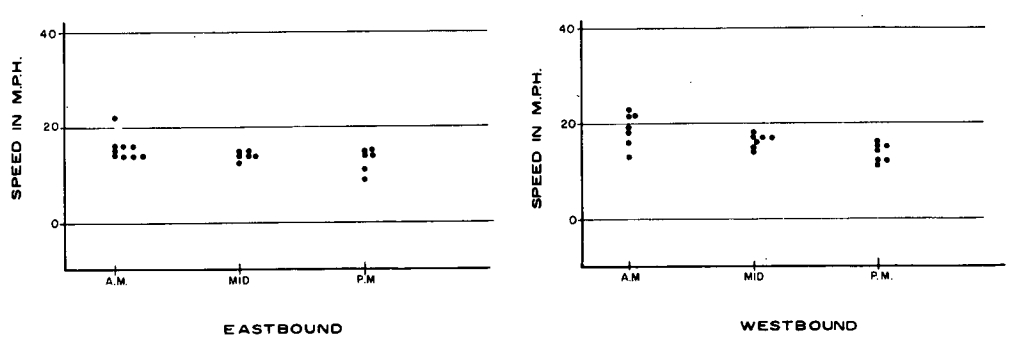


Figure G-27. Speed and delay runs, "after" data, Central Avenue.

TABLE G-14

CENTRAL AVENUE, SPEED AND DELAY DATA

CONDITION	DIRECTION	TIME PERIOD	NO. OF OBS.	CELL MEAN (MPH)	CELL VARIANCE (MPH)
"Before"	EB	AM	15	13.67	5.11
		Midday	13	12.42	2.44
		PM	11	13.45	3.70
	WB	AM	20	17.64	4.92
		Midday	14	13.52	5.62
		PM	14	13.99	7.95
"After"	EB	AM	9	15.73	5.96
		Midday	6	14.57	0.39
		PM	6	12.77	5.16
	WB	AM	7	18.81	11.37
		Midday	7	16.33	2.22
		PM	7	13.76	4.12

in which

N_1 = the size of the "before" sample = 71;
s = the pooled estimate of the standard deviation = 72.47; and
Z = the standard normal deviate = 1.96.

It was determined that an "after" sample (N_2) of 10 observations would be needed to detect a difference of at least $\Delta = 50$ vph between the "before" and "after" conditions.

The "after" volume counts were taken at the foregoing location and were grouped as shown in Figure G-33. Bartlett's test showed unequal variance between cells. The unequal variances resulted from a high value in the AM eastbound "before" data and the wide spread of points in the midday westbound "after" data. Table G-15 gives the means and variances for each cell.

The individual means were compared by use of a t test. Table G-16 gives the results of these tests, showing a slight decrease in the eastbound morning period and a small increase in the midday period "after" volumes.

Statistical Analysis of Lane Distribution.—The number of vehicles traveling in each lane was recorded during the evening peak hour of 4:30 to 5:30 PM (Tables G-17 to G-20). Lane 1 is always the curb lane.

A chi square analysis was performed on the foregoing data to determine if the conditions "before" and "after" the improvement were different. The chi square analysis tests the null hypothesis that there was no change in the lane distribution. If the null hypothesis is rejected, then the conditions "before" and "after" are not the same. Table G-21 gives the results of this analysis, which shows a difference in lane distribution in the three-lane sections only.

Summary and Conclusions

The results of Experiment B86 showed no measurable change in volume, a small increase in speed, and a slight difference in lane distribution due to the revision of lane markings on Central Avenue.

Speed Changes.—The mean speed of all speed and delay

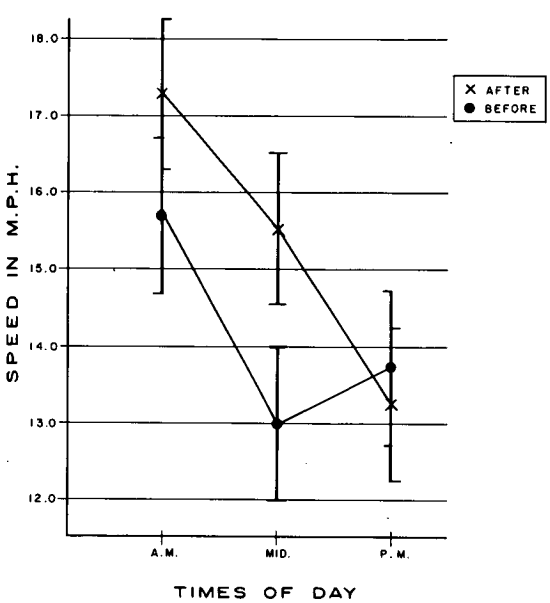


Figure G-28. Speed and delay runs, interaction between "before" and "after" with time.

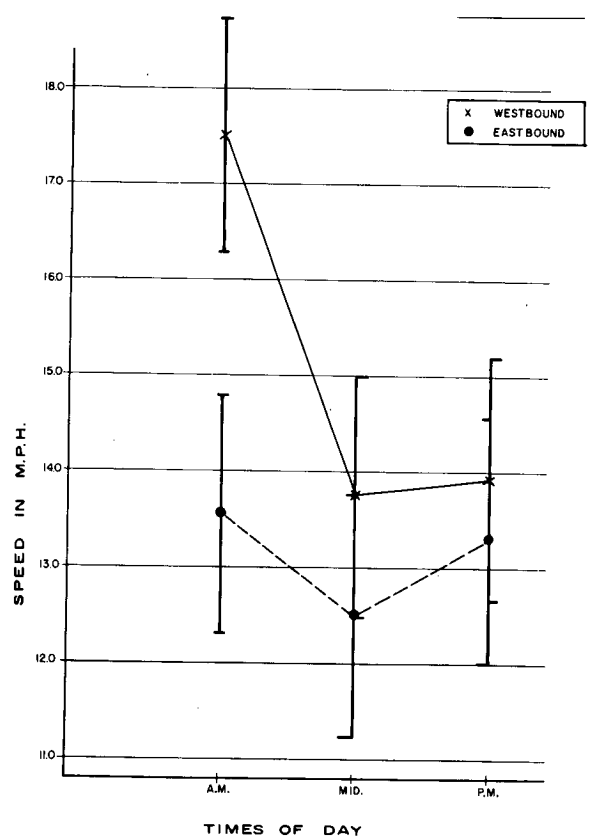


Figure G-29. Speed and delay runs, interaction between direction and time.

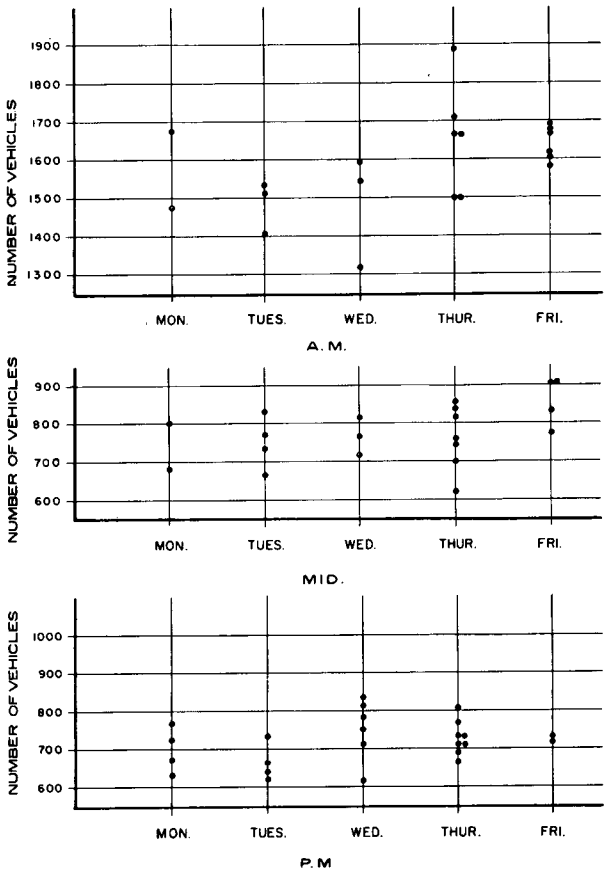


Figure G-30. Volume counts, "before" data, Central Avenue eastbound.

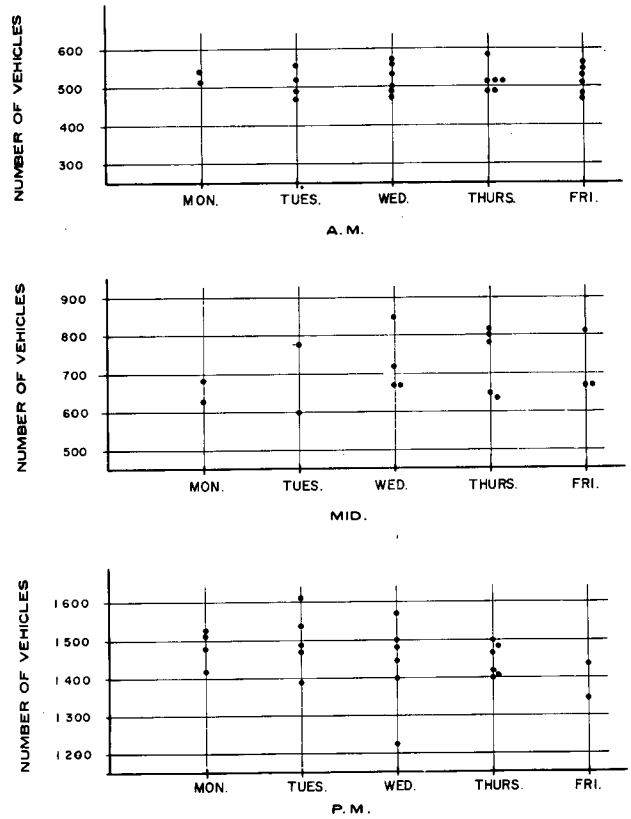


Figure G-31. Volume counts, "before" data, Central Avenue westbound.

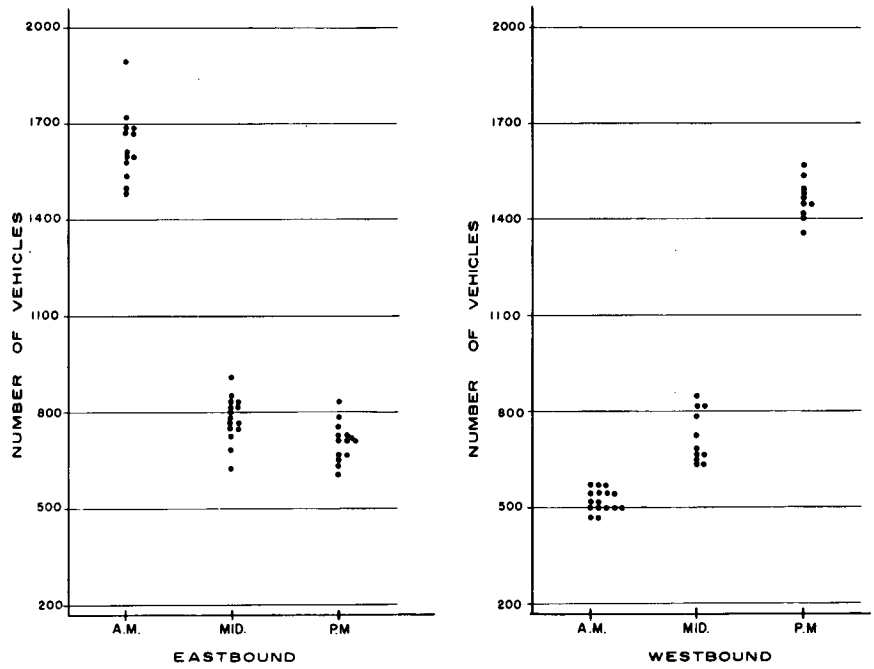


Figure G-32. Volume counts, "before" data, Central Avenue.

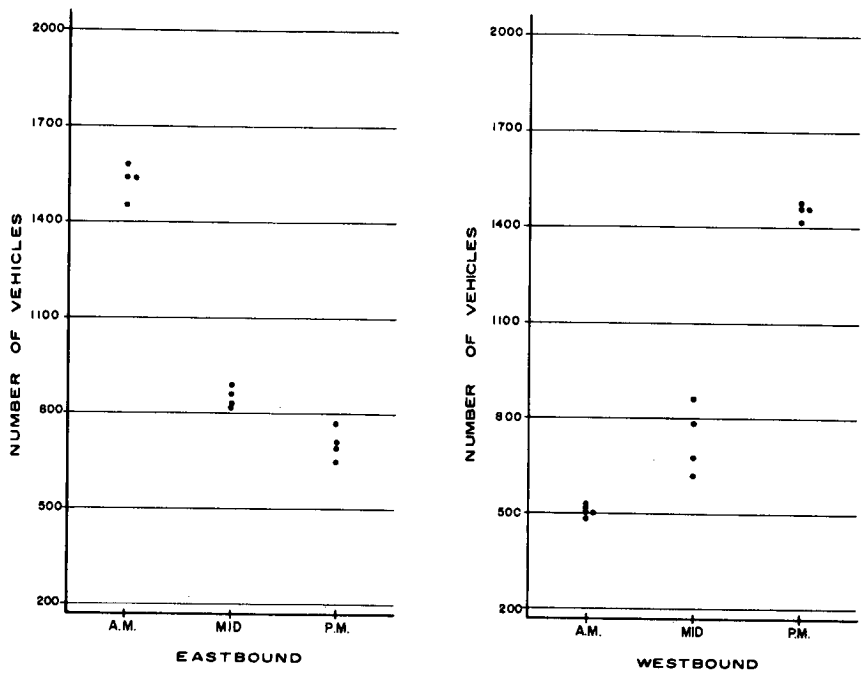


Figure G-33. Volume counts, "after" data, Central Avenue.

TABLE G-15

CENTRAL AVENUE, VOLUME DATA

CONDITION	DIRECTION	TIME PERIOD	NO. OF OBS.	CELL MEAN (VPH)	CELL VARIANCE (VPH)
"Before"	EB	AM	13	1631.00	11524.25
		Midday	15	779.47	4932.00
		PM	14	707.29	3509.92
	WB	AM	16	521.19	1240.57
		Midday	11	719.00	6000.60
		PM	10	1458.30	4141.44
"After"	EB	AM	4	1523.50	2713.67
		Midday	4	848.00	904.67
		PM	4	708.25	2280.93
	WB	AM	5	512.20	265.20
		Midday	4	740.00	11885.33
		PM	4	1448.25	678.93

TABLE G-16

CENTRAL AVENUE VOLUME, *t* TESTS

DIRECTION	TIME PERIOD	MEAN "BEFORE"	MEAN "AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$ LEVEL
EB	AM	1631.0	1523.5	−107.5	Yes
	Midday	779.5	848.0	+68.5	Yes
	PM	707.3	708.3	+1.0	No
WB	AM	521.2	512.2	−9.0	No
	Midday	719.0	740.0	+21.0	No
	PM	1458.3	1448.3	−10.0	No

TABLE G-17

CENTRAL AVENUE LANE DISTRIBUTION BETWEEN SOUTH 17th STREET AND SOUTH 18th STREET (PM PEAK PERIOD)

DIRECTION	CONDITION	VEHICLES, BY LANE 1	2	3	TOTAL
WB	"Before"	420	654	626	1700
	"After"	349	802	644	1795
	Total	769	1456	1270	3495
EB	"Before"	Parking	247	163	410
	"After"	Parking	211	162	373
	Total	0	458	325	783

TABLE G-18

CENTRAL AVENUE LANE DISTRIBUTION BETWEEN SOUTH EIGHTH AND SOUTH NINTH STREET (PM PEAK PERIOD)

DIRECTION	CONDITION	VEHICLES, BY LANE 1	2	3	TOTAL
WB	"Before"	Parking	992	806	1798
	"After"	231	815	707	1753
	Total	231	1807	1513	3551
EB	"Before"	Parking	190	206	396
	"After"	Parking	169	200	369
	Total	0	359	406	765

TABLE G-19

CENTRAL AVENUE LANE DISTRIBUTION BETWEEN SECOND STREET AND THIRD STREET (PM PEAK PERIOD)

DIRECTION	CONDITION	VEHICLES, BY LANE 1	2	TOTAL
WB	"Before"	91	158	249
	"After"	121	163	284
	Total	212	321	533

TABLE G-20

CENTRAL AVENUE LANE DISTRIBUTION BETWEEN HIGH STREET AND SUMMIT STREET (PM PEAK PERIOD)

DIRECTION	CONDITION	VEHICLES, BY LANE 1	2	TOTAL
WB	"Before"	297	505	802
	"After"	262	395	657
	Total	559	900	1459
EB	"Before"	111	185	296
	"After"	84	184	268
	Total	195	369	564

TABLE G-21

CHI SQUARE ANALYSIS, LANE DISTRIBUTION

LOCATION	DIRECTION	NO. OF LANES	NULL HYPOTHESIS
17th St. to 18th St.	WB	3	Rejected
	EB	2	Accepted
Eighth St. to Ninth St.	WB	3	Rejected
	EB	2	Accepted
Second St. to Third St.	WB	2	Accepted
High St. to Summit St.	WB	2	Accepted
	EB	2	Accepted

runs before revising the lane markings was 14.12 mph. The mean speed of all runs after revising the lane markings increased to 15.53 mph. This increase of 1.21 mph, which represents a saving of 37 sec over the entire length of 9,640 ft, was found to be significant at the 95-percent confidence level.

The mean speed during the midday period increased from 12.97 to 15.45 mph as a result of the lane marking revision. This represented a saving of about 80 sec over the foregoing length of roadway for each midday trip.

There was no significant difference in "before" or "after" mean speeds during the morning or evening peak periods. Thus, the improvement of "after" mean speeds of all runs for the three periods resulted entirely from the increase experienced during the midday periods.

Lane Distribution.—There were no significant differences in lane distribution at the 95-percent confidence level in any of the sections marked for two travel lanes in either direction. There was, however, a significant difference in lane distribution noted for the westbound afternoon peak period between South 17th and South 18th Streets (in the 60-ft-wide section) when three travel lanes were provided in one direction. In this section, the use of the left lane remained constant, the curb lane decreased, and the middle lane increased.

As Figure G-23 shows, the 19 ft of pavement adjacent to the curbs was not marked into lanes during the period of the "before" measurements. Marking the pavement for three 10-ft lanes added a lane line resulting in the selection of the middle lane by more drivers (Table G-17).

Volume Comparison.—The statistical test showed a significant change at the 95-percent confidence level of volumes in the eastbound direction during the morning and the midday periods. The differences were quite small, however, and, inasmuch as the roadway was operating at less than peak capacity, probably were due to factors other than the lane markings. The eastbound mean volume decreased by 108 vehicles, or 6.6 percent, during the morning peak hour, and increased by 69 vehicles, or 8.8 percent, during the midday hour. The evening peak-hour mean volume in the eastbound direction and the mean volumes for all three time periods in the westbound direction did not show any significant change at the 95-percent confidence level.

Convenience and Safety.—There was little apparent difference in operational characteristics noted during field observations, excepting that traffic generally used the pavement areas in a more orderly fashion after the lane marking was completed.

The major point of interest is the change of lane occupancy in the six-lane section. Evidently drivers felt more confined in the curb lane after the lane striping had been completed and preferred the center lane in greater numbers. Fewer of those drivers who had a choice (i.e., were not preparing to make a right turn at a nearby intersection) elected to use the curb lane. However, even though more drivers used the center lane, which is free from bus and turning vehicle interference, the average speed was not impaired and short-period volumes were not substantially changed.

Lane striping, because it does produce more orderly flow and better lateral spacing and defines driver responsibilities more definitely, probably will have a favorable influence on accident experience. Road length analysis from an accident study indicates an average of about 200 accidents per year for this portion of Central Avenue.

Lane Marking Revision at Central Avenue and West Market Street—Experiment A47

Experiment A-47 is concerned with the intersection of two major arterial streets, each serving commuter traffic between the suburban and downtown areas. The predominant

directional flow, inbound in the morning and outbound in the evening, combined with the high number of turns between the two streets, presents special problems in the design of the most efficient operation of this intersection.

Experiment A47 is designed to measure the improvements in flow through the intersection of Central Avenue and West Market Street in Newark, resulting from a revision of lane markings that provides an additional approach lane (from two to three lanes) on the Central Avenue westbound approach. This experiment was performed in conjunction with the lane marking revision on Central Avenue (Experiment B86).

Experimental Area

The intersection of Central Avenue and West Market Street, located in the corridor area to the west of the city's center, is circled in Figure G-34.

Central Avenue is 50 ft in width and, prior to the experiment, was marked with a center line only, east of the intersection. It operated as two lanes in each direction. West of the intersection Central Avenue is 60 ft wide, marked with a center line and two lane lines. The 60 ft of pavement are used as six lanes.

Northwest of the intersection, West Market Street is 40 ft wide and was marked with a center line only. It operated as four lanes. Southeast of the intersection, the pavement is 55 ft wide, marked for a three-lane approach and a two-lane exit.

Parking is prohibited from 7:00 to 9:00 AM on the south side of Central Avenue and on the west side of West Market Street, and from 4:00 to 6:00 PM on the north side of Central Avenue and on the east side of West Market Street. These details are shown in Figure G-35.

The traffic signal controller has a single dial with a 90-sec cycle. Central Avenue is allocated 52 percent of the cycle; West Market Street is allocated 48 percent of the cycle.

Purpose and Scope

This is an experiment to determine the improvement in traffic flow at the intersection of Central Avenue and West Market Street resulting from a revision of lane marking, which provides an additional lane on the Central Avenue westbound approach leg. The specific lane marking revisions are shown in the lane marking "before" and "after" plans, Figures G-36 and G-37.

The changes in the number of vehicles clearing each cycle and the number of vehicles stopped each cycle during the morning and evening peak traffic periods and during a midday period were used to evaluate the changes in traffic flow. Primary interest was in improvement resulting from the revised lane marking at the Central Avenue westbound approach during the evening rush hour.

Design of Experiment

The block of Central Avenue east of West Market Street is 50 ft wide, originally marked for four lanes of traffic but adequate for five lanes. A marking plan was prepared for this block having a center line 30 ft south of the north curb line. Lane lines divided the westbound direction into three lanes and the eastbound direction into two lanes. Transition sections adjacent to the experiment area were used to introduce the necessary offsets to match existing and revised markings.

The measurements used to determine the effect of the change in traffic flow were as follows:

1. Vehicles through per green interval were recorded by an observer on each of the four approaches, who classified and recorded exiting direction for each vehicle in each lane by cycle. These data are to be used in the First Level Analysis to measure the change in capacity.
2. Vehicles stored per red interval were recorded by an observer who counted the vehicles in each lane of each approach stored at the end of each red interval. The data are to be used in the First Level Analysis to measure the decrease in delay.
3. Arrival rate per 3-sec interval was recorded by an observer who counted the number of arriving vehicles on each approach by 3-sec intervals. When a 15-min counting period was completed on one approach, the next approach was counted.
4. Traffic volumes were measured using automatic traffic recorders on each approach.

To avoid possible seasonal variations in traffic, both "before" and "after" measurements were scheduled to be made between Memorial Day and Labor Day. "Before" and "after" measurements were taken on the same day of the week to avoid the possible effects of daily variations in traffic. The day was divided into three time periods for measurements:

1. Morning peak—from 7:30 to 8:30 AM.
2. Midday period—from 3:30 to 4:30 PM.
3. Evening peak—from 4:30 to 5:30 PM.

Implementation of Improvement

Central Avenue in this area is under the jurisdiction of the Essex County Highway Department, whose policy has been to paint a center line only, east of West Market Street, and a center line and two lane lines west of West Market Street.

To assist in the experiment, the Essex County Highway Department painted the center line and two inner lane lines east of Fourth Street and west of West Market Street. The city of Newark painted the area from Fourth Street to West Market Street and the two additional outer lane lines west of West Market Street. In addition, the city of Newark painted over the original center line with black traffic paint where the old and new markings did not coincide. The painting was completed on July 28, 1968.

Surveys

Counts were scheduled to be made for three days each week for a two-week period in the "before" study to ensure that a sample of adequate size was collected. As a result of analysis of these data, the "after" surveys were reduced to

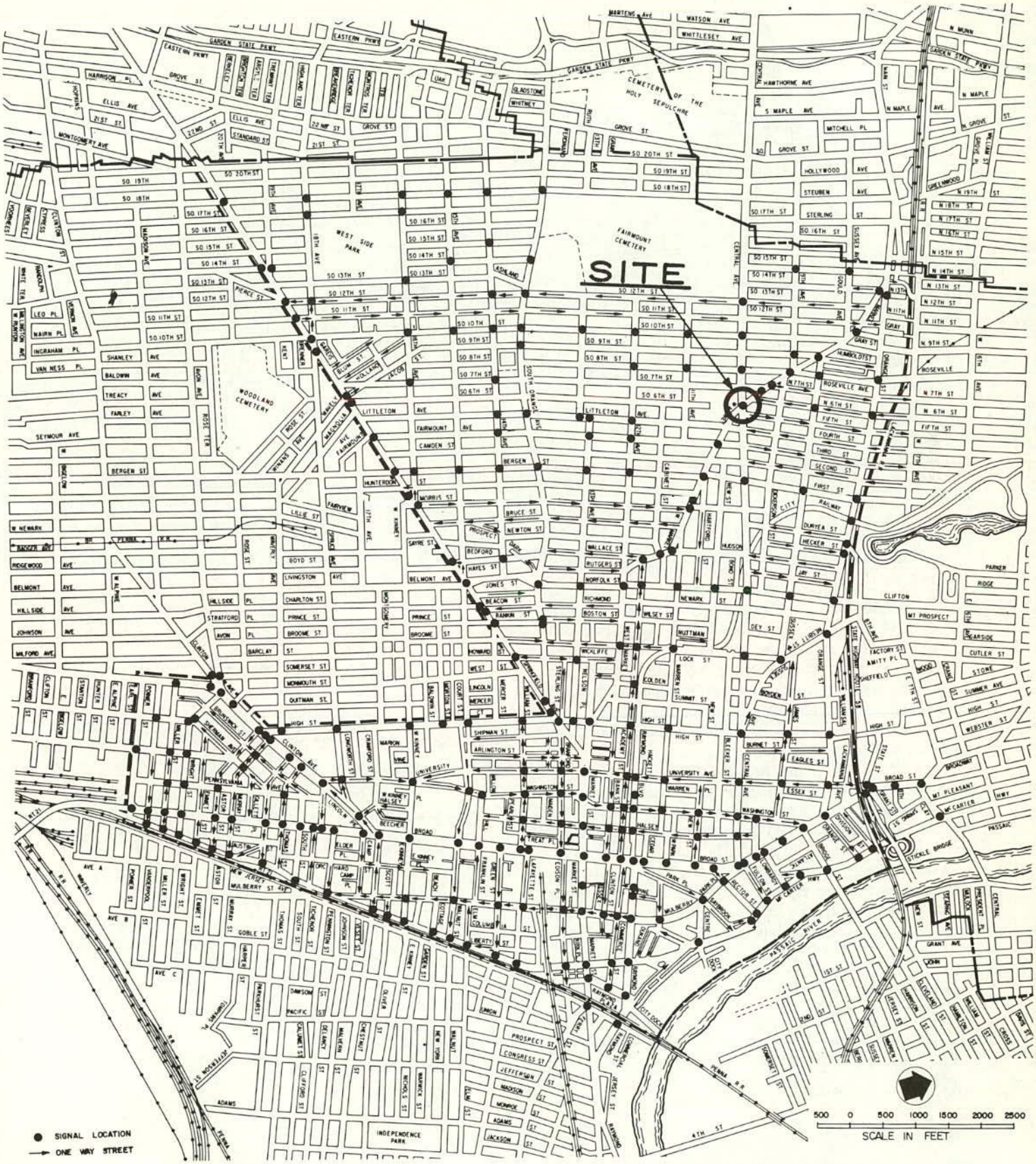


Figure G-34. Location map, Experiment A47.

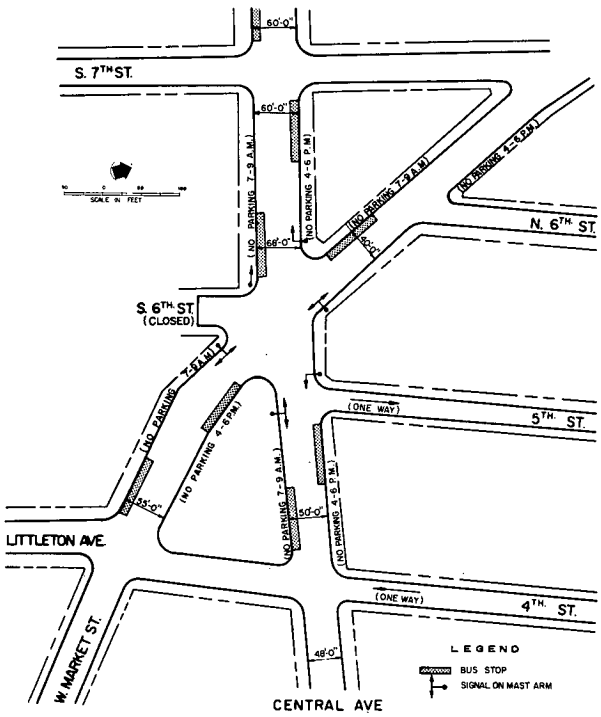


Figure G-35. Traffic controls, Central Avenue and West Market Street.

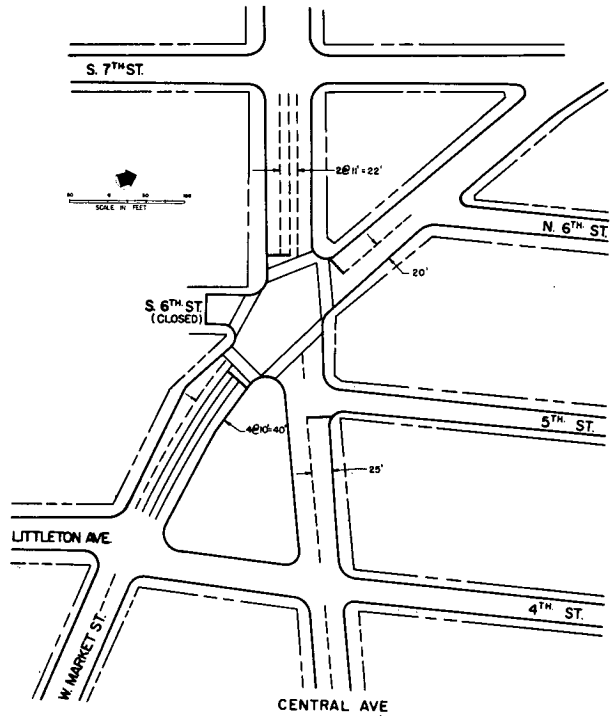


Figure G-36. Lane markings "before," Central Avenue and West Market Street.

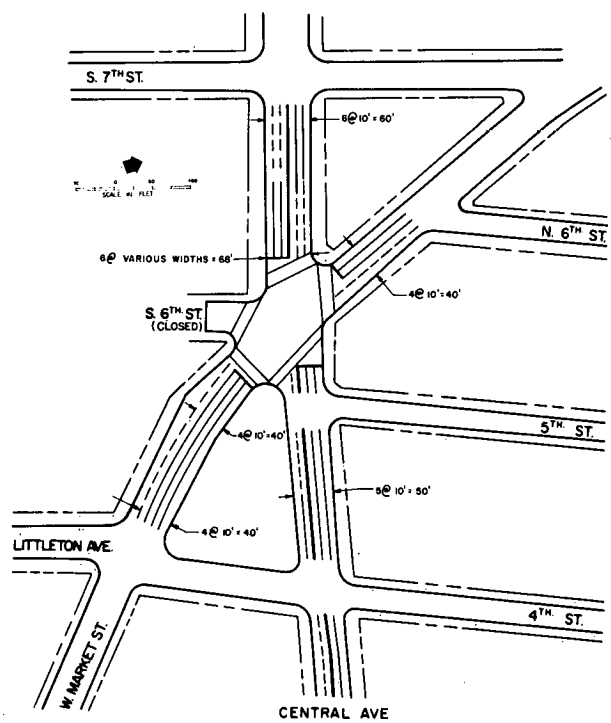


Figure G-37. Lane markings "after," Central Avenue and West Market Street.

three days of a one-week period. Count information was obtained for each time period given in Table G-22.

During the evening peak hour of the "before" study, the queue length on Central Avenue westbound often extended past the upstream traffic signal (First Street), a distance of 1,100 ft. Because it was not possible to determine if the backup beyond 1,100 ft was caused by the West Market Street intersection or the upstream signal, the queue length was recorded only as far as 1,100 ft. The arrival rate during the red interval was not recorded during the times of extreme backup.

Twenty-five feet of pavement were available for each direction of traffic on Central Avenue east of the intersection prior to the lane marking revision. This allowed two moving lanes eastbound on this leg, even when vehicles were parked at the south curb. The lane marking revision provided only 20 ft for eastbound traffic; these were not sufficient for two moving lanes when vehicles were parked at the eastbound curb. During the morning rush hour, when parking was prohibited at the south curb, illegal

TABLE G-22

SUMMARY OF MEASUREMENT PERIODS

	MORNING PEAK		MIDDAY		EVENING PEAK	
DAY	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
Mon.	—	—	5/27	8/5	5/27	7/29
Tues.	5/28	7/30	6/4	7/30	6/4	7/30
	6/4	—	—	—	—	—
Wed.	6/5	7/31	—	—	—	—

parkers reduced eastbound movement to one lane. The Newark Police Department cooperated by removing these illegally parked vehicles.

First Level Analysis

The purpose of the First Level Analysis is to evaluate the significance of changes in local traffic flow caused by the improvement. This objective was accomplished through the statistical comparison of "before" and "after" measurements of certain quantities that were judged to be indicators of traffic flow improvement. The number of vehicles through the intersection per cycle and the number of saturated cycles were studied at the intersection for this purpose. Only the outbound approaches (i.e., Central Avenue westbound and West Market Street northbound) were statistically analyzed for this experiment.

Parking restrictions for the westbound direction begin at 4 PM. Consideration of this change in road conditions indicated the need for adjustment of the time periods used during the survey. Therefore, the records of the half-hour period from 4 to 4:30 PM were deleted from the midday surveys and combined with the records of the evening peak hour.

Analysis, therefore, was performed on data representing three time periods: 7:30 to 8:30 AM, 3:30 to 4:00 PM, and 4:00 to 5:30 PM.

Field data were summarized by cycle in preparation for the First Level Analysis. The number of saturated cycles and the number of vehicles through during the green interval were obtained directly from field survey forms.

Analysis.—The number of vehicles through by cycle, shown in Figures G-38 and G-39, were arranged in three factor tables: days, "before" and "after," and time of day for each approach. Bartlett's test for homogeneity of variance, which was performed on the foregoing data, rejected the null hypothesis of equal variance. The data were regrouped, eliminating the factor of days. The ratio of the variances for each time period in the "before" and "after" conditions, by approach, was then compared with the critical values of the F distribution. If the "before" and "after" variances were equal, a t test was used to determine if a difference existed between the "before" and "after" conditions, as given in Table G-23.

The results given in Table G-23 generally indicate that there is no significant difference in volume between the measurements for the "before" and "after" conditions except for:

1. Central Avenue westbound during the morning and midday periods.
2. West Market Street northbound during the midday period.

The frequency of saturated cycles was recorded in two-by-two contingency tables (Table G-24).

Summary and Conclusions

The results of Experiment A47 show that substantial improvement in traffic flow at an intersection can be realized by increasing the width (from two to three lanes) at a critical intersection approach. The measurements used to evaluate improvements resulting from the revised lane marking at this intersection are the number of vehicles clearing each cycle and the number of saturated cycles.

Vehicles Through Per Cycle.—The comparison of "before" and "after" vehicles through each cycle showed no meaningful differences during the time periods studied for any approach leg. During the evening peak hour, Central Avenue westbound averaged 25.7 vehicles through each cycle "before" and 26.0 vehicles through each cycle "after." Considering a shorter interval, there was no substantial change during the peak 15-min period (5:15 to 5:30 PM) on this approach either, 29.7 vehicles clearing each cycle "before" and 30.5 clearing "after." The lack of any significant volume change, as a result of the additional lane, is

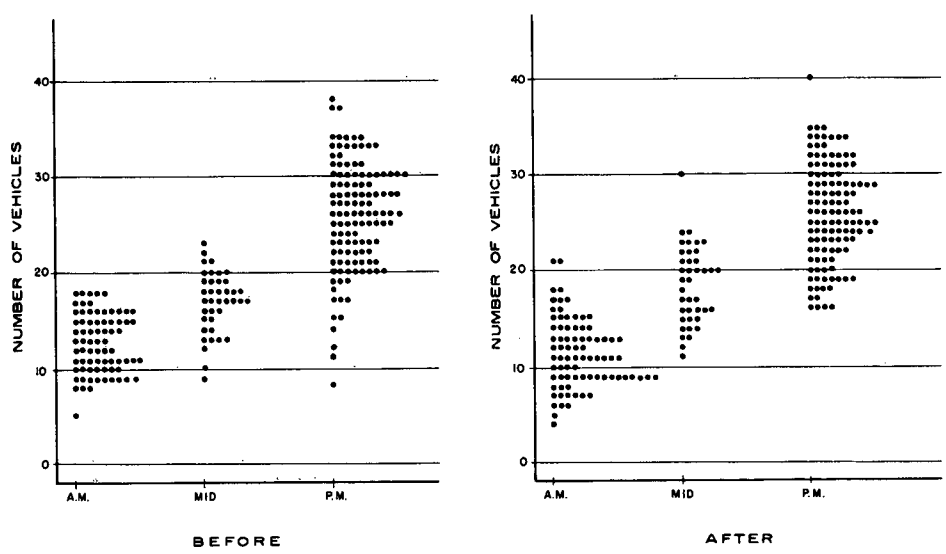


Figure G-38. Vehicles through per cycle, Central Avenue westbound.

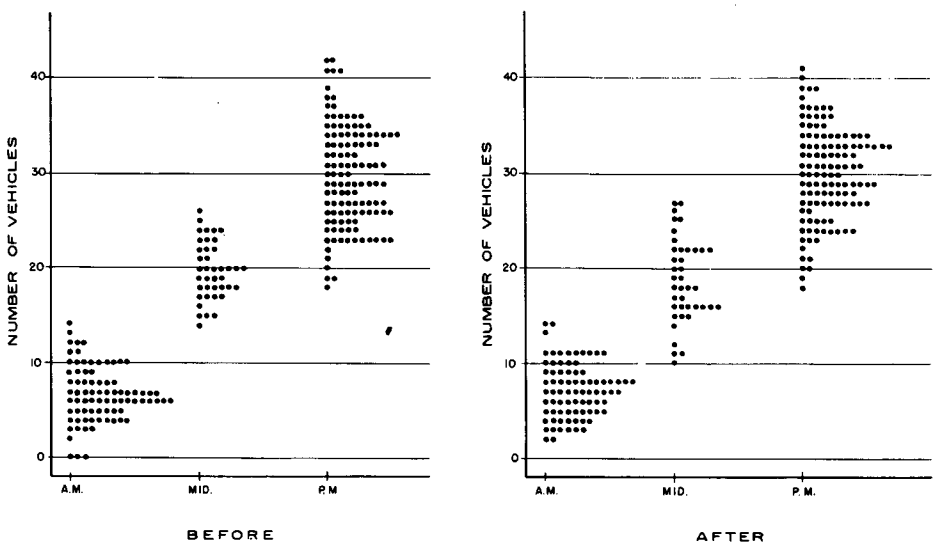


Figure G-39. Vehicles through per cycle, West Market Street northbound.

probably due to the capacity limitations of the upstream intersection, which could not feed additional vehicles to this approach.

Number of Saturated Cycles.—There was a significant decrease in the number of saturated cycles on Central Avenue westbound and on West Market Street northbound during the evening peak hour, after marking the additional lane on Central Avenue. Before the change, 97 percent of the cycles were saturated on Central Avenue westbound. This was reduced to only 4 percent after the revision.

A reduction of saturated cycles was experienced also on West Market Street northbound during the evening peak hour. This reduction, from 52 percent "before" to 17 percent "after," is probably due to the decrease in the time required to clear Central Avenue westbound. Prior to the revision, the Central Avenue traffic often usurped the right-of-way long after its green interval, holding back the West Market Street traffic.

The improvement for each approach leg during an evening peak hour is shown in Figures G-40 and G-41.

A chi square analysis of the data in Table G-24 indicated a significant reduction of saturated cycles at the $\alpha = 0.05$ level between the "before" and "after" conditions for the PM peak traffic on both approaches.

Convenience and Safety.—By visual observation during the "before" study, the queue of vehicles westbound on Central Avenue extended approximately 1,100 ft to the next traffic signal on most cycles during the evening peak period, and this backup continued until 6 PM.

During the "after" study, the queue was appreciably shorter on most cycles, and there was little congestion after 5:30 PM.

TABLE G-23

ALTERNATE ANALYSIS, CENTRAL AVENUE AND WEST MARKET STREET, VEHICLES THROUGH PER CYCLE

APPROACH	TIME PERIOD	CONDITION	VARIANCE	ARE VARIANCES EQUAL?	TEST	MEAN VALUES	DIFFERENCE	SIG. @ $\alpha=0.05$
Central Ave. WB	AM	"Before"	9.30	Yes	t	12.8	−1.5	Yes
		"After"	12.70			11.3		
	Midday	"Before"	10.05	Yes	t	16.9	+1.7	Yes
		"After"	16.36			18.6		
	PM	"Before"	31.48	Yes	t	25.7	+0.3	No
		"After"	26.98			26.0		
West Market St. NB	AM	"Before"	8.28	Yes	t	6.7	+0.5	No
		"After"	7.60			7.2		
	Midday	"Before"	9.10	Yes	t	19.8	+1.6	Yes
		"After"	14.67			21.4		
	PM	"Before"	30.26	Yes	t	29.7	+0.4	No
		"After"	23.40			30.1		

TABLE G-24

NUMBER OF SATURATED CYCLES, CENTRAL AVENUE AND WEST MARKET STREET

APPROACH	TIME PERIOD	CONDITION	CYCLES SATURATED	CYCLES NOT SATURATED	CYCLES ALL
Central Ave. WB	AM	"Before"	1	73	74
		"After"	0	80	80
		All	1	153	154
	PM	"Before"	116	3	119
		"After"	5	115	120
		All	121	118	239
West Market St. NB	AM	"Before"	0	80	80
		"After"	1	79	80
		All	1	159	160
	PM	"Before"	52	48	100
		"After"	21	99	120
		All	73	147	220

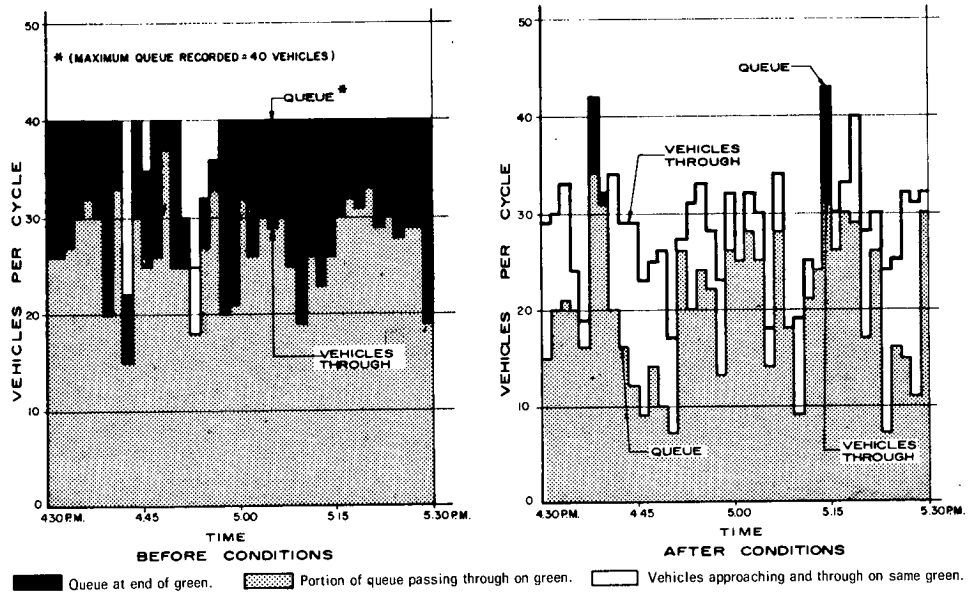


Figure G-40. Vehicles through and queue lengths per cycle, Central Avenue westbound.

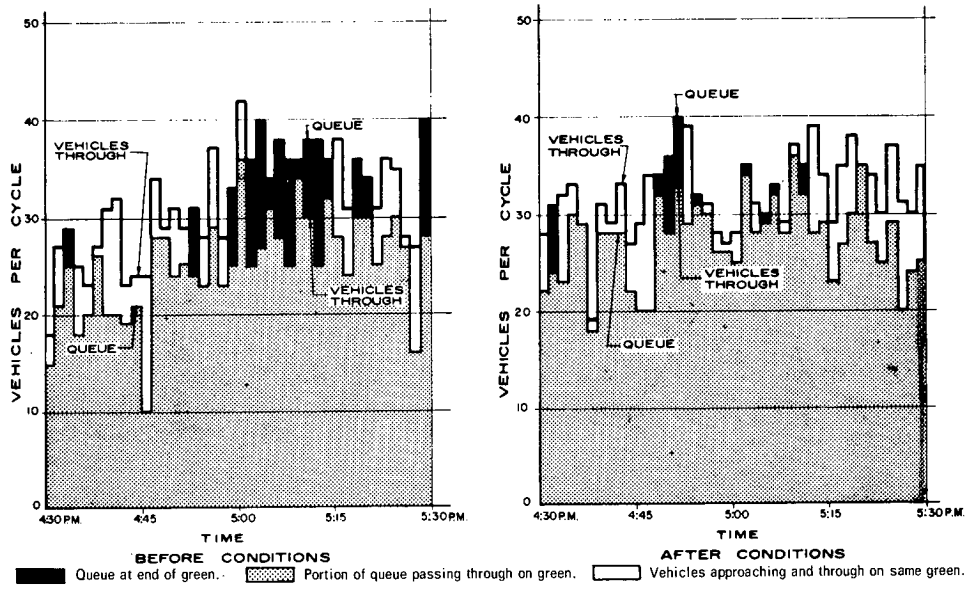


Figure G-41. Vehicles through and queue length per cycle, West Market Street northbound.

The additional lane was used primarily for storage during the red interval. During the green interval, most drivers used the two left lanes, with the right lane used primarily by buses and right-turning vehicles.

In addition, during the "before" study, westbound vehicles on Central Avenue required all of the available green time and had not cleared the intersection when West Market Street received a green indication. During the "'after" study, unused green time at the end of the phase allowed the intersection to clear before the West Market Street traffic started to move. Figure G-40 shows the relationship between westbound vehicles stored at the end of the red interval and the number of vehicles that cleared the intersection on the following green interval. This survey was conducted between 4:30 and 5:30 PM, when 39 of 40 cycles were saturated. For the same period, after the third lane was in use, the number of saturated cycles decreased from 40 to 3.

On West Market Street northbound, without any changes, the number of saturated cycles decreased from 15 to 9, out of a total of 40 cycles (Fig. G-41).

The reduction of saturated cycles implies a commensurate reduction in the amount of time that drivers are required to wait during green intervals. This is a decided advantage when the convenience of the traveling public is being considered.

Although the "after" period was too short to permit a valid analysis of accidents, a review of the accident records reveals that 11 accidents, mostly rear-end collisions, were reported in 1966, and 5 were reported in 1967. A reduction in the number of vehicles stopped should produce a decrease in the number of rear-end collisions.

Lane Control Markings, Main Street and Third Street—Experiment D67

The purpose of Experiment D67 is to determine the effectiveness of supplemental lane control pavement markings and curb-mounted signs on lane use when used in conjunction with overhead lane control signs. This investigation was conducted in the vicinity of the intersection of Main and Third Streets in Louisville (Fig. G-42).

U.S. 31E and U.S. 460, after crossing the Clark Memorial Bridge, join Main Street at Second Street, and the routes continue west along Main Street, which is one-way. Both routes turn north on Third Street, which is a two-way street north of Main Street. South of Main Street, Third Street is one-way southbound and is a major arterial crossing the CBD. The volume of traffic turning south on Third Street is considerably more than the traffic turning north. The number of trucks is also relatively large.

Experimental Area

East of its intersection with Third Street, Main Street is 67 ft wide and marked for six travel lanes. West of the intersection, it is 60 ft wide and marked for four travel lanes. East of the intersection, on the north side, parking is prohibited at all times; stopping is prohibited from 7 to 9 AM and 4 to 6 PM. Stopping is not permitted at any time on the south side of Main Street east of the intersection. Lane-use control devices consisting of overhead signs are suspended over the Main Street approach (Fig. G-43). Third Street is 42 ft wide and is marked for two southbound lanes on the southern leg of the intersection and for one northbound lane and three southbound lanes on the northern leg of the intersection.

The traffic signal at Main Street and Third Street is a part of the progressive system on Main Street. The signal is set so that 49 percent of the cycle time is allotted to Main Street and 51 percent is alloted to Third Street. The signal at Main and Second Streets is fully actuated. Therefore, Main Street traffic platoons arriving at Third Street may not be in progression.

Traffic volumes for the morning and afternoon peak hours of an average weekday (AWDT) in 1968 are shown in Figure G-44.

Design of Experiment

Observations of traffic turning left from Main Street into Third Street indicated that traffic was not using the second (optional) turn lane. Four sets of arrows were installed on the Main Street approach, and a sign entitled THIS LANE MUST TURN LEFT was installed adjacent to the southernmost left-turn lane on Main Street (Fig. G-43). These improvements were completed by November 1, 1968.

The following "before" and "after" measurements were recorded to determine changes in traffic flow through the intersection:

1. The number of vehicles turning left from Main Street into Third Street by lane on the green signal indication and on the red signal indication. A sign at this location permits a left turn on red after stopping.
2. The number of vehicles proceeding straight on Main Street from each lane.
3. The number of vehicles stopped during the red signal indication on Main Street per lane per cycle.

These measurements were confined to the three southern lanes on Main Street.

The "before" measurements were taken on Thursday, October 17, 1968; Monday, October 21, 1968; and Thursday, October 24, 1968. The first "after" measurements were taken on Thursday, November 7, 1968, thereby allowing a period of six days during which drivers had the opportunity of familiarizing themselves with the new conditions. Additional "after" measurements were obtained on Monday, November 11, 1968, and Tuesday, November 12, 1968.

Analysis and Conclusions

The number of vehicles stopped on red, the number of vehicles through, the number of left turns by lane, and the number of vehicles stopped in each of the three lanes were analyzed to determine the effectiveness of the pavement markings in improving lane use. Analyses were performed on data recorded during the three time periods of (1) 7:30 to 9 AM, (2) 9 AM to 4 PM, and (3) 4 to 5:30 PM. Initially, "before" data surveyed on Monday, October 21, 1968, and "after" data for Monday, November 11, 1968

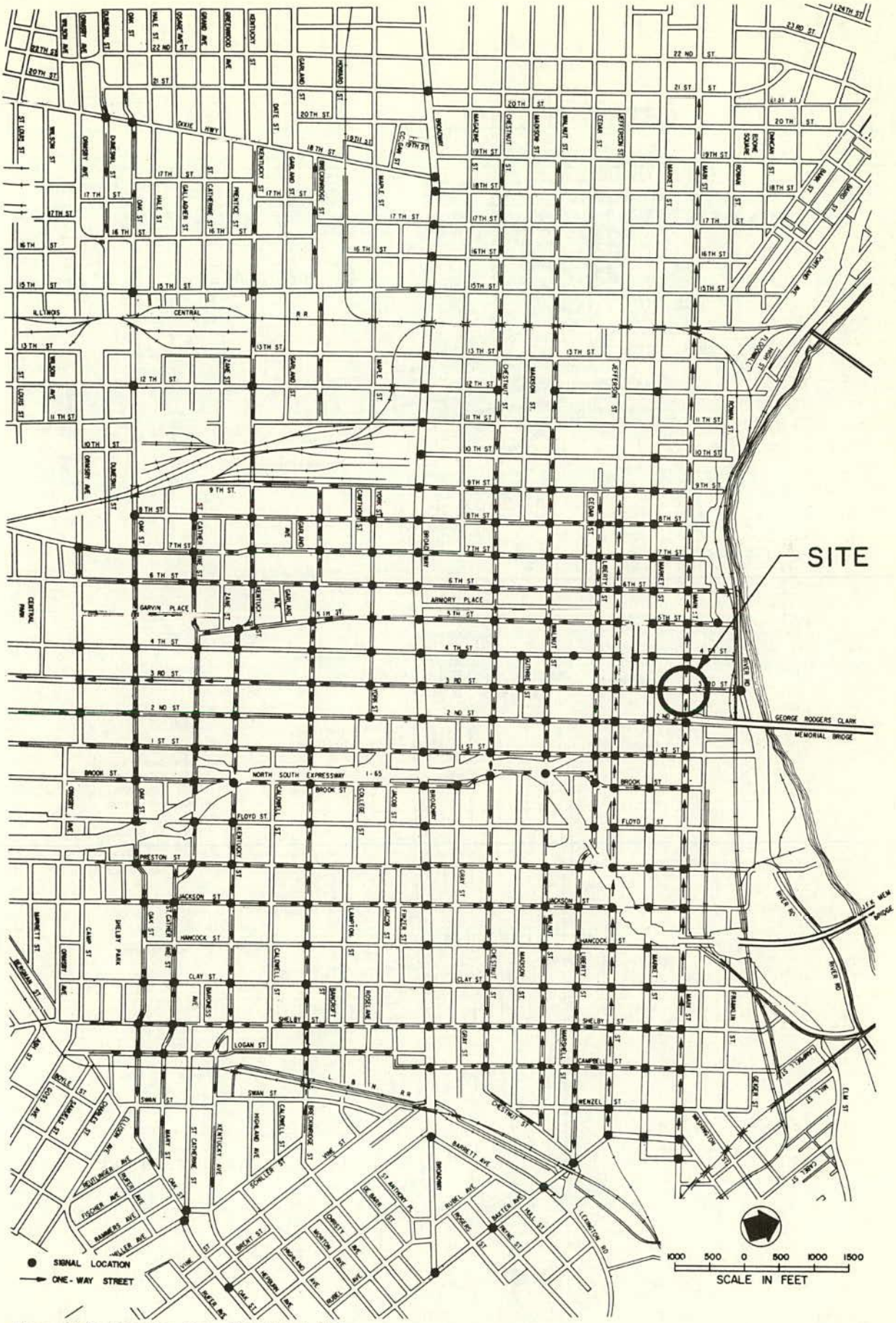


Figure G-42. Location map, Experiment D67.

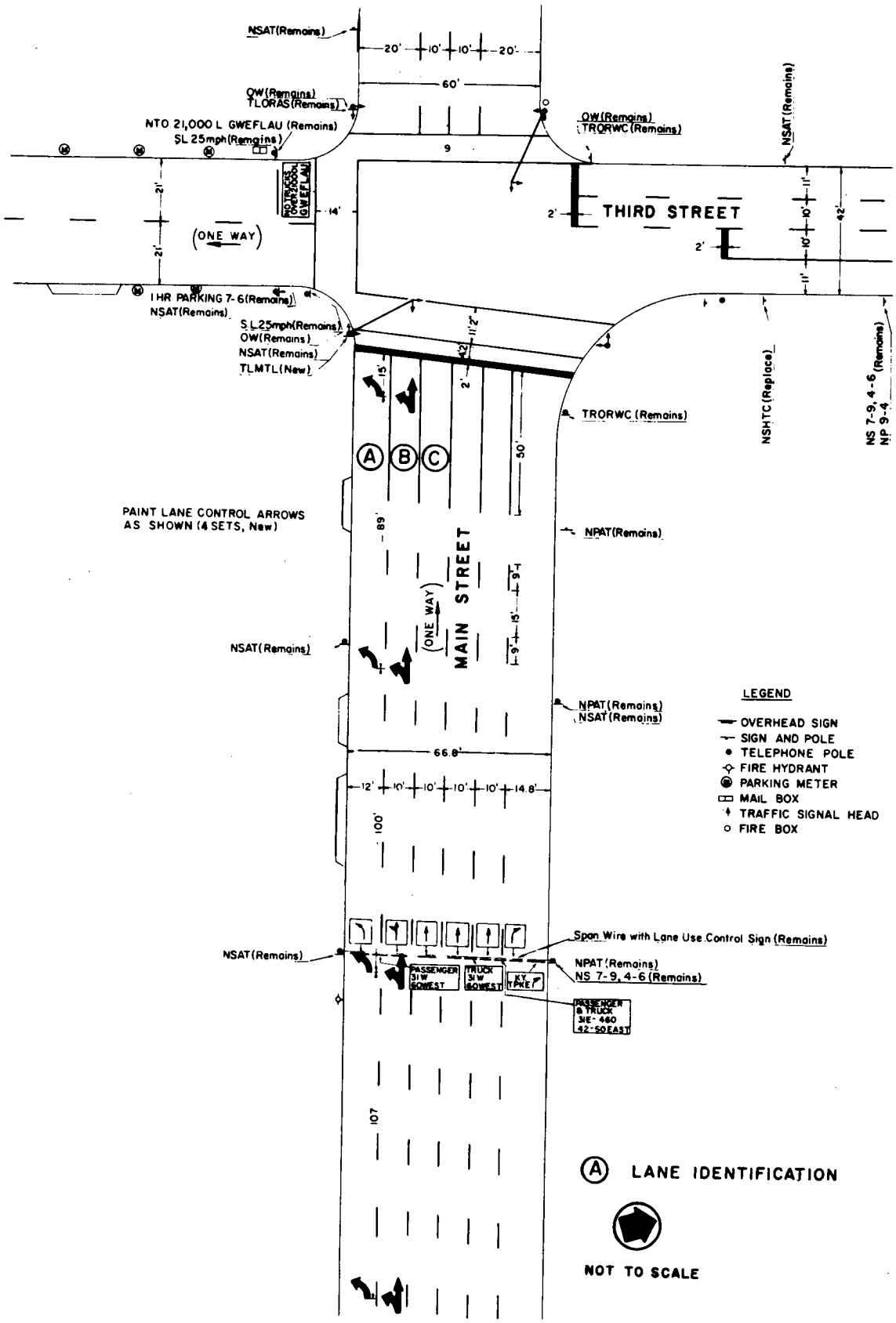


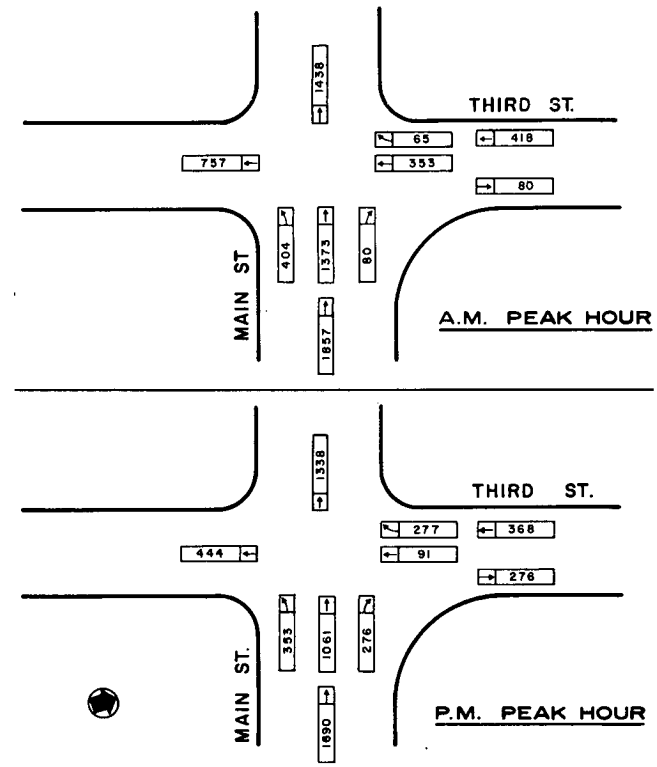
Figure G-43. Design plan.

(Veterans' Day), were included in the analysis; however, these measurements were eliminated after preliminary review showed that information gathered on November 11, 1968, was found to be significantly different from that gathered on the other days.

Surveyed data were summarized by cycle, tabulating the number of vehicles stopped on red, number of left turns per lane, and the number of vehicles through per lane. These data were recorded directly from the field sheets in accordance with the lane and movement designations shown in Figure G-45.

Analysis of the measurements indicated that the supplementary pavement arrows and a lane control sign did not produce a change in the proportion of vehicles turning left from each of the two left-turn lanes (Table G-25). However, a significant shift from the left-turn lanes (Lanes A and B) to the straight-through lane surveyed (Lane C) was noted for vehicles traveling straight through the intersection. The analysis also indicated the number of vehicles through in each lane was more evenly distributed during the "after" period (Table G-26).

There might have been greater use of the auxiliary left-turn lane if the LEFT TURN ON RED AFTER STOP sign had not been present. Left-turning drivers may have been reluctant to use the auxiliary lane, because they might have been stopped on red behind a straight-through vehicle.



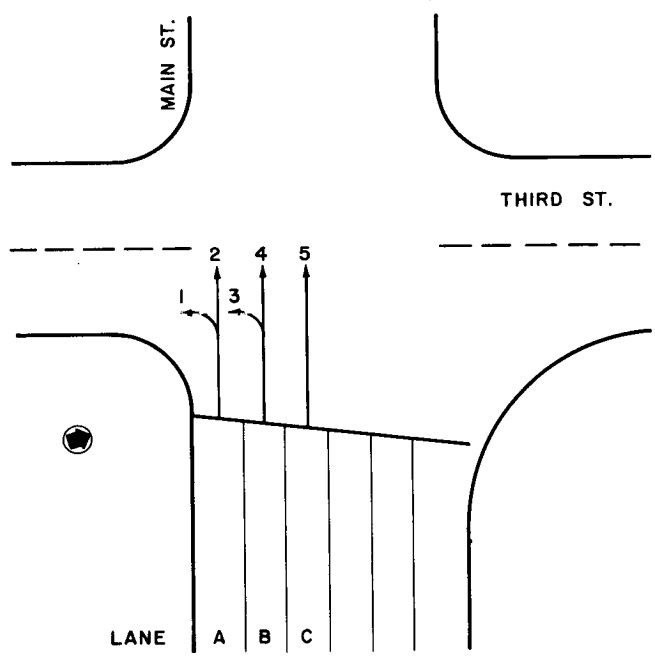


Figure G-45. Lane and movement designation.

Raymond Boulevard and McCarter Highway Left-Turn and Pedestrian Control—Experiment A7

Experiment A7 was designed to improve capacity, safety, and convenience of all forms of traffic traversing the experimental area. The key location within this area is the Raymond Boulevard-McCarter Highway intersection. Most of the field measurements made for evaluating changes in traffic flow, queue lengths, etc., were made at this location.

Improvements made within the study area include revisions to lane striping to provide auxiliary left-turn storage lanes and better definition of lanes for through and right-turning vehicles; revisions to existing parking regulations during hours of peak traffic flow made necessary by the changed lane marking; revisions to phasing and timing of the traffic signal to provide protected left-turn movements as necessary to accord with traffic desires; and regulation of the flow of pedestrians on crosswalks to reduce vehicle-pedestrian conflicts.

Experimental Area

The area involved in Experiment A7 is bounded on the east by the intersection of Raymond Plaza East with Raymond Boulevard; on the west by the intersection of Pine Street with Raymond Boulevard; on the south by the intersection of Market Street with McCarter Highway; and on the north by the intersection of Centre Street with McCarter Highway. The location of this experimental area with respect to the Newark study area is shown in Figure G-46.

Raymond Boulevard is one of the major east-west arterial streets of downtown Newark, and McCarter Highway (N.J. 21) is a major north-south arterial extending beyond Newark to serve communities of the Passaic River Valley. The high volume of southbound traffic on McCarter Highway desiring to turn east onto Raymond Boulevard and the high percentage of heavy trucks using McCarter Highway are evident in the survey data (Fig. G-50).

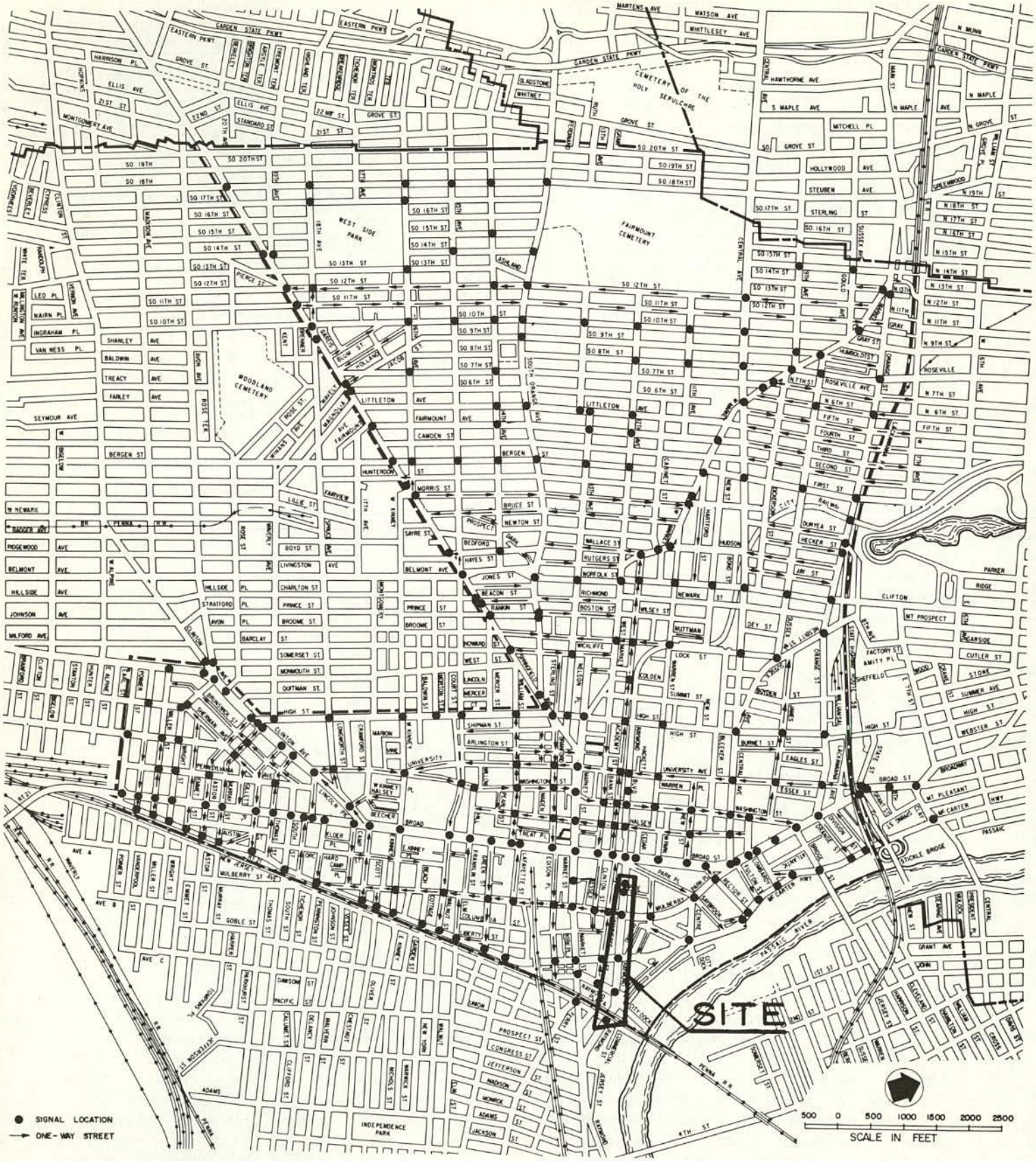


Figure G-46. Location map, Experiment A7.

TABLE G-25

NUMBER OF LEFT TURNS PER CYCLE

LANE	"BEFORE" VEH/CYCLE	"BEFORE" %	"AFTER" VEH/CYCLE	"AFTER" %	DIFFERENCE (VEH/CYCLE)	CHANGE (%)
A	5.0	89	5.3	86	0.3	6.0
B	0.6	11	0.9	14	0.3	50.0
Total	5.6	100	6.2	100		

TABLE G-26

LANE USE

MOVEMENT	CONDITION	LANE USE (%), BY LANE A	B	C	ALL
Straight only	"Before"	0.8	49.5	49.7	100.0
	"After"	0.4	45.3	54.3	100.0
Straight and left	"Before"	31.9	36.0	32.1	100.0
	"After"	32.2	33.7	34.1	100.0

Details of pavement widths, markings, regulations governing traffic operations, signal phasing, and signal timing in operation at the time the experiment was begun are shown in Figures G-47, G-48, and G-49. The traffic signal offset plans in effect at that time are shown in Figures G-51 and G-52.

Convenience and Safety

Shortly after the beginning of the project, Commerce Street, between Raymond Plaza West and McCarter Highway, was permanently closed for an urban redevelopment project. This closing forced many pedestrians walking between the downtown area and the Pennsylvania Railroad Station to relocate their path from Commerce Street to Raymond Boulevard. These pedestrians mainly used the south crosswalk at McCarter Highway, adding severely to existing congestion during the periods from 7 to 9 AM and from 4 to 6 PM. The Commerce Street closing also rerouted a number of local bus lines which, with other diverted traffic, resulted in a 500-percent increase in volume of east-to-south left-turning (Fig. G-50).

The presence of parked vehicles on the north curb of Raymond Boulevard between McCarter Highway and Raymond Plaza West severely limited the capacity for westbound Raymond Boulevard traffic. A similar condition existed at the south curb of Raymond Boulevard between McCarter Highway and Mulberry Street. The service station in the northeast quadrant daily parked service vehicles in the curb lane either on McCarter Highway or Raymond Boulevard, adding to the turbulence being experienced.

Prior to implementation of the improvement plan, traffic operations at the Raymond Boulevard-McCarter Highway intersection during the morning and evening periods of peak traffic flow could be described as chaotic. The severity of the congestion resulting from increased traffic volumes was drastically multiplied by the disregard displayed by many motorists and pedestrians for the traffic signal control and a lack of consideration for the rights of others using this intersection.

The reasons for this general attitude of disrespect toward traffic regulation were lack of enforcement and inadequacies of signal controls and lane markings. Absence of marked auxiliary left-turn lanes resulted in joint occupancy of the lane adjacent to the center line on all approaches by both through and left-turning vehicles. The detrimental effect of this condition on operations was most noticeable on the north approach.

At the beginning of Phase C (Fig. G-48), if traffic desiring to proceed from north to south in the center lane blocked a motorist desiring to turn left, the latter often turned out of this lane and drove south in the northbound lanes, sometimes for as much as 200 ft in advance of the intersection. This often resulted in a double line of north-to-east vehicles turning in the intersection.

The briefness of the protected left-turn phase, about 10.6 sec, in the presence of a continual heavy demand also resulted in the north-to-east left-turning vehicles tailgating each other into and through the intersection to prevent south-to-north vehicles from beginning their movement at the start of Phase A, even though the latter movement had been assigned the legal right-of-way by the traffic signal control. In turn, south-to-north vehicles would continue to enter the intersection after the beginning of Phase B. This situation was further aggravated by pedestrians occupying the east crosswalk during most or all of Phase C, which effectively blocked left-turning vehicles from clearing the intersection. These pedestrians normally began their crossing at the end of Phase B and governed their crossing strictly by the stopping of east-west traffic flow.

With two significant differences, a similar situation continually occurred during Phase B between east-to-south left-turning vehicles and those vehicles traveling from west to east. The left-turning vehicles in this situation did not have the right-of-way and the pedestrians rightfully entered the crosswalk at the beginning of Phase B.

It is interesting to note that these east-to-south left-turning motorists deliberately blocked the west-to-east through movement, even though they were aware that they would be unable to enter the south crosswalk until the heavy pedestrian movement had crossed.

The great majority of pedestrians using this intersection are commuters walking in an eastbound or westbound direction between the Pennsylvania Railroad Station and downtown Newark. Most pedestrians crossing Raymond Boulevard are rail commuters having an origin or destination in the downtown area north of Raymond Boulevard. A common practice of a great many of these pedestrians is to jaywalk across Raymond Boulevard between McCarter Highway and Raymond Plaza West.

At the beginning of this project, the intersection of Raymond Boulevard with McCarter Highway was identified from police accident records as the location having the

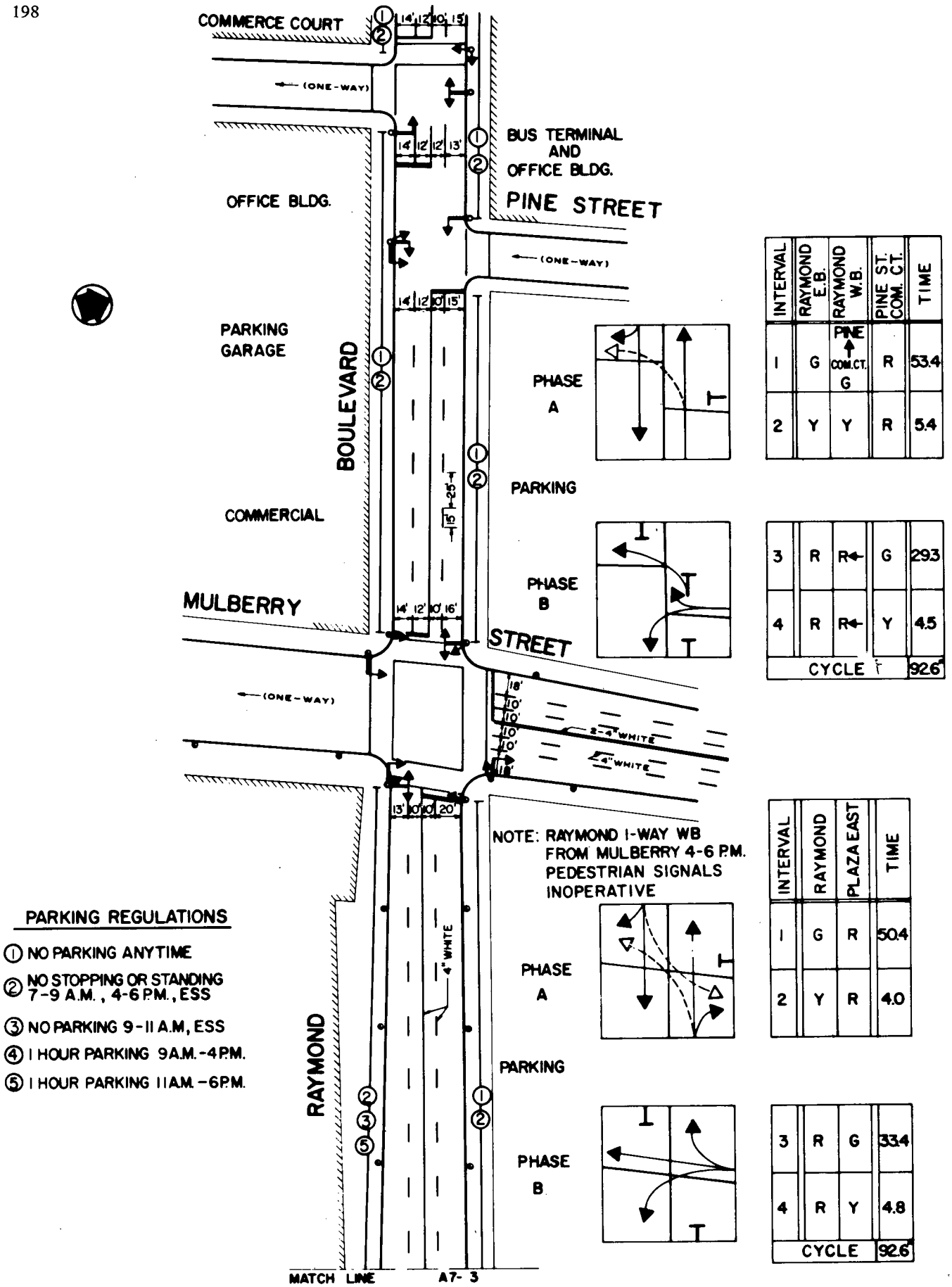


Figure G-47. Vicinity map, "before."

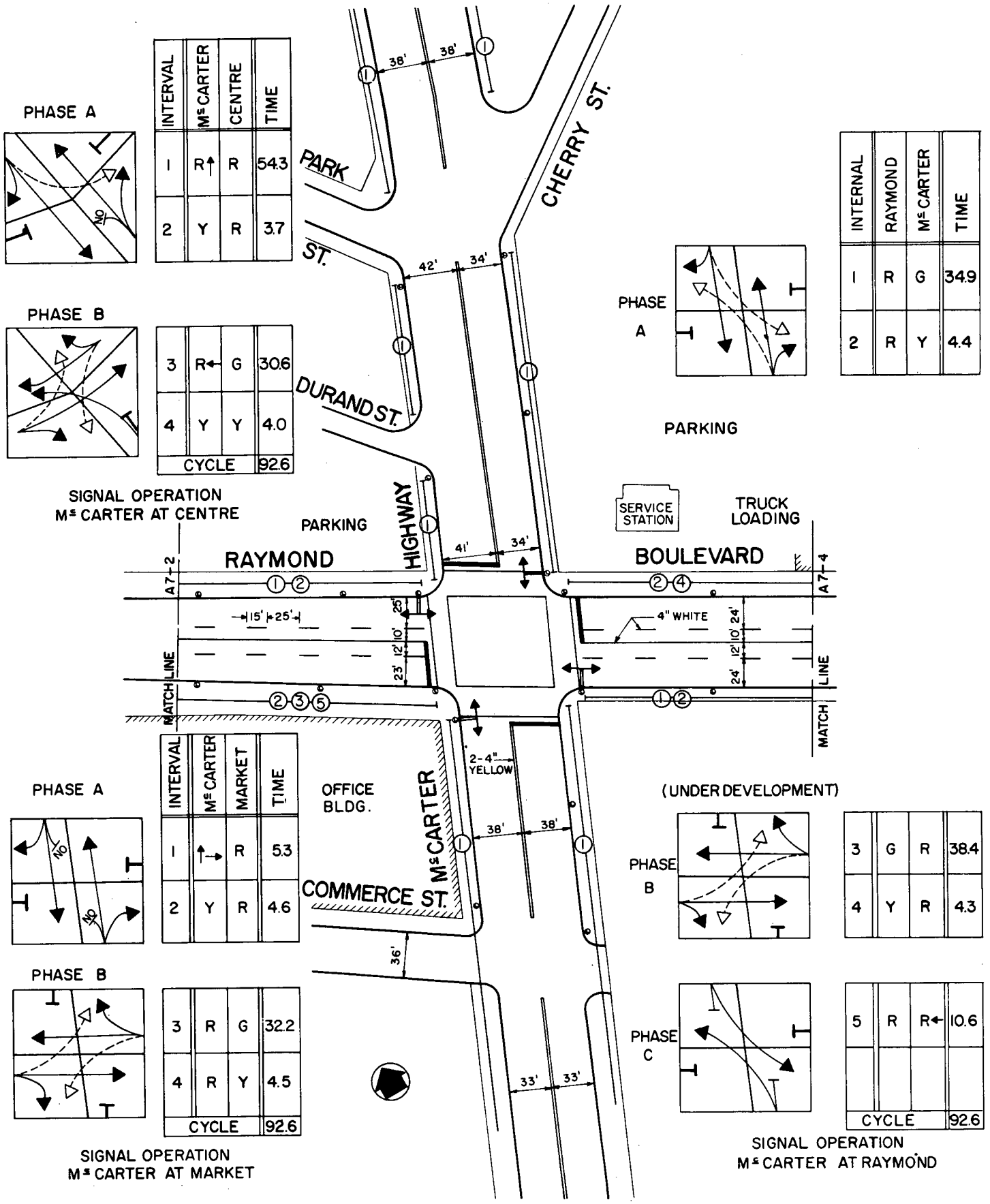


Figure G-48. Vicinity map, "before."

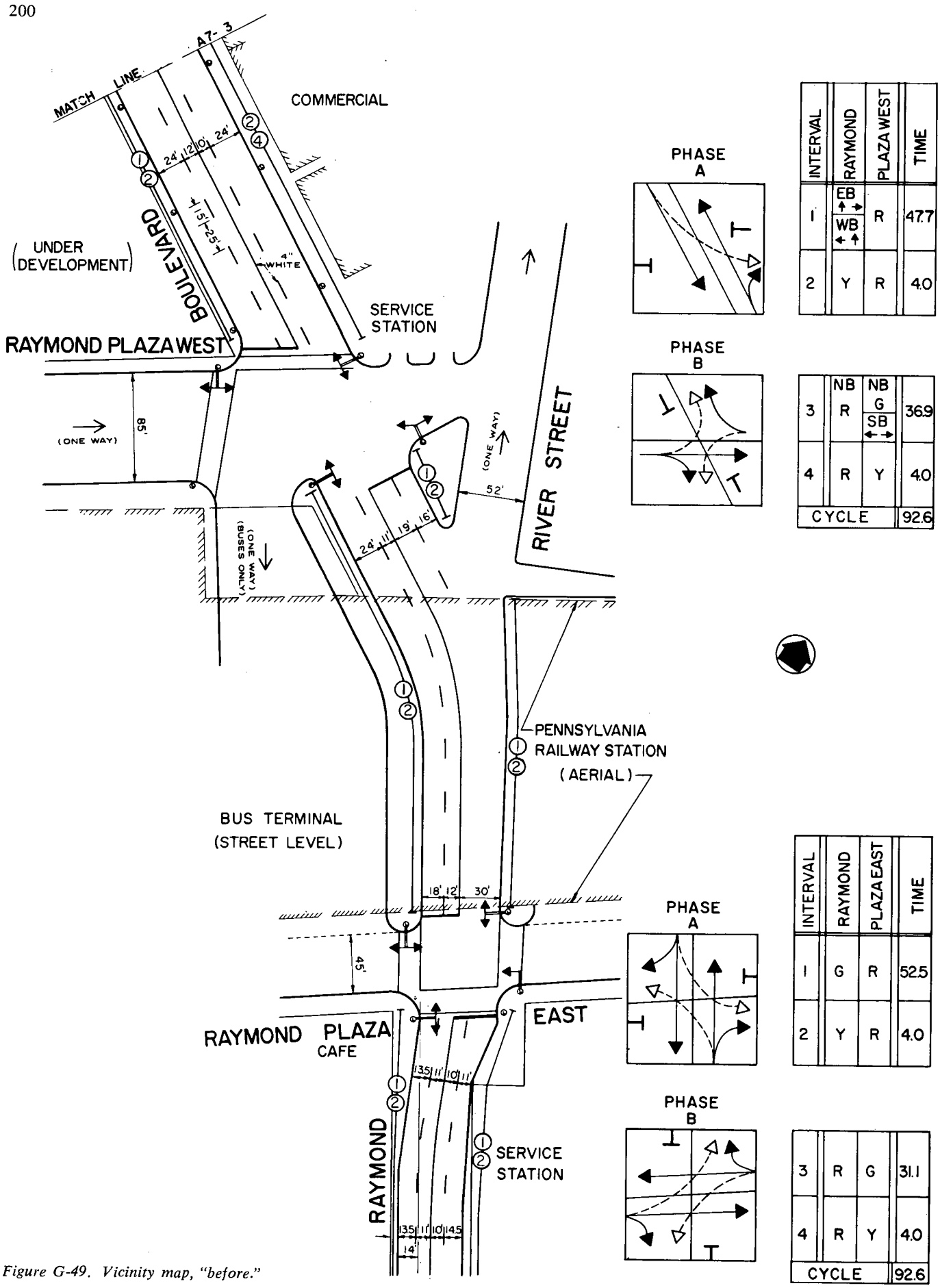


Figure G-49. Vicinity map, "before."

highest frequency of accidents in the study area. A pilot study of accidents at this location was made. The findings of this study showed that a total of 326 collisions were reported at this location for the years 1961 through 1966, inclusive—an average of over 50 collisions per year. The majority of the collisions involved southbound vehicles turning east in conflict with northbound vehicles. A high frequency of right-angle collisions involving vehicles disobeying the traffic signal control was also noted. Collision diagrams for the years 1938, 1940, and 1941 reveal that this location experienced a total of 75 collisions for those three years.

Of the vehicles involved in the 326 reported accidents, 90 percent were passenger cars. Only 6 buses and 13 tractor-trailer trucks were involved. Two fatal accidents, one involving a pedestrian, and 137 personal injury accidents are included in this total. Rear-end collisions were the most frequent type of accident, comprising approximately 80 percent of the total. One-third of the rear-end collisions involved a vehicle stopped while waiting to make a left turn.

Experimental Design

The traffic-signal phasing and associated timing that was in operation prior to this experiment (Fig. G-48) provided a simultaneous, protected left-turn phase for the opposing movements on McCarter Highway. This gave no recognition to the unbalanced demand as indicated by the data shown in Figure G-50, or to the fact that exclusive left-turn lanes were not provided. Demand for the east-to-south left turn was not considered.

The plan of improvement was designed and implemented in stages to permit a separate determination of the effects of each change on traffic. Two stages were planned:

1. Provision for lane lines and auxiliary left-turn storage lanes on each approach.
2. Revisions of signal operation and timing to provide

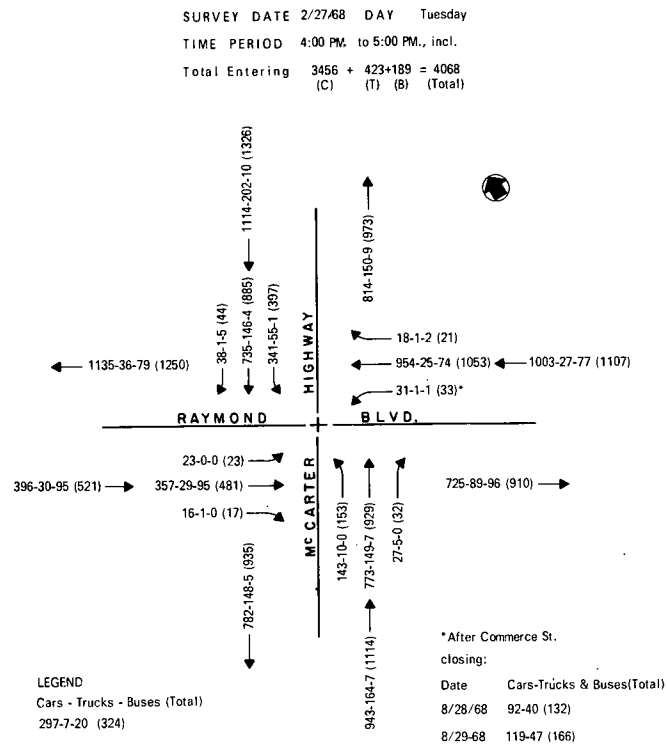


Figure G-50. Volume survey, "before."

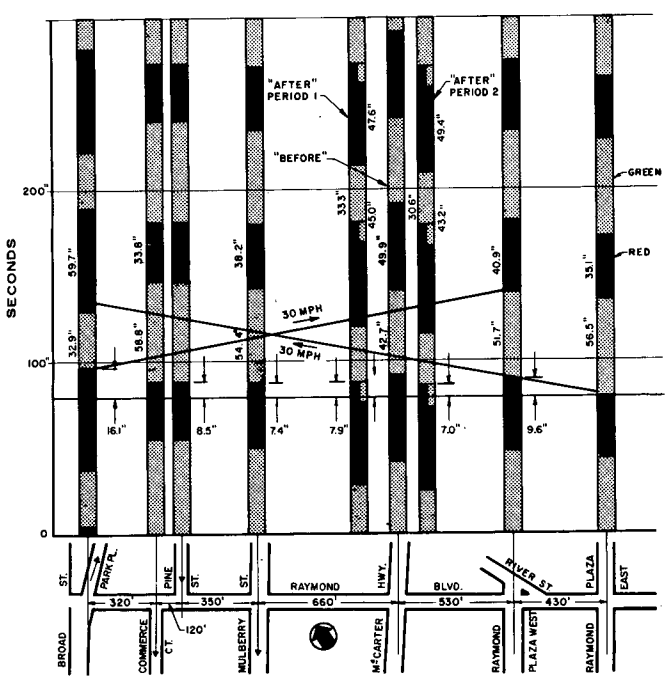


Figure G-51. Offset plans, Raymond Boulevard.

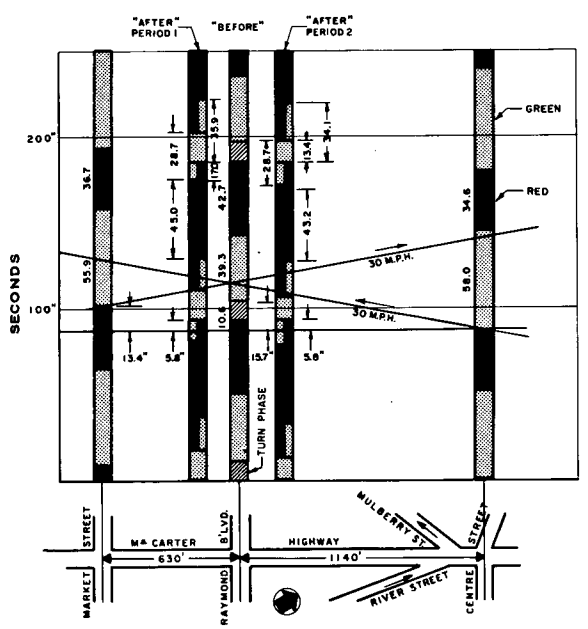


Figure G-52. Offset plans, McCarter Highway.

pedestrian control and separate phases for high-volume left turns.

Manpower requirements for adequate surveillance of traffic operations necessitated dividing the lane marking stage into two parts. The first part dealt with Raymond Boulevard and the second dealt with McCarter Highway.

To provide auxiliary left-turn lanes for east- and westbound Raymond Boulevard traffic at McCarter Highway, it was necessary to re-mark Raymond Boulevard between Raymond Plaza East and Pine Street (Figs. G-53, G-54, and G-55). This plan required that all parking, standing, or stopping at both north and south curbs between these limits be prohibited during periods of peak traffic. Regulations listed and shown in Figures G-47, G-48, and G-49 indicate that most of the required ordinances were in existence. A field inspection revealed that signs stating these regulations were posted but were ineffective. Daily, vehicles were observed parked illegally at the north curb between Raymond Plaza West and McCarter Highway and at the south curb between McCarter Highway and Mulberry Street. No enforcement action against this illegal parking was observed.

The fact that a large percentage of parking meters in the study area are missing, broken, or inoperative, coupled with a low schedule of fines, is partially responsible for the public disregard of parking regulations. The fact that enforcement is inconsistent or lacking altogether adds to this attitude.

The city agreed to effect the changes required by the plan of improvement and to ensure enforcement of all parking regulations for the duration of the experiment.

The marking plan, as designed, specified that lane use signs, required by the *Manual on Uniform Traffic Control Devices,* were to be installed over each auxiliary left-turn lane. However, these signs were not installed.

The second part of the marking plan provided for auxiliary left-turn lanes for McCarter Highway at Raymond Boulevard, as well as defining three through lanes for each approach (Fig. G-54). It should be noted that the lack of lane lines on McCarter Highway (Fig. G-48) resulted in both north- and southbound traffic consistently forming in three lines, both as stopped queues and as moving traffic. This part of the marking plan required no revisions to the existing posted parking regulations.

The second stage of work involved revising the existing traffic signal phasing to that shown in Figure G-56. This required replacement of the existing traffic signal control equipment and rewiring of the underground electrical distribution system. Controller replacement was necessary, as the existing control equipment was incapable of providing the design sequence. Wiring revisions provided a sufficient number of electrical conductors to connect the added pedestrian and vehicular signal indications required by the design sequence.

The revised phasing was provided by means of a three-dial, pretimed controller, modified to coordinate with the PR-type signal system, and further modified to provide a flashing DON'T WALK pedestrian clearance interval. The revised phasing displayed signal indications informing pedestrians in each crosswalk of the appropriate time to walk and ensured that an adequate pedestrian clearance interval was provided. To minimize potential conflicts, pedestrian crossing intervals were separated from the advance and trailing protected left-turn phases. These latter phases were individually timed in accordance with the volume demands present for the time period being controlled.

The north-to-east left turn was provided a protected, trailing phase in coordination with the southbound platoon movement from the next upstream traffic signal at Centre Street. The east-to-south left turn was provided an advance protected phase, because these motorists habitually failed to lawfully yield right-of-way to the west-to-east through movement.

The signal offset relationships between the McCarter Highway-Raymond Boulevard intersection and adjacent intersections on both McCarter Highway and Raymond Boulevard were revised (Figs. G-51 and G-52). To avoid injecting an additional variable into the experiment, the offset relationships were altered as little as possible.

The signal revision stage was divided into two parts for surveillance and evaluation. The first part involved the implementation of the revised signal operation which, at the request of the city, was done without any public notice or other educational effort. The second part involved the use of police officers to reduce signal violations and control pedestrians.

Analysis

The two separate stages of improvement were implemented in parts. Each part was preceded and followed by surveys of vehicles stopped during the red signal interval and vehicles proceeding through the intersection during the following green interval. The surveys were conducted from 7:30 to 8:30 AM and 4:30 to 5:30 PM. Some additional data were taken during midday, 1 to 2 PM, for design purposes. This period was found to be very different from the morning and evening peak periods, largely owing to the reduction in pedestrian volumes. Only the peak periods were analyzed. The date and time of these surveys are given in Table G-27, and summaries of the observed traffic volumes for each of the experimental conditions are given in Tables G-28 and G-29. To determine the number of saturated cycles, the number of vehicles stopped and through were counted separately for each cycle, resulting in some vehicles being counted more than once. These vehicles are reflected by the mean number of vehicles not clearing during saturated cycles. A saturated cycle is defined as a cycle during which the number of vehicles stopped on the red signal interval exceeded the number of vehicles proceeding through the intersection during the following green signal interval.

The term "saturated cycle" should not be confused with the term "loaded cycle," as defined in the *Highway Capacity Manual* (*1*). Some observed cycles, classified by definition as "saturated," occurred when demand was light or less than average, owing to events occurring in specific lanes or within the intersection area, and the potential capacity was not being approached.

It should also be noted that the number of westbound vehicles on Raymond Boulevard indicated as being stopped

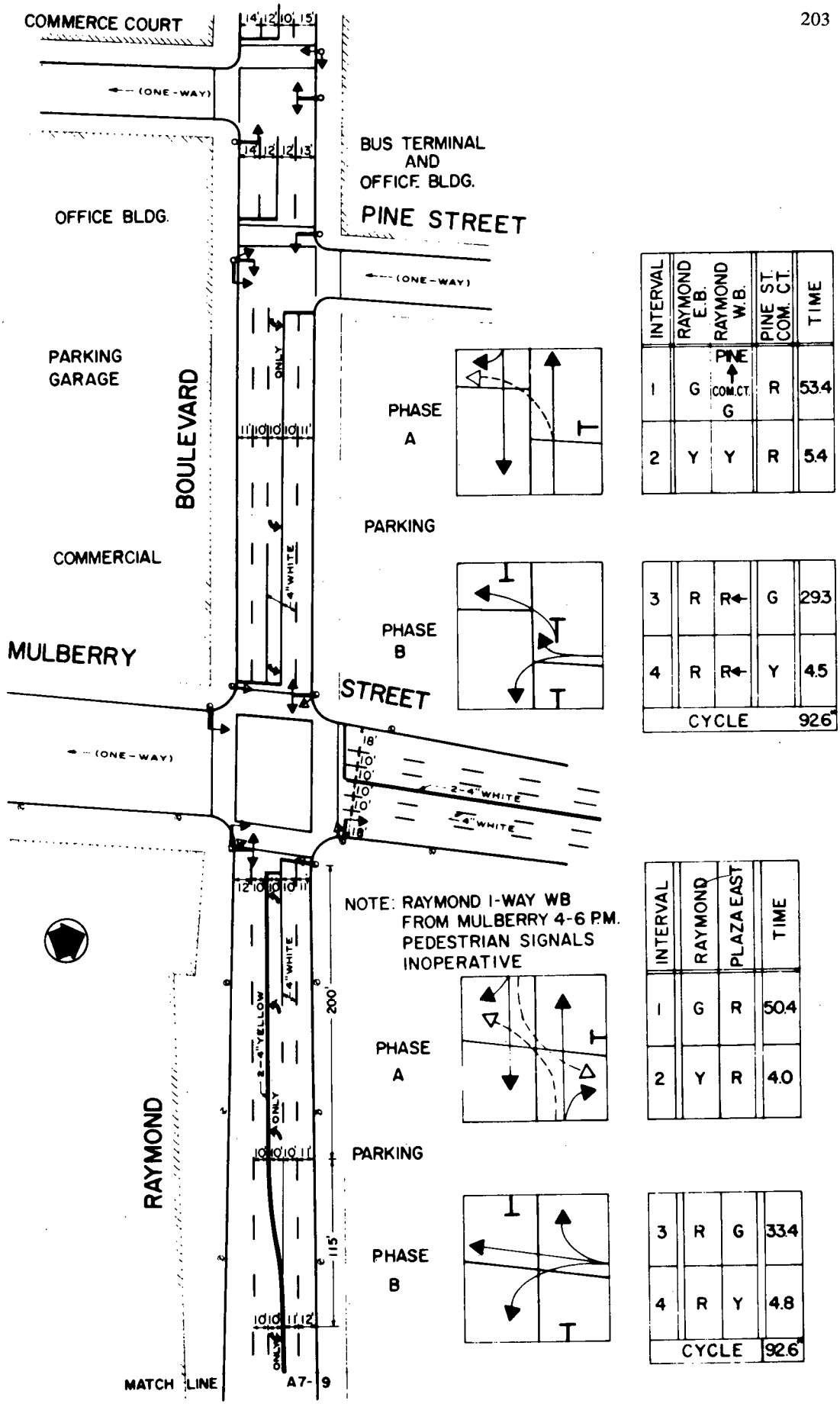


Figure G-53. Vicinity map, "after."

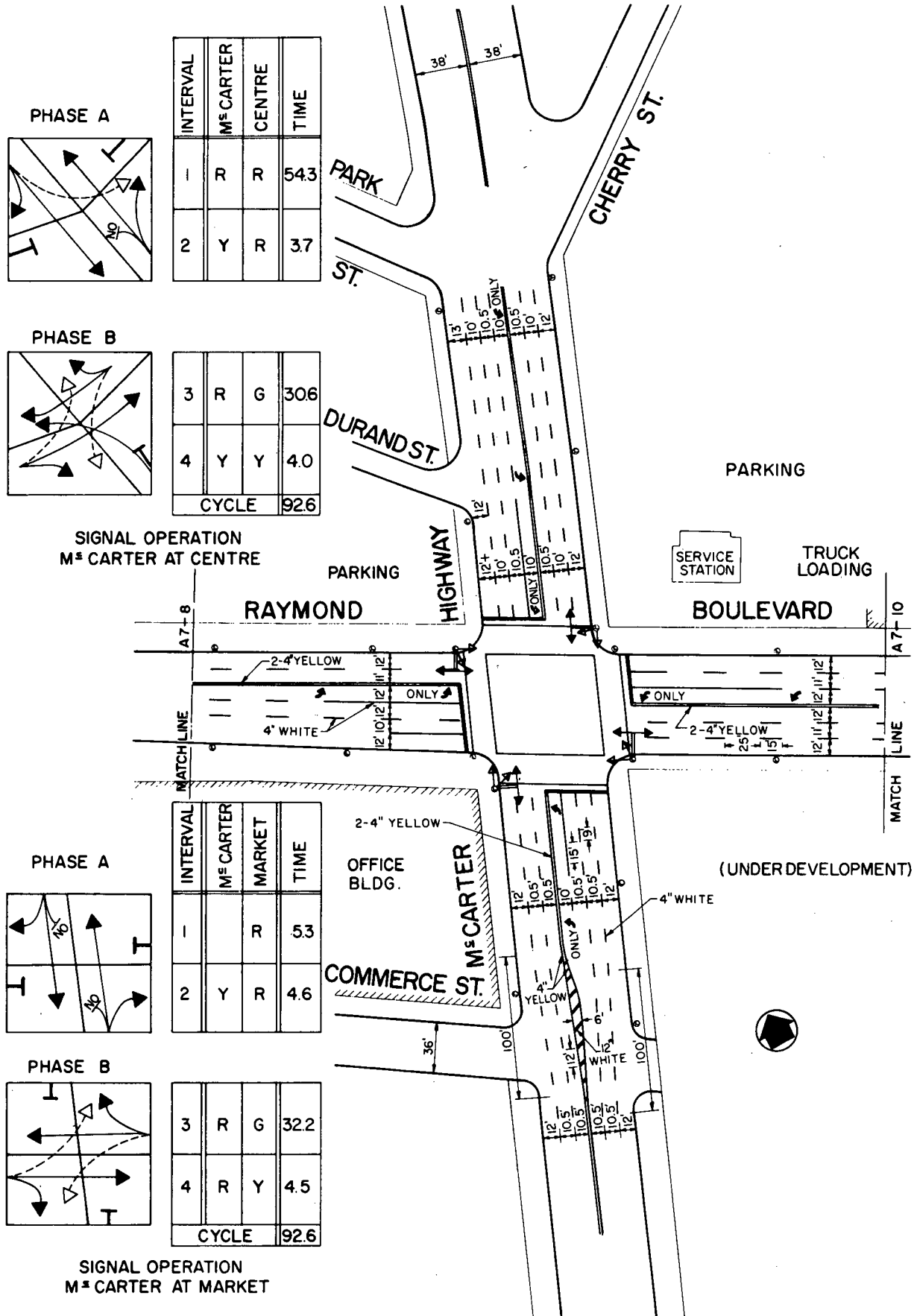


Figure G-54. Vicinity map, "after."

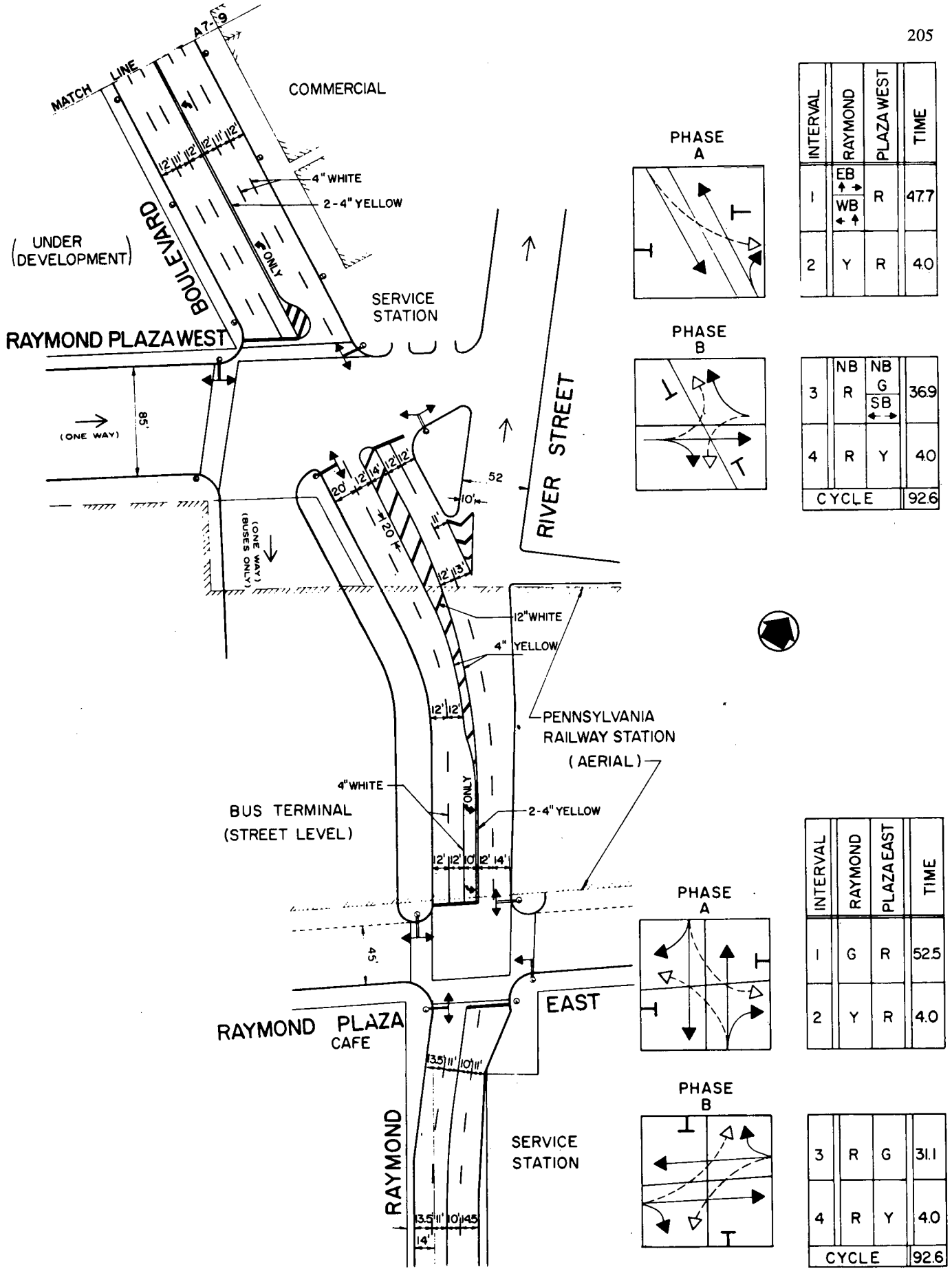
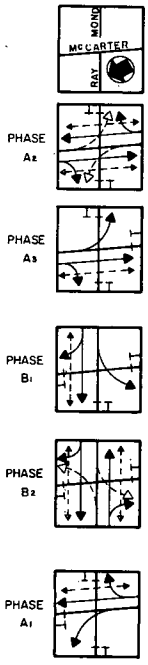


Figure G-55. Vicinity map, "after."



INTERVAL	COLOR SEQUENCE								SIGNAL INTERVAL TIMING*		
	McCARTER		RAYMOND		CROSSWALK						
	NB	SB	EB	WB	N	S	E	W	OFF PEAK	AM	PM
1	G	G	R	R	DW	DW	F DW	W	13.4	12.5	8.9
2	Y	G	R	DW	DW	DW	F DW	W	4.5	4.5	4.5
3	R	←G	R	R	DW	DW	DW	W	9.0	6.3	8.1
4	R	←G	R	R	DW	DW	DW	F DW	8.1	8.1	8.1
5	R	Y	R	R	DW	DW	DW	F DW	4.5	4.5	4.5
6	R	R	R	←G	W	DW	DW	DW	12.6	9.0	9.9
7	R	R	R	G	W	DW	DW	DW	2.7	2.7	2.7
8	R	R	G	G	W	W	DW	DW	11.7	18.9	15.3
9	R	R	G	G	F DW	F DW	DW	DW	9.9	9.9	10.8
10	R	R	Y	Y	F DW	F DW	DW	DW	4.5	4.5	4.5
11	←G	R	R	R	DW	DW	W	DW	7.2	7.2	7.2
12	←G	R	R	R	DW	DW	F DW	DW	1.8	1.8	5.4
13	G	R	R	R	DW	DW	F DW	DW	2.7	2.7	2.7
					TOTAL CYCLE				92.6	92.6	92.6

*Offpeak operation: 5:45 PM to 7:15 AM and 8:45 AM to 4:15 PM
AM operation: 7:15 AM to 8:45 AM PM operation: 4:15PM to 5:45 PM

Figure G-56. Traffic signal operations, "after."

does not totally reflect those actually stopping. The stopped queues could be counted only to the intersection of Raymond Plaza West with Raymond Boulevard. Queues extending beyond this point could not be seen by the survey personnel. In addition, the storage area available for stopped queues under the "before" conditions in the PM period was limited by vehicles illegally parked in the north curb lane. The surveyed increase in vehicles stopped during the "after" conditions can be attributed mainly to the increased storage area available for queued vehicles in the curb lane. The effect of illegal parking at this location was demonstrated by the data resulting from surveys made on October 15 and 16, 1968.

With no illegally parked vehicles present on October 15, there were no full-block-length queues observed; and the lanes as delineated were fully used, resulting in mean values of 36.0 and 22.8 for vehicles through and stopped per cycle, respectively.

With vehicles illegally parked in the north curb lane during a portion of the survey hour on October 16, 19 of the 40 cycles were observed to experience block-length queues. Even with the reduced storage area available for stopped queues, the mean number of vehicles stopped per cycle increased only 4.6 per cycle to 27.4. The reduction in available lanes resulting from these parked vehicles is also reflected by a reduction of 2.1 in the mean number of vehicles through per cycle to 33.9. This reduction took place even though almost half the observed cycles experienced maximum demand.

The limitations imposed by physical conditions in counting stopped vehicles are illustrated by the data of Oc-

TABLE G-27

1968 "BEFORE" AND "AFTER" VOLUME SURVEYS

DATA GROUP	CONDITION	DIRECTION	LOCATION	MON. PM	TUES. AM	TUES. PM	WED. AM	WED. PM	THURS. AM	THURS. PM	FRI. AM
1	"Before" Raymond marking	E-W	Mulberry and McCarter					8/28 [a]	8/29	8/29 [a]	8/30
2	"After" Raymond marking	E-W	Mulberry and McCarter					9/4 [a]	9/5	9/5 [a]	9/6
3	"Before" McCarter marking	N-S	McCarter	10/7	10/8	10/8	10/9				
4	"After" McCarter marking ("After" Raymond marking) ("Before" signal revision)	All	McCarter	10/14	10/15	10/15	10/16	10/16	10/17	10/17	10/18
5	"After" signal revision ("Before" enforcement)	All	McCarter		11/19	11/19	11/20	11/20	— [b]	11/21	— [b]
6	With enforcement	All	—		12/3 [c]	12/3	12/4 [d]	12/4 [e]	12/5 [f]	12/5	12/6 [g]

[a] WB only as Raymond is one-way WB from Mulberry 4-6 PM.
[b] Signal at Raymond-McCarter stuck—survey cancelled.
[c] Enforcement only for 10 min—officer late.
[d] 15-min survey without enforcement—officer absent.
[e] Heavy rain during survey.
[f] Signal at Raymond Plaza East stuck.
[g] Signal north on McCarter at Fulton St. stuck.

TABLE G-28

MEANS OF TRAFFIC VOLUMES OBSERVED ON RAYMOND BOULEVARD AT McCARTER HIGHWAY

	EB					WB					
		VEH/CYCLE		SATURATED CYCLES			VEH/CYCLE			SATURATED CYCLES	
CONDITION	DAYS	ALL THROUGH	ALL STOP	NO./HR	VEHICLES NOT CEARING/ CYCLE	DAYS	LEFT	ALL THROUGH	ALL STOP	NO./HR	VEHICLES NOT CLEARING/ CYCLE
					(*a*) 7:30–8:30 AM						
"Before"	2	26.1	14.8	0	0	2	2.9	19.6	15.0	4.0	5.4
"After" lane marking—1	2	29.2	28.0	16.0	5.1	2	3.1	18.7	7.7	0	0
"After" lane marking—2	4	29.4	18.2	1.5	1.0	4	3.6	21.6	10.7	0.3	3.3
"After" signal revision	3	28.7	23.9	8.7	8.1	3	3.9	21.4	14.0	1.3	1.8
"After" enforcement	1	26.2	19.0	2.0	6.0	1	4.2	21.1	13.1	0	0
					(*b*) 4:30–5:30 PM						
"Before"	2	12.9	6.0	0	0	2	3.7	33.1	26.1	6.0	3.9
"After" lane marking—1	2	14.2	8.9	3.0	6.2	1	3.8	31.9	17.2	0	0
"After" lane marking—2	4	14.5	8.1	0.5	7.0	4	4.6	34.9	25.8	4.0	7.1
"After" signal revision	3	14.2	6.9	1.0	1.0	3	4.4	33.1	29.1	8.3	4.6
"After" enforcement	2	13.6	7.7	1.5	4.0	1	4.5	32.7	29.9	1.2	5.8

Note: 40 cycles of signal operation were observed for each survey hour.

TABLE G-29

MEANS OF TRAFFIC VOLUMES OBSERVED ON McCARTER HIGHWAY AT RAYMOND BOULEVARD

	NB						SB					
		VEH/CYCLE			SATURATED CYCLES			VEH/CYCLE			SATURATED CYCLES	
CONDITION	DAYS	LEFT	ALL THROUGH	ALL STOP	NO./HR	VEHICLES NOT CLEARING/ CYCLE	DAYS	LEFT	ALL THROUGH	ALL STOP	NO./HR	VEHICLES NOT CLEARING/ CYCLE
						(*a*) 7:30–8:30 AM						
"Before"	2	1.4	19.1	13.1	0.5	2.0	2	8.2	27.8	19.7	4.5	3.9
"After" lane marking	4	1.2	19.2	12.4	0.3	1.0	4	9.1	29.4	20.0	5.5	3.8
"After" signal revision	3	1.2	18.1	21.9	25.7	6.3	3	9.3	29.3	25.4	5.3	4.8
"After" enforcement	1	1.5	18.9	22.0	30.0	4.4	1	8.7	28.4	23.4	6.0	4.5
						(*b*) 4:30–5:30 PM						
"Before"	2	4.6	21.9	14.5	1.0	1.5	2	7.3	32.5	45.2	27.5	21.1
"After" lane marking	4	5.1	22.6	15.0	1.3	1.6	4	10.7	36.8	28.8	5.5	10.7
"After" signal revision	3	5.3	22.0	22.5	14.7	3.6	3	10.7	35.2	35.8	17.3	8.3
"After" enforcement	2	5.2	21.9	26.3	25.0	7.7	2	10.0	33.8	35.9	21.0	8.7

Note: 40 cycles of signal operation were observed for each survey hour.

tober 16. Only six saturated cycles were recorded when almost half the observed cycles experienced full-block queues of stopped vehicles.

Lane delineation of Raymond Boulevard was the first part of the experiment implemented. The revised markings were placed by necessity on September 1, 1968; as a result, the Labor Day holiday, September 2, 1968, intervened between the "before" and "after" measurements. A statistical analysis of the survey data in data groups 1 and 2 (Table G-27), using the analysis of variance (ANOVA) technique, indicated the following, as given in Table G-30:

1. The number of eastbound vehicles through per cycle increased at Mulberry Street and at McCarter Highway in the AM period.
2. The number of eastbound vehicles stopped per cycle increased at Mulberry Street and at McCarter Highway in the AM period.
3. The number of westbound vehicles stopped per cycle decreased at McCarter Highway and increased at Mulberry Street in the PM period.

An additional analysis was performed on this same survey data using the chi square tecehnique to compare "before" and "after" frequency of occurrence of saturated cycles. The results of this analysis, also given in Table G-30, indicated:

1. The number of saturated cycles at McCarter Highway increased for eastbound traffic in the AM period and for westbound traffic in the PM period.

The surveys made in October at the Raymond Boulevard intersection with McCarter Highway (Eastbound saturated cycles, Table G-30) were planned to serve a dual purpose. The data for all approaches were intended as "before" data for the signal revision stage. The data for the east-west approaches also were intended as a second set of "after" data for the lane delineation of Raymond Boulevard, because conditions on this street did not change between these surveys. The means for these surveys are also given in Table G-30 for ease of comparison with both the "before" data and the first set of "after" data.

The increase in the mean number of eastbound vehicles through in the AM period remained relatively constant for these latter "after" data. This could be interpreted as reflecting a real increase in capacity due to the revised lane delineation, or the increase may have been all or partly due to seasonal increases in traffic volumes occurring after the Labor Day holiday.

That the increases in the mean number of vehicles stopped and saturated cycles observed for eastbound traffic in the AM period in the first "after" data were no longer evident in these latter "after" data suggests that the time period required for motorists to become familiar with changes of this magnitude is longer than the week allowed by the first set of measurements. This is shown by Figure G-57, which compares cycle-by-cycle operations for the same AM period on the same aproach for all three experimental conditions.

The increase in the mean number of vehicles stopped and saturated cycles observed, for westbound PM traffic in the latter "after" data, may have been due to the frequent parking violations at the north curb of the east approach to the McCarter Highway intersection. These violations, together with an apparently greater traffic demand, undoubtedly influenced the traffic operations that were observed.

The effects of designating a lane for exclusive left-turn use cannot be totally evaluated from the mean number of westbound vehicles per cycle observed making this movement for the various experimental conditions. While an increase in volume is evident, the absence of overhead lane use signs and the presence of illegal parking in the north curb lane resulted in a considerable number of through vehicles using the left-turn lane. On several occasions in the "after" conditions, almost half of the vehicles using this lane were observed to proceed straight ahead. Even when enforcement was provided, these violations continued at a high rate. Some were deliberate; but a number were inadvertent, owing to the pavement markings being hidden under stopped vehicles.

Lane delineation of McCarter Highway, the second part of the marking improvement, resulted in significant changes in driver operating habits and intersection efficiency. This is shown by Figure G-58, which compares "before" and "after" cycle-by-cycle operations on the north approach.

With reference to Table G-29, the mean number of vehicles through increased for both northbound and southbound traffic movements in both AM and PM periods, and the mean number of southbound vehicles stopped per cycle decreased from 45.2 to 28.8.

The most evident change in this stage of the experiment was the increase in the mean number of southbound vehicles through per cycle during the PM period from 32.5 to 36.8, or an increase of 13.2 percent. As Table G-29 indicates, the left-turning movement alone accounted for 3.4 of the 4.3 vehicles-per-cycle increase. A similar change can be noted in the mean numbers given for the AM period.

The increase in total volume suggests that a number of motorists had changed their normal routing to take advantage of the left-turn lane and better organized traffic flow. The left-turn lane, in effect, added a lane to both the north and south approaches. The efficiency of this auxiliary left-turn lane was further demonstrated by the fact that nearly all left turns were made from it, as compared to the two-lane operation frequently occurring in the "before" conditions. As a result of this situation, the left turn in the "after" condition frequently was unable to clear all waiting vehicles when other lanes on the north approach were cleared. This is not reflected in the survey totals of saturated cycles, which include the total of all vehicles on the approach. The lack of overhead lane use signs on this approach did not result in many through vehicles being inadvertently trapped, as was the situation on the east approach to this intersection, primarily due to the high demand for the left turn at the north approach.

The signal revision stage of the improvement was intended to permit and encourage southbound, second-lane left turns by the use of a "lagging" protected signal phase and overhead lane use signs. As previously mentioned, overhead signs were not provided by the city; and the potential benefits of the intended operations were not

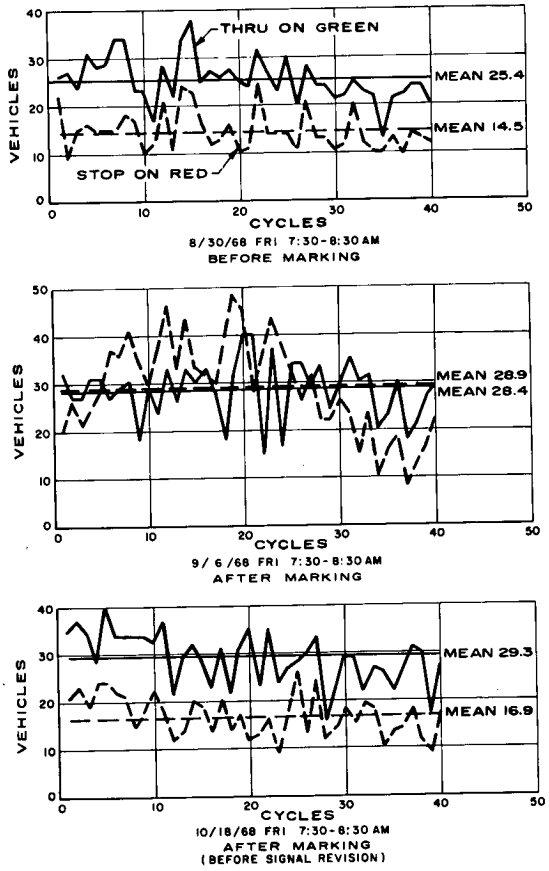


Figure G-57. Vehicles through and vehicles stopped per cycle, eastbound Raymond Boulevard at McCarter Highway.

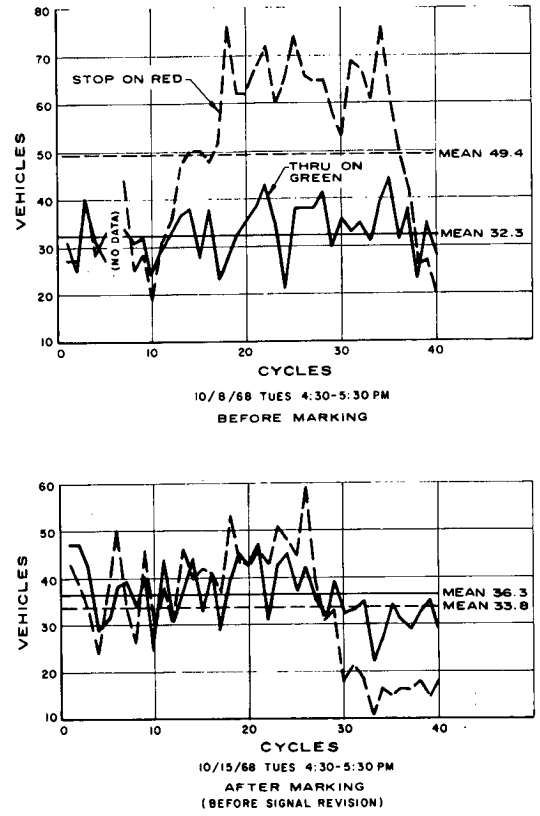


Figure G-58. Vehicles through and vehicles stopped per cycle, southbound McCarter Highway at Raymond Boulevard.

TABLE G-30

RAYMOND BOULEVARD LANE DELINEATION

INTERSECTION	MEAN "BEFORE"	MEAN "AFTER" 1	DIFFERENCE	CHANGE (%)	MEAN "AFTER" 2	DIFFERENCE (FROM "BEFORE")	CHANGE (FROM "BEFORE") (%)
			(*a*) EB vehicles through per cycle, 7:30–8:30 AM				
Mulberry St.	22.3	25.0	+2.7	+12.1	—	—	—
McCarter Hwy.	26.1	29.2	+3.1	+11.9	29.4	+3.3	+12.6
			(*b*) EB vehicles stopped per cycle, 7:30–8:30 AM				
Mulberry St.	7.7	13.7	+6.0	+77.9	—	—	—
McCarter Hwy.	14.8	28.0	+13.2	+89.2	18.2	+3.4	+23.0
			(*c*) WB vehicles stopped per cycle, 4:30–5:30 PM				
Mulberry St.	10.7	13.5	+2.8	+26.2	—	—	—
McCarter Hwy.	26.1	17.2	−8.9	−34.1	25.8	−0.3	−1.1
			(*d*) EB saturated cycles, 7:30–8:30 AM				
Mulberry St.	0/78	6/79	NS	+7.6	—	—	—
McCarter Hwy.	0/78	32/80	Sig.	+40.0	6/160	—	+3.8
			(*e*) WB saturated cycles, 4:30–5:30 PM				
Mulberry St.	0/80	0/80	NS	0	—	—	—
McCarter Hwy.	12/79	1/68	Sig.	−13.7	16/160	—	−5.2

NS = not significant.

realized, as the single-lane, left-turn movement operation continued in this stage of the experiment.

No changes of any magnitude in the numbers of vehicles through per cycle resulted from the revisions in signal operation, as shown in Figure G-48 for the "before" conditions and in Figure G-56 for the "after" conditions. The south approach, however, experienced increases in both the mean number of vehicles stopped per cycle and the number of saturated cycles occurring during both the AM and PM periods. Both of these latter effects can be attributed to the 49.2-percent lesser amount of cycle time assigned to this approach by the signal revisions, from 56.5 to 28.7 sec. The slight upgrade on this approach magnified the effects of this reduction in green time, due to the high percentage of trucks in the traffic stream. This percentage was about 25 in the AM period and about 13 in the PM period. These effects were not unexpected, as the primary emphasis of the signal revision was to expedite movement from the north approach.

The revised traffic phasing resulted in some unexpected problems due to pedestrians in the south crosswalk leaving the curbs before they received the WALK signal indication. These pedestrians would begin their crossings as soon as northbound traffic stopped, hazardously interfering with the southbound through traffic flow and delaying the east-to-south left-turn movement during the following phase. The fact that, generally, pedestrian signal indications of the study area did not provide an adequate clearance interval, together with the fact that many existing pedestrian signals were inoperative, undoubtedly contributed to the disregard displayed for such controls.

These problems, together with a continuing disrespect for traffic controls on the part of motorists, resulted in survey data that did not indicate a significant improvement. Observation, however, indicated that traffic generally appeared to flow more smoothly, with less congestion and fewer signal violations than occurred previously.

Comparison of the cycle-by-cycle plots of vehicles stopped and through for the "before" conditions ("after" marking), as shown in Figure G-57 for the west approach and in Figure G-58 for the north approach, with similar plots for the "after" conditions ("after" signal revisions) shown in Figures G-59 and G-60 for these same approaches, respectively, may be interpreted as showing slightly less turbulence with the exception of several signal cycles.

The beneficial effect of providing a flashing DON'T WALK signal indication for the pedestrian clearance interval was very apparent. In the "before" condition, pedestrians were observed to cross until the conflicting traffic began moving. Most pedestrians obeyed the flashing DON'T WALK indication; others were observed to hurry to complete their crossing when the flashing operation began.

In an effort to identify any changes in quality of flow resulting from the revised signal phasing, a number of "floating-car" speed and delay runs were made. The intersections of Park Place and Raymond Plaza East were used as the terminal points for the east-west Raymond Boulevard runs, a total distance of 2,410 ft (0.46 mile). Eastbound runs were not made in the PM period, because Raymond Boulevard operates one-way westbound from Mulberry

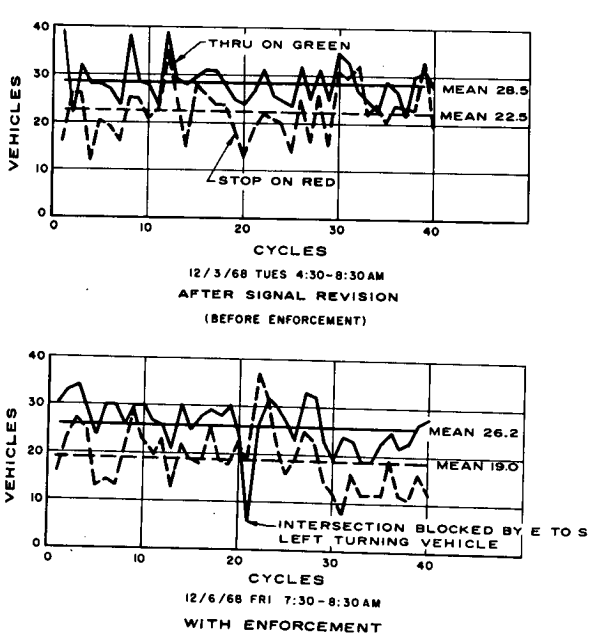


Figure G-59. Vehicles through and vehicles stopped per cycle, eastbound Raymond Boulevard at McCarter Highway.

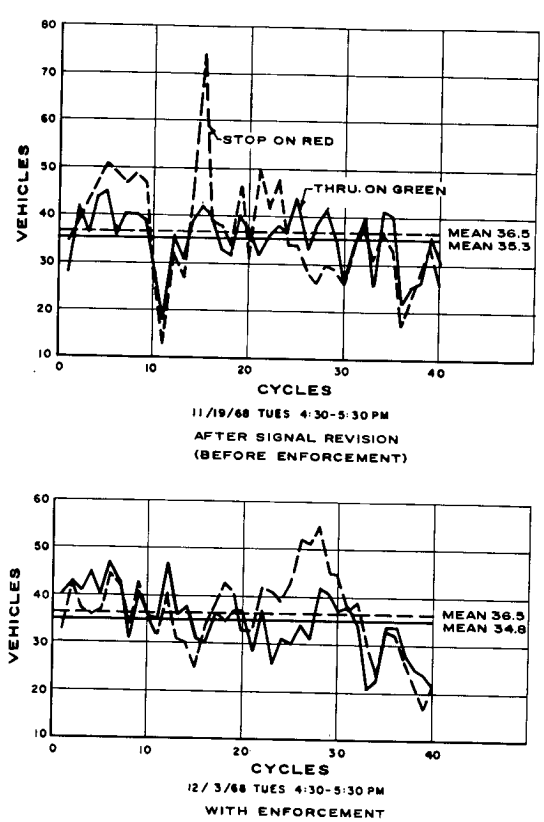


Figure G-60. Vehicles through and vehicles stopped per cycle, southbound McCarter Highway at Raymond Boulevard.

TABLE G-31

NUMBER OF SPEED AND DELAY RUNS, McCARTER HIGHWAY AT RAYMOND BOULEVARD

	SPEED AND DELAY RUNS							
	EB		WB		NB		SB	
CONDITION	AM	PM	AM	PM	AM	PM	AM	PM
"Before" signal revision (after marking)	8	—	7	8	6	7	7	8
"After" signal revision (before enforcement)	21	—	21	18	11	8	10	8
With enforcement	—	—	—	9[a]	—	8	—	8

[a] In rain.

TABLE G-32

MEANS OF AM PERIOD SPEED AND DELAY RUNS

DIRECTION	CONDITION	TOTAL TRIP TIME (SEC)	AVERAGE SPEED (MPH)	TOTAL STOP TIME (SEC)	NO. OF STOPS	RUNS STOPPED AT RAYMOND AND MCCARTER (%)
EB	"Before" signal revision	252	6.5	130	4.2	63
	"After signal revision	151	10.9	67	1.9	86
	Net change	−101	+4.4	−63	−2.3	—
	Percent change	−40.1	+40.4	−48.5	−54.8	+23
WB	"Before" signal revision	166	9.9	71	2.7	0
	"After" signal revision	137	12.0	62	1.9	0
	Net change	−29	+2.1	−9	−0.8	—
	Percent change	−17.5	+21.2	−12.7	−29.6	0
NB	"Before" signal revision	110	14.0	36	1.0	50
	"After" signal revision	113	13.6	41	1.2	100
	Net change	+3	−0.4	+5	+0.2	—
	Percent change	+2.7	−2.9	+12.2	+20.0	+50
SB	"Before" signal revision	120	12.8	52	1.4	57
	"After" signal revision	108	14.3	36	1.3	60
	Net change	−12	+1.5	−16	−0.1	—
	Percent change	−10.0	+11.7	−30.8	−7.1	+3

Street between 4:00 and 6:00 PM (Fig. G-47). The intersections of Market Street and Saybrook Place were used as the terminal points for the north-south McCarter Highway runs, a total distance of 2,260 ft (0.43 mile).

Table G-31 gives the total number of runs made, by direction and time period. Table G-32 gives the mean values for all AM period runs, and Table G-33 gives identical information for the PM period.

A comparison of the mean values for the conditions of "before" and "after" the signal revision indicated:

1. For the AM period, eastbound, westbound, and southbound traffic experienced decreases in total trip time, total stop time, and number of stops, with resulting increases in average speed. Quantitatively, the predominant eastbound traffic indicated the greatest improvement, with decreases of

TABLE G-33

MEANS OF PM PERIOD SPEED AND DELAY RUNS

DIRECTION	CONDITION	TOTAL TRIP TIME (SEC)	AVERAGE SPEED (MPH)	TOTAL STOP TIME (SEC)	NO. OF STOPS	RUNS STOPPED AT RAYMOND AND MCCARTER (%)
WB	"Before" signal revision	302	5.5	159	4.5	38
	"After" signal revision	252	6.5	127	4.1	67
	Net change	−50	+1.0	−32	−0.4	—
	Percent change	−16.6	+18.2	−20.1	−8.9	+29
	With enforcement [a]	261	6.3	140	4.3	75
	Net change (from "after")	+9	−0.2	+13	+0.2	—
	Percent change	+3.6	−3.1	+10.2	−4.9	+8
NB	"Before" signal revision	130	11.9	47	1.6	14
	"After" signal revision	239	6.5	143	2.6	88
	Net change	+109	−5.4	+96	+1.0	—
	Percent change	+83.8	−45.4	+204.3	+62.5	+74
	With enforcement	196	7.8	95	2.5	100
	Net change (from "after")	−43	+1.3	−48	−0.1	—
	Percent change	−18.0	+20.0	−33.6	−3.8	+12
SB	"Before" signal revision	140	11.0	62	1.7	63
	"After" signal revision	183	8.4	105	1.6	75
	Net change	+43	−2.6	+43	−0.1	—
	Percent change	+30.7	−23.6	+69.4	−5.9	+12
	With enforcement	122	12.6	53	1.1	100
	Net change (from "after")	−61	+4.2	−52	−0.5	—
	Percent change	−33.3	+50.0	−49.5	−31.3	+25

[a] In rain.

101 sec in total trip time, 63 sec in total stop time, and 2.3 fewer stops per run, with a resulting increase in average speed of 4.4 mph. Northbound traffic, conversely, experienced the reverse conditions.

2. For the PM period, the predominant westbound flow experienced decreases in total trip time of 50 sec, in total stop time of 32 sec, and in the number of stops of 0.4 per run, with a resulting increase in average speed of 2.1 mph. Northbound and southbound traffic experienced increases in total trip time and total stop time, with resultant increases in average speeds. Northbound traffic also experienced an increase in the number of stops per run. (No eastbound runs were made.)

The second part of the signal revision attempted to deal with the problem of violations. Police enforcement was used to ensure motorist and pedestrian observance of the signal control.

The survey data for vehicles stopped and vehicles through yielded no indications of significant change in this stage of the experiment. Insufficient data were obtained for detailed analysis owing to the reasons given in Table G-27.

Speed and delay runs were made also in both survey periods for this part of the experiment; but, as with other survey data, only a portion of the data obtained could be analyzed. Summaries of these data are also given in Table G-33.

A comparison of the mean values for the conditions "without enforcement" ("after" signal revision) and "with enforcement," as given in Table G-33, indicated that for the PM period, decreases in total trip time, total stop time, and number of stops were experienced by both northbound and southbound traffic, with resultant increases in average speeds. In addition, the values for westbound traffic recorded with rain falling were not greatly different than for those without enforcement on dry pavement.

These results lead to the conclusion that the beneficial effects on quality of flow by motorist and pedestrian obedience to traffic signal control are important and mea-

surable and contribute significantly to efficiency of operations and safety.

After enforcement was relinquished, pedestrians and motorists were observed to continue obedience to the traffic signal controls for a period of a few days, before reverting to the disrespect evident in the "before" conditions, illustrating the residual effects of education by enforcement. This effect was most evident among pedestrians.

It is interesting to compare the similar cycle-by-cycle operations on the north approach for the PM period under favorable weather conditions (Fig. G-60) with those shown in Figure G-61 for unfavorable weather conditions. The obviously less efficient operations shown in Figure G-61 were due mostly to approaching traffic reverting to the three-lane operation that existed prior to any experimentation, as previously described. This was a result of the revised lane delineation being obscured by water film. The advantages to be gained through the use of overhead lane use signs were pointedly demonstrated by this experience.

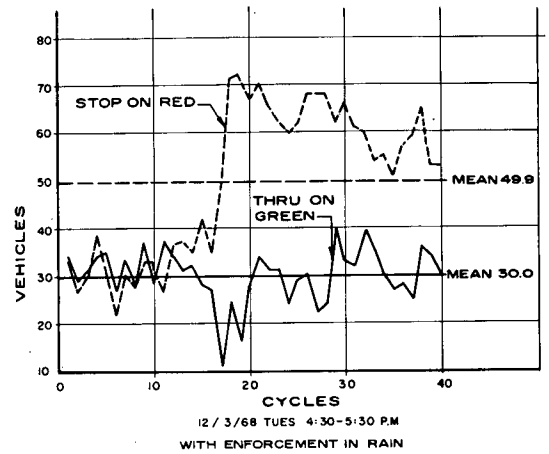


Figure G-61. Vehicles through and vehicles stopped per cycle, unfavorable weather conditions, southbound McCarter Highway at Raymond Boulevard.

Conclusions

The conclusions determined by analysis of survey data and observations of operations are:

1. Lane delineation can increase intersection capacity and reduce vehicular conflicts and accompanying congestion.
2. Mandatory lane use defined by pavement delineation without overhead signing failed to achieve maximum effectiveness during periods of peak traffic or inclement weather.
3. Revisions to signal operations to accommodate traffic desires can be effective in reducing conflicts and may result in smoother and safer traffic operations, depending on motorist and pedestrian obedience to such operations.
4. The level of maintenance provided, together with the reasonableness of signal interval timing, influences the degree of motorist and pedestrian acceptance of traffic signal control.
5. Implementation of engineered controls without proper education of the public influences the degree of public acceptance and observance.
6. Continual enforcement is necessary to achieve a public attitude of respect for traffic regulations.

CURB LANE CONTROLS

Parking Revisions at Central Avenue and High Street—Experiment A68

Experiment A68 examines the effects produced by providing additional lanes on the legs of this intersection through the elimination of parking and standing and a retiming of the traffic signals to compensate for the additional capacities resulting from these parking adjustments.

Experimental Area

The area of Experiment A68 is the intersection of Central Avenue, High Street, and Sussex Avenue (Figs. G-62 and G-63). Central Avenue is a major two-way arterial street extending westward from the CBD to the adjacent community of East Orange. High Street is a two-way arterial street extending in a north-south direction along the western edge of the downtown area of Newark. Sussex Avenue is a collector street extending northwestward from the intersection. It is a one-way street for one block west of the intersection and then becomes two-way.

Central Avenue has a pavement width of 40 ft, east of the intersection, that is divided into equal lanes by a center line extending for a distance of aproximately 70 ft from the intersection. West of the intersection the pavement width is 48 ft and is marked with a center line and two lane lines. These lanes vary in width to eliminate an offset of the center lines at the intersection. High Street is 40 ft wide, and the roadway is divided by a center line that extends a distance of approximately 50 ft from the intersection in both directions. Three bus stops, a cab stand, and a loading zone are located in the immediate vicinity of the intersection (Fig. G-63).

The flow of traffic through the intersection from 4:30 to 5:30 PM on a typical day (Tuesday, September 17, 1968) is shown in Figure G-64.

The traffic signals are under the jurisdiction of the city of Newark. The controller is pretimed, with two 90-sec dials that maintain a simultaneous offset relationship with the signalized intersections to the north, south, and west. The other signals on Central Avenue, east of High Street, are part of the city of Newark PR system, to which there is no constant offset relationship. Dial 2 is in operation from 4:00 to 6:00 PM on weekdays, and Dial 1 is in operation at all other times. During evening peak hours the signal operates with 49 percent of the cycle allocated to High Street, with a northbound advance of 2 percent (Fig. G-65).

Design of Experiment

East of High Street, Central Avenue is 40 ft wide, originally marked for two lanes. An additional lane was provided by eliminating parking for the entire length of the block, approximately a distance of 725 ft. On High Street south

Figure G-62. Location map, Experiment A68.

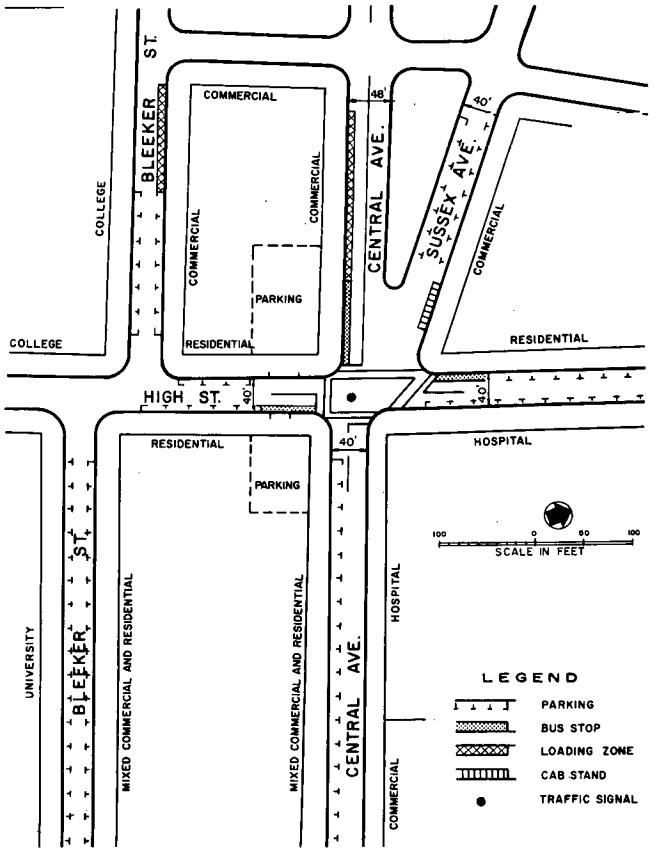


Figure G-63. Vicinity map.

of Central Avenue, where the 40-ft width was originally marked for two lanes, an additional lane was provided on the east side by eliminating parking for a distance of 250 ft. "Before" and "after" conditions for these approaches to the intersection are shown in Figures G-66 and G-67.

"Before" and "after" settings of the traffic signals at this intersection are shown in Figure G-65.

The "before" measurements were scheduled for the evening peak period of 4:30 to 5:30 PM on Tuesday, Wednesday, and Thursday, September 17, 18, and 19. The "after" measurements were scheduled for the 4:30 to 5:30 PM evening peak period of Tuesday, Wednesday, Thursday, September 24, 25, and 26. Observations made on Tuesday, September 24, showed that the intersection operated well, that traffic had adapted to the new conditions, and that "after" surveys could begin.

Analysis and Conclusions

The results of Experiment A68 show that substantial improvements in traffic flow at this intersection were realized as a result of:

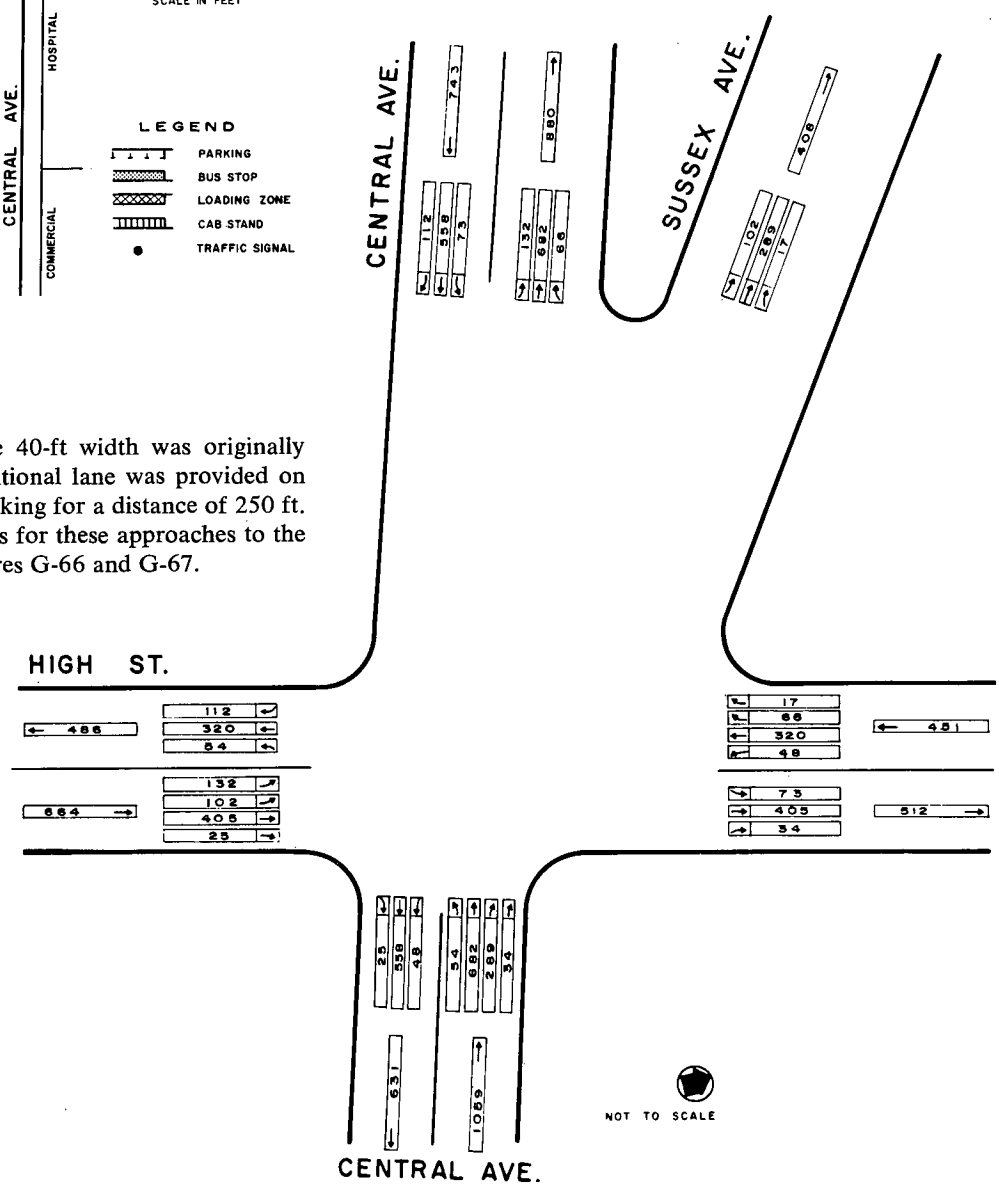
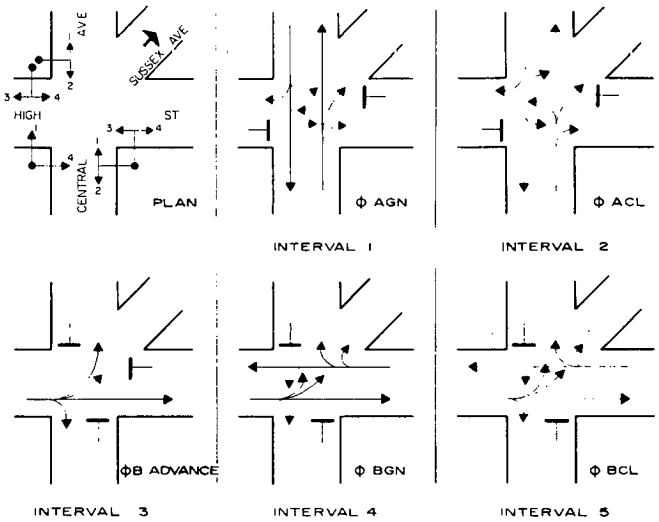


Figure G-64. Traffic flow, PM peak hour (4:30 to 5:30 PM).



STREET	SIGNAL NO.	INTERVAL 1	2	3	4	5
CENTRAL AVE EB	1	G	A	R	R	R
CENTRAL AVE WB	2	G	A	R	R	R
HIGH ST. NB	3	R	R	G	G	A
HIGH ST. SB	4	R	R	R	G	A
"BEFORE"- TIME IN %		44	5	2	44	5
"AFTER"- TIME IN %		44	5	12	34	5

CYCLE = 90 SECONDS

Figure G-65. Traffic signal sequence and timing.

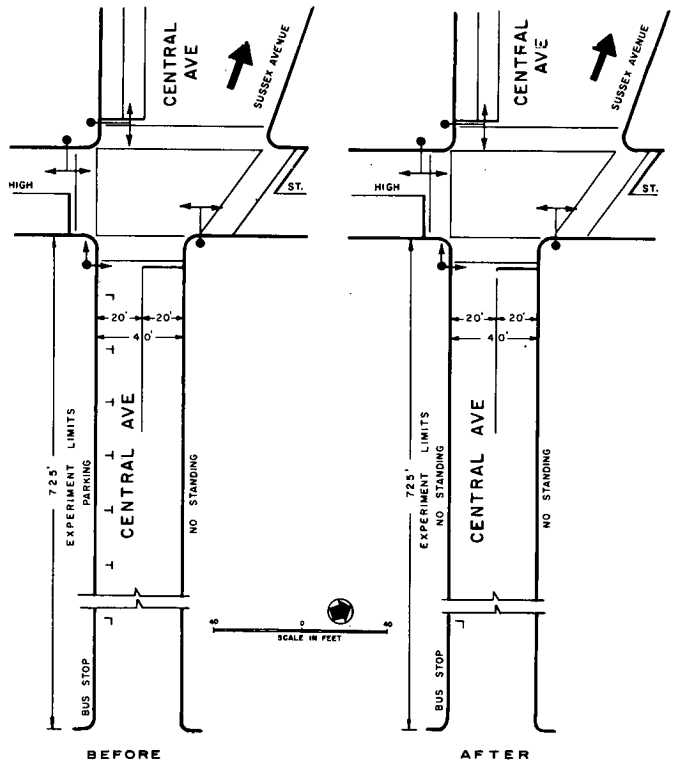


Figure G-66. Central Avenue traffic controls.

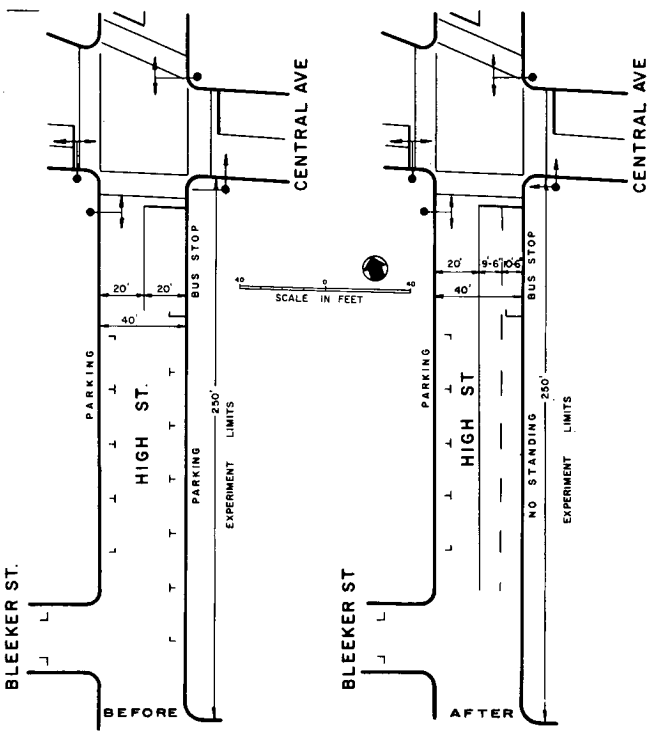


Figure G-67. High Street traffic controls.

1. Separation of left-turning traffic and other traffic on a single-lane approach to a traffic signal by provision of a left-turn lane.
2. Provision of an equal number of entrance and exit lanes at a signalized intersection.
3. Modification of the signal phasing so that there is correct apportionment of green time.

An additional lane was provided on the northbound High Street approach, south of the intersection, and the green time was reduced from the "before" condition. Despite the reduction in green time, there was a significant improvement in traffic flow northbound. There was a 40-percent reduction in the number of vehicles stopped during the red interval per cycle and a 42-percent reduction in the number of saturated cycles. A comparison of the length of queues for the "before" and "after" conditions showed a considerable reduction in the queue length for the "after" condition.

Traffic flow on High Street southbound (off-peak direction) was not improved in the "after" condition owing to the decrease in green time and the reduction in the number of gaps for left-turning traffic. This resulted in a 36-percent increase in the number of vehicles stopped during the red interval per cycle and a 20-percent increase in the number of saturated cycles.

Traffic flow on Central Avenue eastbound was impeded in the "before" study by the need for traffic to merge into one lane while crossing the intersection. After an additional lane had been provided on the east side of the intersection and, after reapportionment of the green time, the number of vehicles stopped on red and the number of

saturated cycles decreased significantly. Saturated cycles were reduced by 90 percent and vehicles stopped were reduced by 21 percent.

Traffic flow on Central Avenue westbound was significantly improved, the number of vehicles stopped being reduced by 26 percent and the number of saturated cycles being reduced by 81 percent in the "after" period as a result of the increased green time.

Truck Loading Restrictions, University Avenue Between Orange Street and James Street—Experiment C123

Experiment C123 studies the effects of parking restrictions on truck loading activity and traffic characteristics on University Avenue in Newark. This north-south collector street in the center of Newark provides direct access to many of the parking facilities in the downtown area (Fig. G-68). Between Orange Street and James Street traffic flows are impeded during the morning peak hour by truck loading activity at the wholesale meat establishments on both sides of the street.

The puropse of this experiment is to measure the changes in travel time caused by the elimination of a truck loading zone along the west curb face of University Avenue between Orange Street and James Street.

Experimental Area

University Avenue provides for one-way movement southbound from Broad Street to Court Street. Pavement width is 30 ft in the area of the experiment, and land use is commercial on both sides of the street. Parking is prohibited during the morning peak hour, except at loading zones.

Traffic signals at Orange Street and James Street have a two-phase pretimed controller. Timing and offsets are shown in Figure G-69.

Experimental Design

The existing loading regulations on the west side of University Avenue between Orange Street and James Street permit truck loading from 7:00 to 9:00 AM. This regulation was changed and replaced by a regulation stating NO STANDING during those same hours (Fig. G-70). Truck loading commercial interests along the street from Orange Street to James Street were advised of the change prior to the date of implementation on June 2, 1969. The surveillance system for this experiment included vehicular counts, speed and delay runs, and a parking study. The number of vehicles through per cycle and the number of vehicles stopped on red per cycle were counted on University Avenue at the intersections of Orange Street and James Street. Speed and delay runs were made between Lackawanna Avenue and Central Avenue, a distance of 1,700 ft. A parking use survey was conducted to determine the turnover rate during the "before" and "after" conditions.

Measurements for the "before" condition were made on May 13, 15, and 16, 1969; the "after" measurements were made on June 10, 12, and 13, 1969.

Analysis

Travel time, delay time, and number of stops were analyzed to detect differences between "before" and "after" conditions. The data were investigated for the entire length of the run from Lackawanna Avenue to Central Avenue, a distance of approximately 1,700 ft. This analysis indicated that minor reductions were made in both travel time and the number of stops. No significant change was detected for the delay time. The results of this analysis are given in Table G-34, with the appropriate α level included.

A further investigation into the speed and delay runs was made by dividing the over-all run into the following three segments:

1. Segment 1—Lackawanna Avenue to Orange Street.
2. Segment 2—Orange Street to James Street.
3. Segment 3—James Street to Central Avenue.

The analysis of the individual segments indicated no significant changes for any of the variables in Segments 2 or 3. Analysis of Segment 1, however, indicated significant improvements of a 17.3-percent decrease in travel time ($\alpha = 0.01$), a 23.2-percent decrease in delay time ($\alpha = 0.025$), and a 20.6-percent decrease in the number of stops ($\alpha = 0.005$).

Table G-34 indicates that the levels of significance are much stronger for Segment 1 than for the over-all run. The reason for this condition is that, although the "before" and "after" means are not significantly different in Segments 2

TABLE G-34

SUMMARY OF SPEED AND DELAY ANALYSIS

	SEGMENT 1—LACKAWANNA AVE. TO ORANGE ST.				OVER-ALL LENGTH—LACKAWANNA AVE. TO CENTRAL AVE.			
	MEAN VALUE				MEAN VALUE			
VARIABLE	"BEFORE"	"AFTER"	DIFFERENCE (%)	SIG. @ α LEVEL SPECIFIED	"BEFORE"	"AFTER"	DIFFERENCE (%)	SIG. @ α LEVEL SPECIFIED
Travel time (sec)	54.8	45.3	−17.3	0.01	154.7	141.8	−8.3	0.10
Delay time (sec)	37.0	28.4	−23.2	0.025	73.7	68.1	−7.6	NS [a]
No. of stops	1.1	0.9	−18.2	0.005	8.4	2.1	−12.5	0.05

[a] Difference not significant for $\alpha \leq 0.10$.

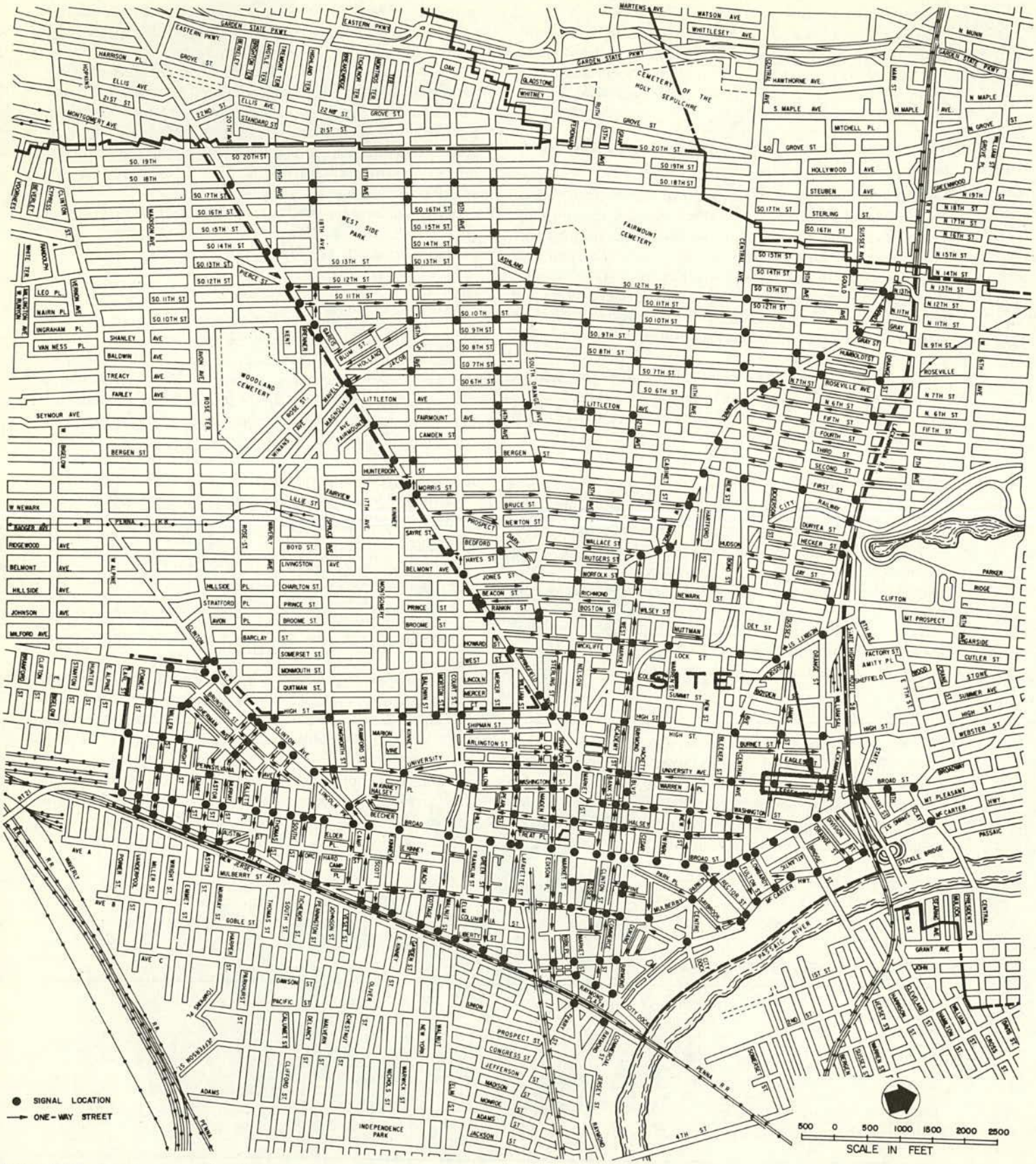
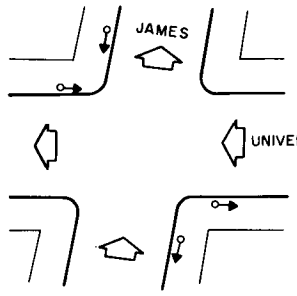


Figure G-68. Location map, Experiment C123.

STREET	INDICATION			
UNIVERSITY - ØA	G	A	R	R
JAMES - ØB	R	R	G	A
TIME - %	55	5	35	5
CYCLE - SEC.	90			



SIGNAL ON PEDESTAL
SIGNAL ON MAST ARM

STREET	INDICATION			
ORANGE ØA	G	A	R	R
UNIVERSITY	R	R	G	A
TIME - %	55	5	35	5
CYCLE - SEC.	90			

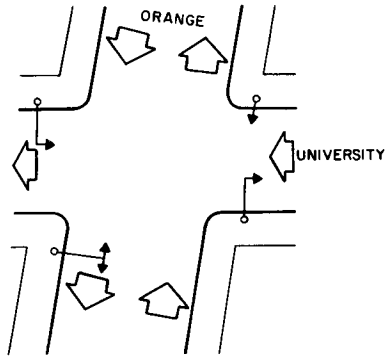


Figure G-69. Traffic signals, sequence and timing.

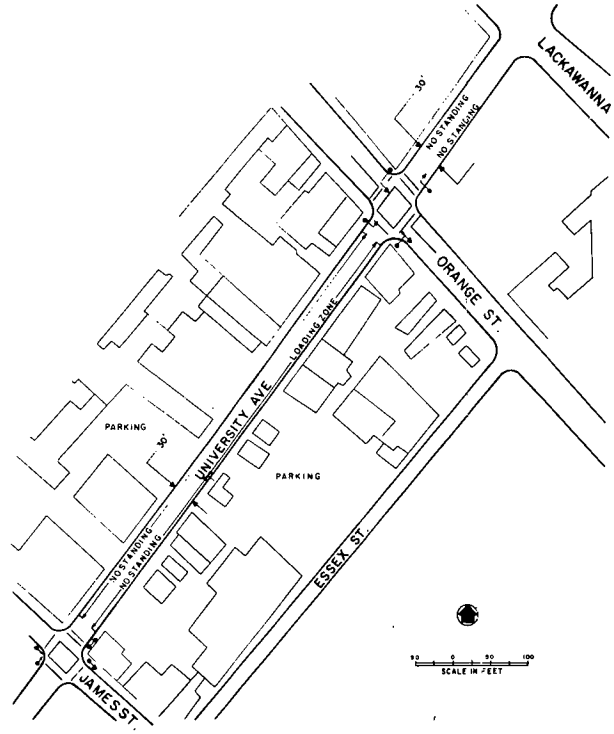


Figure G-70. Traffic controls, "after," AM peak period (7 to 9 AM).

and 3, their variability is increased, thereby requiring a larger percentage of difference at the same level as Segment 1.

Analysis of the parking turnover rate on Segment 2 indicated a 50.9-percent increase between the "before" and "after" conditions.

Conclusions

Based on the foregoing analysis, it is concluded that:

1. Over-all travel time and number of stops were significantly decreased by 8.3 percent and 12.5 percent, respectively.
2. Travel time, delay time, and number of stops were significantly reduced in Segment 1 by 17.3 percent, 23.2 percent, and 18.2 percent, respectively.

Parking Restrictions on Oak Street at Sixth Street—Experiment D2

Experiment D2 examines the effect of providing an additional travel lane by prohibiting parking on the northern side of the Oak Street approach to the intersection of Oak and Sixth Streets in Louisville. The additional lane was designed primarily for use as a left-turn lane for vehicles turning from Oak Street into Sixth Street so that the flow of through traffic on Oak Street would not be delayed by turning vehicles. Oak Street and Sixth Street are both one-way streets in the fringe area of the CBD (Fig. G-71). Oak Street is a part of the crosstown arterial system providing for eastbound trips from residential areas to industrial and commercial districts and is paired with westbound St. Catherine Street. Sixth Street provides for trips from residential areas to the CBD and is paired with southbound Seventh Street.

Experimental Area

Oak Street is 37 ft wide, with 1-hr metered parking on the north side, Monday through Saturday. On the south side of Oak Street stopping is not permitted from 7:00 to 9:00 AM and from 4:00 to 6:00 PM, Monday through Friday (Fig. G-72).

Sixth Street is 36 ft wide, and unmetered parking is permitted south of the intersection on the west side. On the east side of Sixth Street south of the intersection, stopping is not permitted from 7:00 to 9:00 AM and from 4:00 to 6:00 PM, Monday through Friday. North of the intersection on the west side, 1-hr metered parking is permitted between 7:00 AM and 6:00 PM, Monday through Saturday. North of the intersection on the east side, stopping is not permitted from 7:00 to 9:00 AM and from 4:00 to 6:00 PM, Monday through Friday; unmetered parking is permitted at all other times. There are near-side bus stops on both Oak Street and Sixth Street at their intersection.

The signal at the intersection is operated by a two-dial controller on a 60-sec cycle. Dial 1 is in operation from 2:15 to 5:45 PM, with 55 percent of the cycle allotted to Oak Street and 45 percent alloted to Sixth Street. Dial 2 is in operation from 5:45 PM to 2:15 PM the following

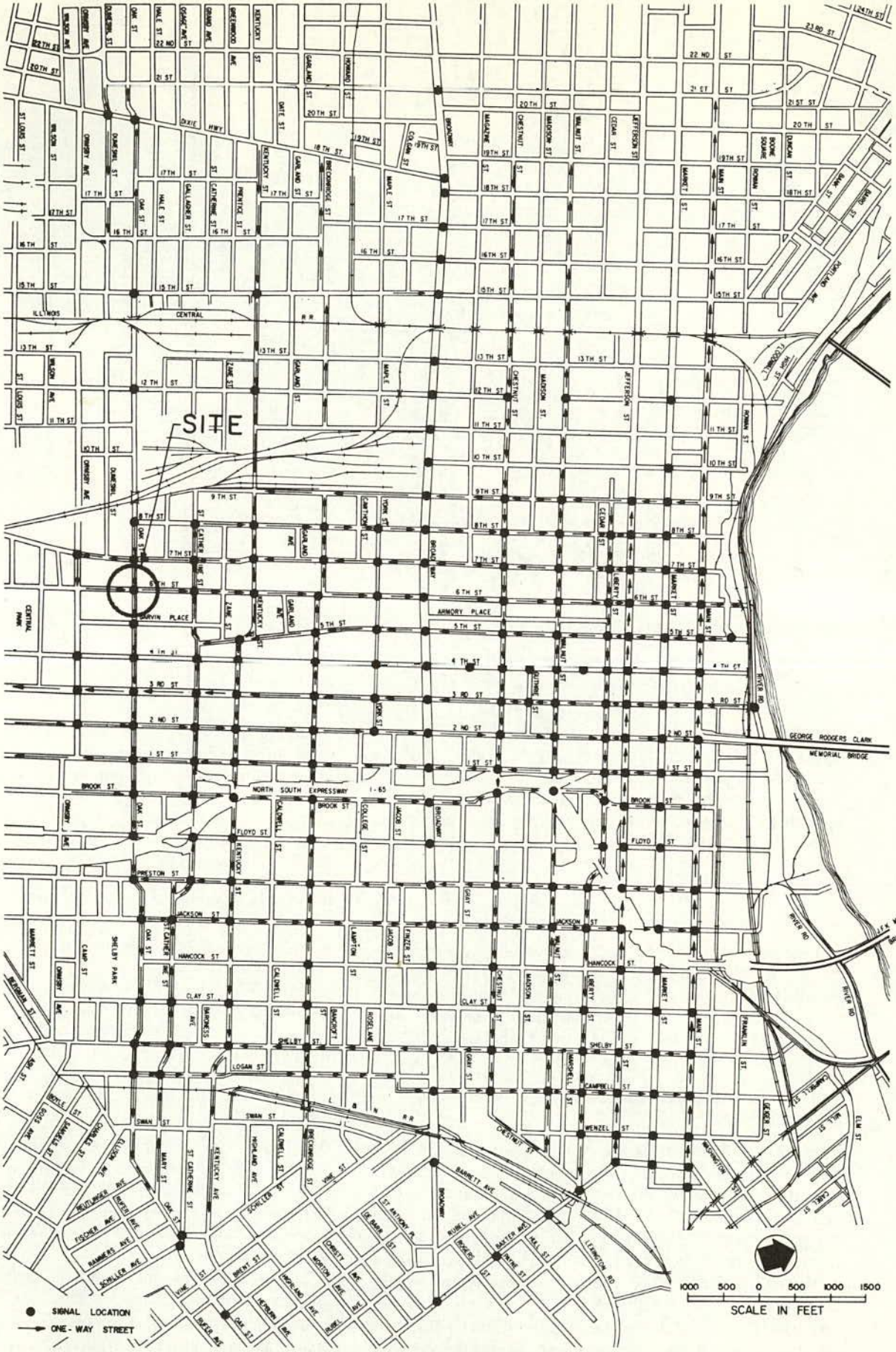


Figure G-71. Location map, Experiment D2.

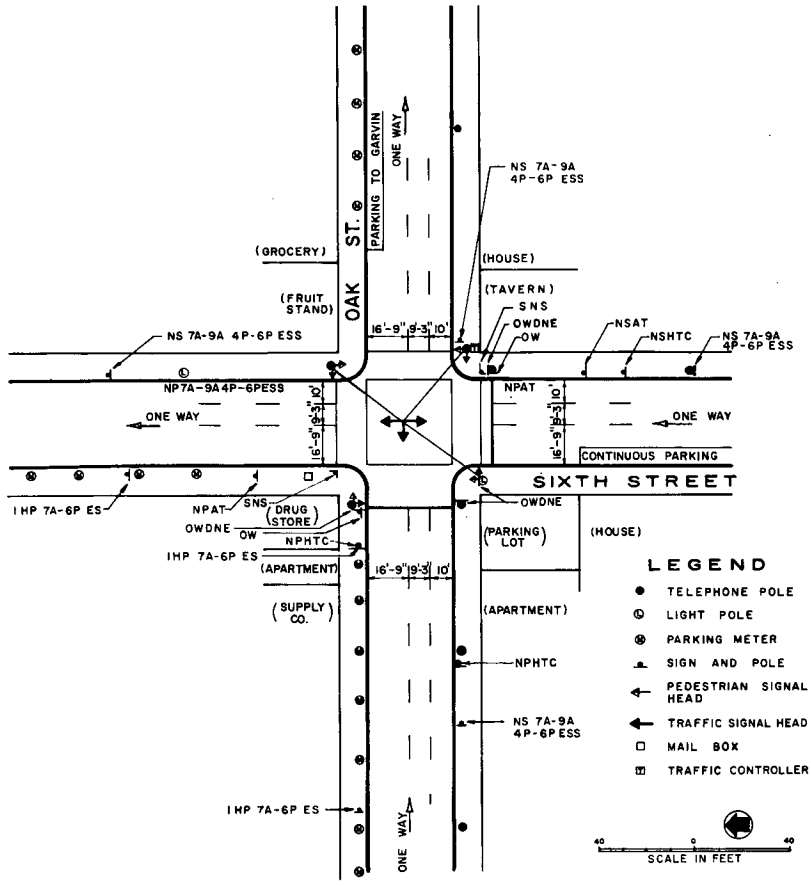


Figure G-72. Vicinity map.

afternoon, with 59 percent of the cycle allotted to Oak Street and 41 percent allotted to Sixth Street. The signal is tied into the progressive system on both Oak and Sixth Streets.

The signs, meter locations, and roadway dimensions are shown in Figure G-72. The peak-hour traffic volumes are given in Table G-36.

Design of Experiment

The effect of prohibiting parking and providing a separate left-turn lane was measured during the peak hours of 7:30 to 8:30 AM and 4:30 to 5:30 PM. Survey data were obtained on the following dates (1968):

	AM		PM	
DAY	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
Monday			7/8	7/15
Tuesday	7/9	7/16		7/16
Wednesday	7/10	7/17	7/10	

The city of Louisville, under strong pressure from merchants in the area, limited the "after" phase of the experiment to only three days, as shown previously; therefore, the time given to drivers to adjust to the new situation may not have been adequate. It was observed that few drivers actually used the restricted area as a left-turn lane. Many continued to turn from the same lane as before, probably due to long-established habit. The curb radius at the intersection is only 12 ft, and this also may have contributed to the drivers' reluctance to use the curb lane.

Characteristics of the traffic stream on the Oak Street approach that were measured were vehicles through per cycle, vehicles stopped on red per cycle, delay time on red per cycle, and delay time on red per vehicle through.

Field data were summarized by cycle in preparation for the First Level Analysis. The number of vehicles through and the number of vehicles stopped on red were recorded from the field sheets. The delay time on red per cycle was calculated from the number of arrivals during each 3-sec period multiplied by the average time these vehicles were stopped. The ratios of delay time on red per vehicle through were calculated for each cycle.

Analysis and Conclusions

During the morning peak hour no significant changes ($\alpha = 0.05$) in any of the measurements were detected. However, significant improvements were observed during the afternoon period. Delay time on red was reduced by 20 percent from a "before" mean value of 52.7 sec to an

"after" mean value of 42.4 sec. Delay time on red per vehicle through was reduced by 25 percent from 2.8 sec per vehicle to 2.1 sec per vehicle. The variability or spread of value for the latter measurement was also reduced. The "before" and "after" mean values for the variables are given in Table G-35.

With relation to the capacity of the Oak Street approach to the intersection, an improvement was found in the afternoon condition. The results of these computations are given in Table G-36.

In general, it may be stated that, in spite of the short duration of the period during which "after" measurements could be made, a significant change in traffic flow was realized as a result of the parking restrictions investigated in this experiment. Perhaps of more significance than the directly measured improvement in traffic flow is the merchant reaction to this experiment, which required its termination after only three days. It has been suggested that small public off-street parking lots in sensitive areas (such as the Oak Street location) could alleviate this situation. It has become apparent from this and other experiments involving parking restrictions that the parking situation is extremely sensitive with regard to public opinion and its consequent political implications. In view of the extreme sensitivity of this matter and the obvious fact that parked vehicles absorb much street capacity, the foregoing suggestion merits serious consideration.

TABLE G-35

OAK STREET EASTBOUND, "BEFORE" AND "AFTER" MARGINAL MEANS

VARIABLE	TIME PERIOD	MEAN "BEFORE"	MEAN "AFTER"
Delay time on red	AM	13.7	13.4
Delay time on red/vehicle through	AM	1.1	1.2
Vehicles stopped on red	AM	1.2	1.5
Vehicles through	AM	11.7	11.3
Delay time on red	PM	52.7	42.4
Delay time on red/vehicle through	PM	2.8	2.1
Vehicles stopped	PM	4.0	4.0
Vehicles through	PM	19.1	20.4

TABLE G-36

PEAK-HOUR TRAFFIC VOLUMES AND ROADWAY CAPACITY

CONDITION	TIME PERIOD	OAK ST. APPROACH VOLUME (VPH)	OAK ST. LEVEL OF SERVICE
"Before"	AM	729	A
	PM	1,209	C
"After"	AM	685	A
	PM	1,279	A

Parking Revision at Oak Street and Shelby Street—Experiment D15

One of the most common methods used to increase capacity on a congested roadway is to create an additional moving lane by restricting curbside parking during periods of peak traffic flow. Both Newark and Louisville have prohibited curbside parking along most of the major roadways during morning and evening rush hours. There remain, however, several opportunities within each city to test the effect of parking restrictions under varying conditions.

Experiment D15 consists of measuring and evaluating changes in traffic flow resulting from a revision of curbside parking restrictions at the approaches to the intersection of Oak Street and Shelby Street.

Experimental Area

The areas considered for experimentation were the eastbound and the southbound approaches of the Oak Street and Shelby Street intersection. This intersection is in the southeastern portion of the Louisville study area (Fig. G-73). Oak Street is one-way eastbound and, paired with St. Catherine Street, serves as the southernmost east-west arterial through the study area. Shelby Street is a one-way southbound arterial serving the eastern corridor of the downtown area.

The Oak Street approach is 36 ft wide, marked for three moving lanes, with parking permitted along the north curbside. The Shelby Street approach is 36 ft wide, marked for two lanes, with parking permitted along both curbsides (Fig. G-74). The intersection is signalized, with a two-dial controller operating on a 60-sec cycle; Oak Street receives 28 sec of green time, and Shelby Street, 26 sec.

Purpose and Scope

The purpose of Experiment D15 is to measure and evaluate the changes in traffic volumes, queue lengths, and vehicle delay time at the intersection of Oak and Shelby Streets resulting from a prohibition of curbside parking along each approach leg. The proposed restrictions are:

1. The prohibition of parking on the east side of Shelby Street between Oak Street and St. Catherine Street to provide an additional lane for left-turning vehicles.
2. The prohibition of parking on the north side of Oak Street for a length of 120 ft from Shelby Street to permit smoother flow through the "S" curve alignment of Oak Street.

Although both restrictions were implemented and measured simultaneously, the measurements were recorded separately for each approach leg to enable an evaluation of each individual change. The field surveys were conducted during morning (7:30 to 8:30 AM), evening (4:30 to 5:30 PM), and midday periods.

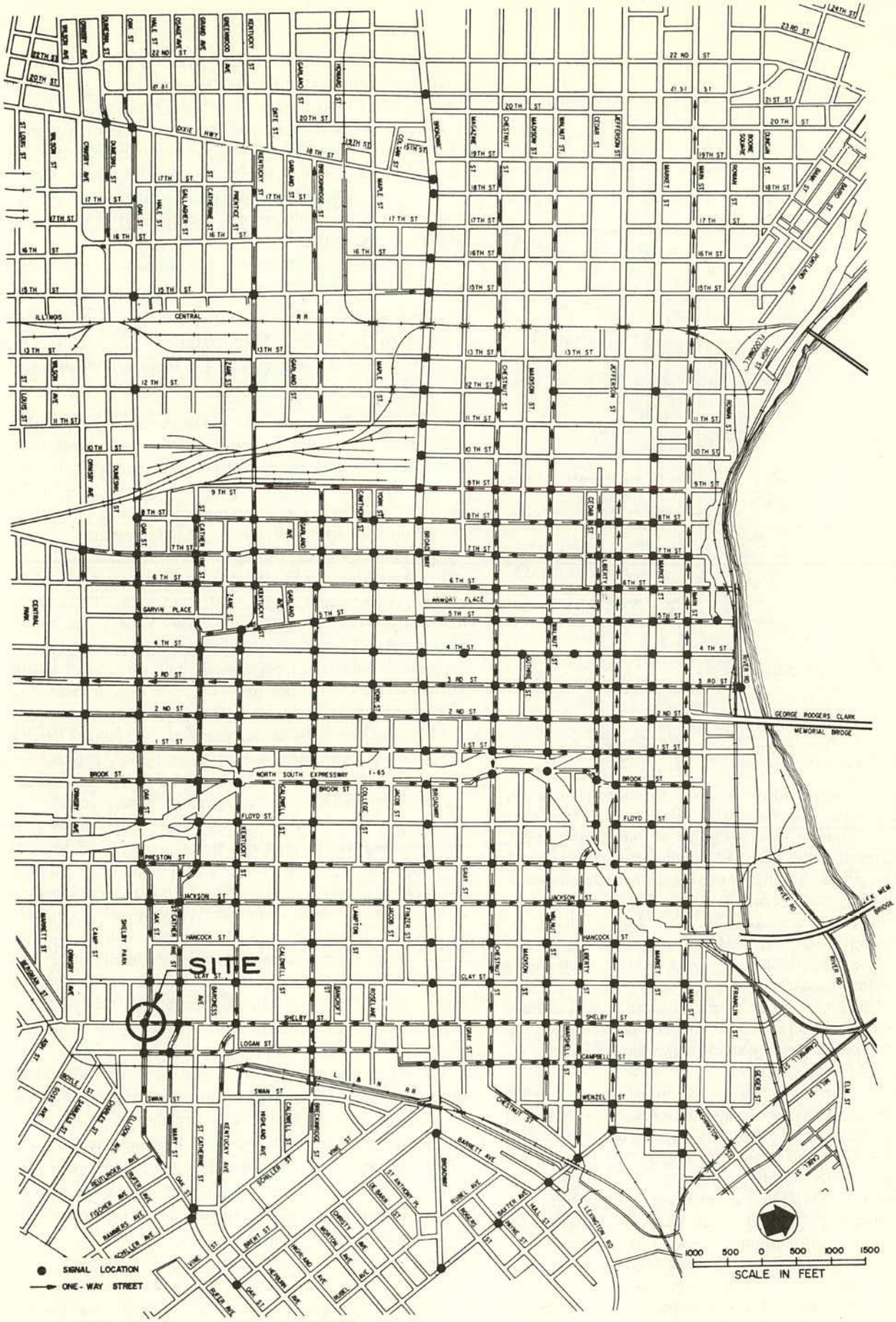


Figure G-73. Location map, Experiment D15.

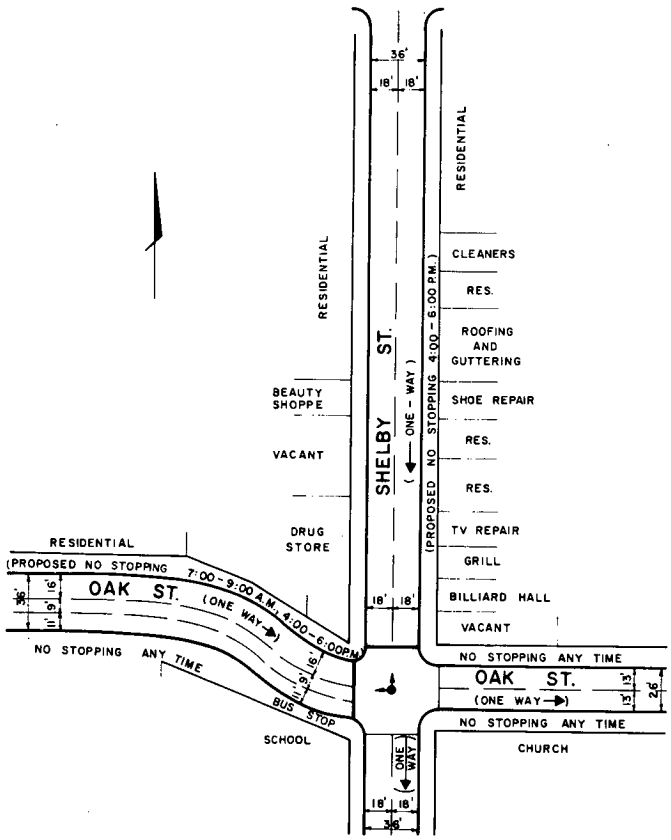


Figure G-74. Vicinity map.

Design of Experiment

During preliminary field observations throughout the study area, there were occasions when lengthy backups were noted on the approaches of Oak Street and Shelby Street. The backups usually occurred during the evening rush hours. The queues became excessive, often extending well beyond the upstream intersection when trains passed through the grade crossings just east and south of the intersection (Fig. G-73), stopping all traffic for several minutes. After the train had passed, several cycles were required before conditions returned to normal.

The prohibition of curbside parking provides additional capacity which reduces backups under normal conditions and enables the intersection to clear more quickly after the occurrence of a train crossing.

The Shelby Street southbound approach is 36 ft wide, painted with a center line only, and parking is permitted on both curbsides. The removal of parking on the east curb provides a 9-ft lane for vehicles turning into Oak Street and retains two lanes for through vehicles.

Oak Street is 36 ft wide, west of Shelby Street, with parking prohibited along the south curb face, and is marked for three lanes. The prohibition of parking on the north curb face would not provide an additional travel lane, because a 9-ft lane is too narrow for the sharp "S" curve alignment of Oak Street west of Shelby Street and because Oak Street east of Shelby Street is only 26 ft wide. However, removal of parking along the north curb line increased use of the north lane (marked for 16 ft with parking), because drivers no longer had to avoid parked vehicles on this sharp curve.

A use study of the adjoining properties showed six retail establishments and seven residences located along the east side of Shelby Street between Oak and Mary Streets (Fig. G-74). Except for a drugstore at the corner of Oak and Shelby Streets, all property on the north side of Oak Street was residential.

After discussion with city officials, it was agreed to implement the following parking restrictions:

1. No stopping 4 to 6 PM on the east side of Shelby Street from Oak Street to Mary Street.
2. No stopping 7 to 9 AM and 4 to 6 PM on the north side of Oak Street within 120 ft of Shelby Street.

Surveillance System Design.—The variables measured to evaluate the effect of this experiment were vehicles through each green interval for each approach leg, recording the vehicle type and turning movement, vehicles stored during each red interval for each approach lane, and the rate of arrival on each approach during the red interval.

Implementation of Improvement.—Standard signs restricting parking, NO STOPPING 4 TO 6 PM, were placed on existing poles spaced about 75 ft apart along the east side of Shelby Street from Oak to Mary Street. Temporary NO STOPPING 7 TO 9 AM AND 4 TO 6 PM signs were taped on existing poles and trees spaced about 40 ft apart on the north side of Oak Street for a distance of 120 ft west of Shelby Street.

In accordance with standard practice in Louisville, the signs were placed 24 hr in advance of police enforcement. The signs were placed on Tuesday, June 18; the compliance by Thursday, June 20, was observed to be very good.

Surveys.—The "before" survey was conducted on June 14 through June 18, and the "after" survey, on June 21 through June 25, 1968. Three time periods (7:30 to 8:30 AM, 1:30 to 2:30 PM, and 4:30 to 5:30 PM) were surveyed during each day of measurement. One man recorded the vehicles stored and vehicles clearing by cycle on the Oak Street aproach, and a second man recorded the Shelby Street approach. A third man recorded the arrival rate each 3 sec, alternating between the Shelby Street and Oak Street approaches; and the fourth man monitored each adjacent signalized intersection.

While the curb lanes were occupied with parked vehicles during the "before" study, the turnover rate was observed to be quite low; and the parking vehicles caused no interference to the moving lanes on either approach. A city policeman assisted in clearing the Oak Street north curb lane and the Shelby Street east curb lane of parked vehicles during the period of "after" measurements, eliminating the need to record illegally parked vehicles.

On the occasions when train crossings stopped traffic through the intersection, the queue length was counted only as far as the upstream intersection, and the arrival rate was abandoned during these congested cycles.

First Level Analysis

Purpose.—The purpose of the First Level Analysis is to evaluate the significance of local changes in traffic flow characteristics caused by the parking restrictions through a statistical comparison of "before" and "after" measurements. The following measurements were studied for each approach to the intersections: vehicles through the intersection per cycle, the number of vehicles queued per cycle, and the vehicle delay time during the red interval per cycle. The delay time per vehicle through, delay time per vehicle stopped, and the percentage of vehicles stopped were also analyzed by cycle.

Summary of Field Data.—The data were summarized by 5-min periods during the morning peak period (7:30 to 8:30 AM) on the Oak Street approach, because the quantities per cycle were quite low. The summarization was performed by cycle for the evening peak period (4:30 to 5:30 PM) on each approach. The two days of morning counts summarized were:

PERIOD		"BEFORE"	"AFTER"
Morning	Day 1	June 17 (Mon.)	June 24 (Mon.)
	Day 2	June 18 (Tues.)	June 21 (Fri.)
Evening	Day 1	June 14 (Fri.)	June 21 (Fri.)
	Day 2	June 18 (Tues.)	June 25 (Tues.)

The number of vehicles clearing each cycle and the number of vehicles stored each cycle were summarized, and the delay time was computed from the rate of arrival during the red interval. Extreme values resulting from unusual happenings (such as a train crossing that stopped all traffic through the intersection for several minutes) were identified for separate analysis.

Oak Street Approach. Morning Peak Hour.—The data for the number of vehicles through, number of vehicles queued, and total delay time by 5-min periods between 7:30 and 8:30 AM were analyzed to determine the significance of "before" and "after" differences on the Oak Street approach. The data for each variable were plotted (Fig. G-75, G-76, and G-77), and a two-way ANOVA, comparing "before" Day 1 and Day 2 values with "after" Day 1 and Day 2 values, was performed.

The analysis indicated that there was no significant difference ($\alpha = 0.05$) between mean values for "before" or "after" number of vehicles through the approaches, or the number of vehicles queued at the approach. There was, however, a significant interaction between days and "before" or "after" vehicles queued. That is, there was an increase of vehicles queued from "before" to "after" for Day 1; there was a corresponding decrease from "before" to "after" for Day 2. The percentage of vehicles stopped, or stop ratio, was studied; and the data were transformed (using $\sqrt{x+1}$) to reduce the difference between variations. The analysis showed no significant difference between "before" and "after" percentage of vehicles stopped. The total delay time was also transformed; and the analysis showed no difference between "before" and "after" values, although there was a difference between days. Even though the volumes were not significantly different, the differences could affect the total delay time. Therefore, the delay per vehicle stopped and the delay per vehicle through were also analyzed. The results showed no significant difference at the 95-percent interval for any of the variables.

Evening Peak Hour.—Data for the number of vehicles through, the number of vehicles queued, and the total delay time per cycle between 4:30 and 5:30 PM are shown in Figures G-78, G-79, and G-80.

1. A two-way ANOVA showed no significant difference between days or between "before" and "after" values of number of vehicles through each cycle on Oak Street (Table G-37). The marginal mean value for the two days

TABLE G-37

COMPARISON OF VARIABLE MEAN VALUES PER CYCLE, 4:30 to 5:30 P.M.

VARIABLE	"BEFORE" MEAN DAY 1	"BEFORE" MEAN DAY 2	"AFTER" MEAN DAY 1	"AFTER" MEAN DAY 2	SIG. TEST (95% LEVEL)
			(*a*) Oak St.		
Vehicles through	18.5	18.4	19.6	18.7	ANOVA—no
Queue length	3.5	2.4	4.6	3.6	Median—no
Percentage stopped	18.3	12.5	22.4	16.6	Transformed ANOVA—no
Delay time (veh/sec)	63.4	30.2	92.3	69.8	Median—no
Delay/vehicle stopped	16.1	11.8	18.7	17.1	Transformed ANOVA—yes
Delay/vehicle through	3.3	1.7	4.4	3.3	Transformed ANOVA—yes
			(*b*) Shelby St.		
Vehicles through	16.0	16.1	16.9	15.6	ANOVA—no
Queue length	6.7	4.7	5.2	6.7	Transformed ANOVA—no
Percentage stopped	40.8	27.9	29.5	41.3	Transformed ANOVA—no
Delay time (veh-sec)	163.2	117.4	132.0	152.1	Transformed ANOVA—no
Delay/vehicle stopped	23.8	23.1	22.7	23.7	Median—no
Delay/vehicle through	9.6	7.1	7.5	9.6	Transformed ANOVA—no

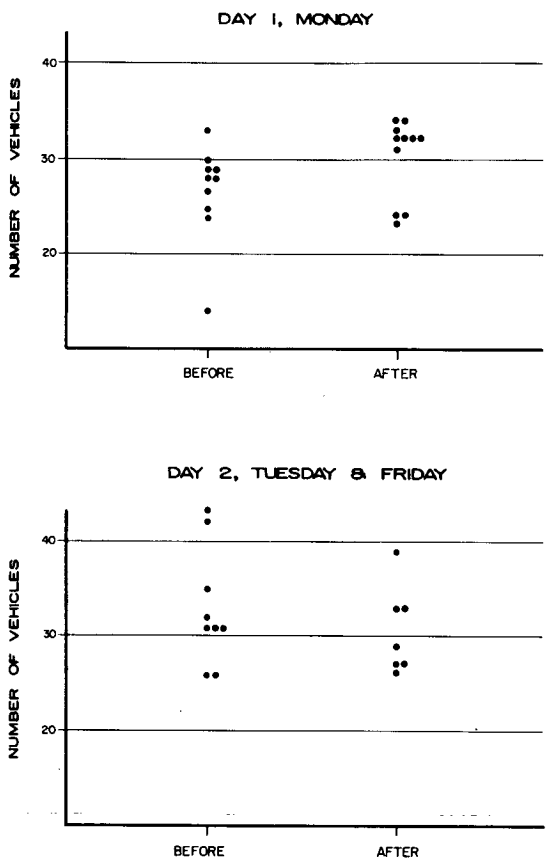


Figure G-75. Number of vehicles through Oak Street eastbound by 5-min periods (7:30 to 8:30 AM*).*

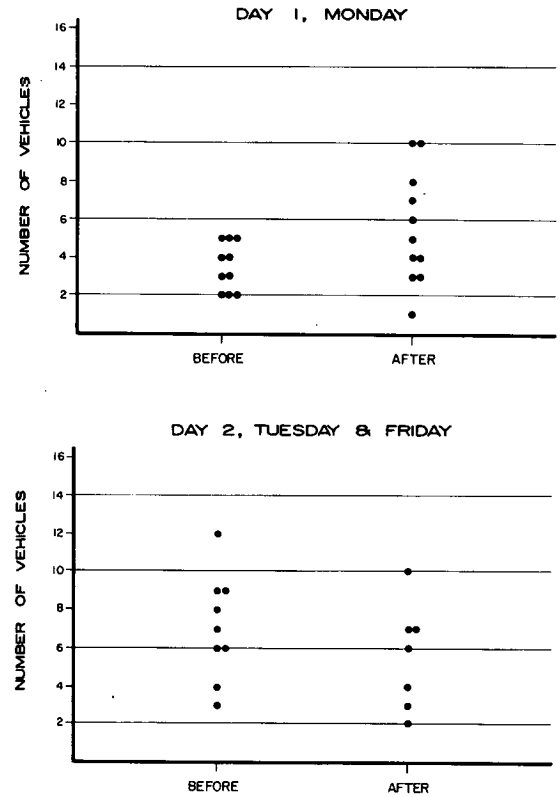


Figure G-76. Number of vehicles queued at Oak Street eastbound by 5-min periods (7:30 to 8:30 AM*).*

"before" was 18.42 vehicles per cycle, and for the two days "after," 19.16 vehicles per cycle.

2. Because of unequal variances between the number of vehicles queued per cycle for all days, a *t* test was performed comparing differences between Day 1 "before" and Day 1 "after." The *t* test showed the differences (3.51 "before" and 4.59 "after") were not significant at the 95-percent level. A median test showed no difference between "before" (2.38) or "after" (3.56) vehicles queued per cycle for Day 2.

3. A median test showed that the increase from 63.4 veh-sec delay for Day 1 "before" to 92.3 veh-sec delay for Day 1 "after" was not significant; nor was the difference significant between "before" and "after" Day 2 delay time of 30.2 veh-sec "before" and 69.8 veh-sec "after."

4. After transforming the data, the ANOVA showed a significant difference between "before" and "after" for both delay per vehicle stopped and delay per vehicle through. The marginal mean for the "before" delay per vehicle stopped was 13.97 sec, and the marginal mean for both "after" days was 17.86 sec. The delay per vehicle through marginal mean was 2.51 sec "before" and 3.83 sec "after."

Shelby Street Approach.—Because there was no parking revision during the morning peak hour, the Shelby Street approach was analyzed for the evening peak hour (4:30 to 5:30 PM) only. Plots of the number of vehicles through,

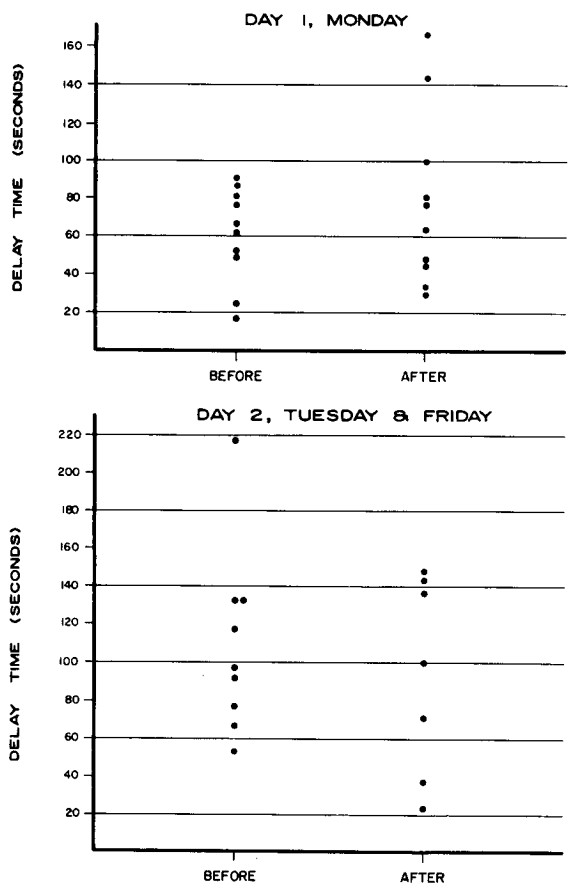


Figure G-77. Vehicle delay time at Oak Street eastbound by 5-min periods (7:30 to 8:30 AM*).*

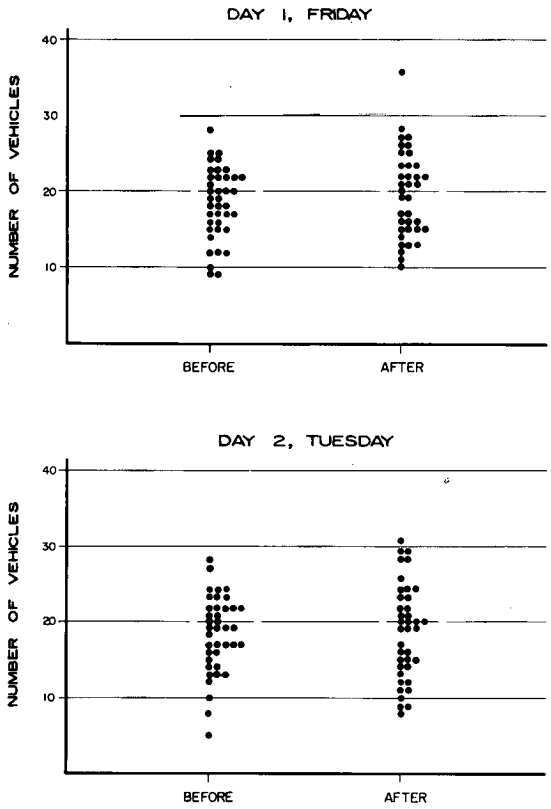


Figure G-78. Number of vehicles through Oak Street eastbound by 60-sec cycles (4:30 to 5:30 PM).

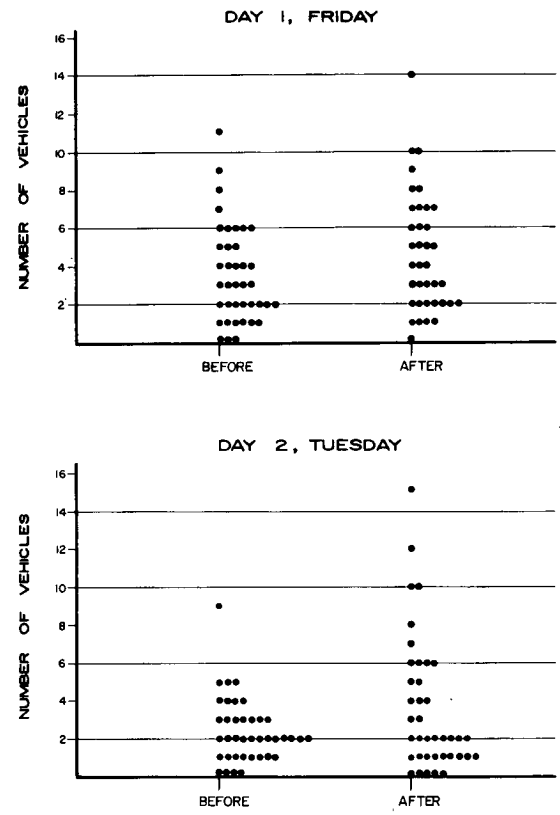


Figure G-79. Number of vehicles queued at Oak Street eastbound by 60-sec cycles (4:30 to 5:30 PM).

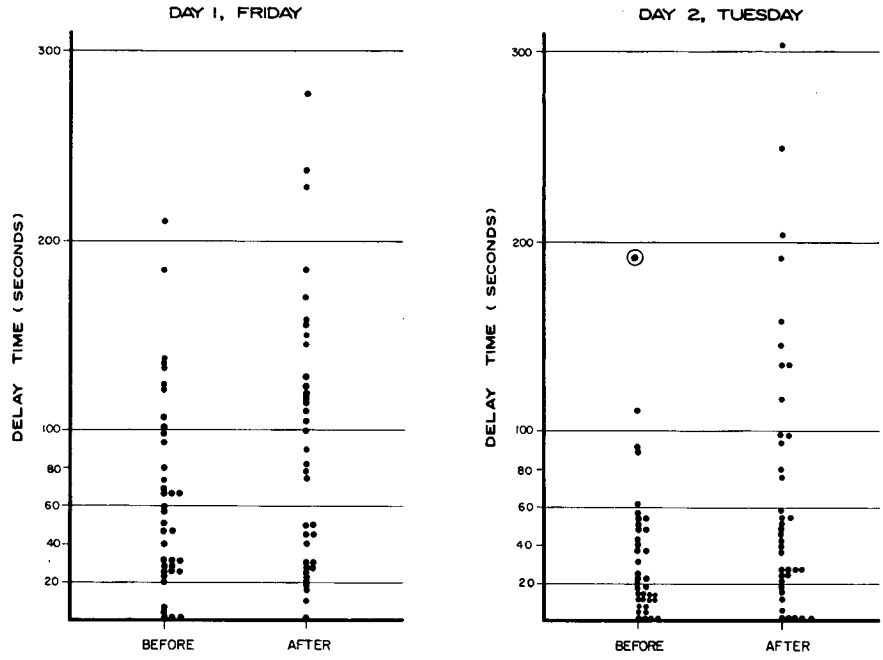


Figure G-80. Vehicle delay time at Oak Street eastbound by 60-sec cycles (4:30 to 5:30 PM).

number of vehicles stopped, and total delay time, summarized by cycle, are shown in Figures G-81, G-82, and G-83.

"Before" and "After" Comparison.—An ANOVA showed no significant difference between "before" (16.06) and "after" (16.26) vehicles per cycle through the approach (Table G-38). After transformation, an ANOVA showed no significant difference between vehicles stopped (or vehicles queued) "before" (5.67) or "after" (5.95), and no difference between "before" (140.36 veh-sec) or "after" (142.04 veh-sec) total delay time. There was a significant interaction between days and "before" or "after" —Day 1 decreased from "before" to "after," whereas Day 2 increased in value for both vehicles stopped and delay time.

A median test showed the delay per vehicle stopped (23.45 sec "before" and 23.18 sec "after") was not significant. An ANOVA of the transformed data for delay per vehicle through showed no significant difference between "before" and "after" (8.32 sec "before" and 8.52 sec "after"), and no difference for the percentage of vehicles stopped "before" and 'after" (34.34 percent "before" and 35.40 percent "after"), although there was a significant interaction between days, Day 1 decreasing while Day 2 increased from "before" to "after" for both variables.

Special Peak Load Study.—The several cycles during and immediately after a train crossing were eliminated from the statistical test for "before" and "after" comparisons. A special study of the first several cycles immediately following the passing of a train was performed to determine if the parking restriction resulted in any difference of "before" and "after" number of vehicles clearing the intersection during these periods of peak loading.

Table G-39 compares the PM average number of vehicles through per cycle. A Wilcoxon Rank Sum test showed that the difference between "before" and "after" measurements on Oak Street for Day 1 was significant at the 95-percent confidence level, but Day 2 was not. The difference for Day 2 between "before" and "after" number of vehicles clearing the Shelby Street approach during the first eight cycles after a train crossing was significant at the 95-percent level, whereas the difference for Day 1 was not.

TABLE G-38

SUMMARY OF VARIABLE MEANS BY APPROACH LEG, VALUE OF DAY 1 AND DAY 2 BY CYCLE

	OAK ST., AM			OAK ST., PM			SHELBY ST., PM		
VARIABLE	"BEFORE"	"AFTER"	Sig. @ $\alpha=0.05$	"BEFORE"	"AFTER"	Sig. @ $\alpha=0.05$	"BEFORE"	"AFTER"	Sig. @ $\alpha=0.05$
Vehicles through	5.97	6.07	No	18.42	19.16	No	16.06	16.26	No
Vehicles stopped	1.06	1.11	No	2.95	4.08	No	5.67	5 95	No
Percentage stopped	23.63	17.85	No	15.39	19.52	No	34.34	35.40	No
Delay time (veh-sec)	17.15	16.67	No	46.82	81.01	No	140.36	142.04	No
Delay/vehicle through (sec/veh)	2.88	2.66	No	2.51	3.83	Yes	8.32	8.52	No
Delay/vehicle stopped (sec/veh)	16.90	14.06	No	13.97	17.86	Yes	23.45	23.18	No

TABLE G-39

COMPARISON OF VEHICLES CLEARING EACH CYCLE (PM)

	VEHICLES CLEARING EACH CYCLE											
	OAK ST.						SHELBY ST.					
	NORMAL			TRAIN CROSSING			NORMAL			TRAIN CROSSING		
CONDITION	MEAN	LOW	HIGH	MEAN	LOW	HIGH	MEAN	LOW	HIGH	MEAN	LOW	HIGH
"Before":												
Day 1	18.5	9	28	27.2	24	30	16.0	10	25	16.9	12	20
Day 2	18.4	5	29	33.0	29	36	16.1	7	23	16.8	9	20
Average	18.4	5	29	30.1	24	36	16.1	7	25	16.9	9	20
"After":												
Day 1	19.6	10	36	32.6	29	39	16.9	10	25	18.8	16	22
Day 2	18.7	8	31	32.0	29	36	15.6	6	22	20.2	16	25
Average	19.2	8	36	32.3	29	39	16.3	6	25	20.0	16	25

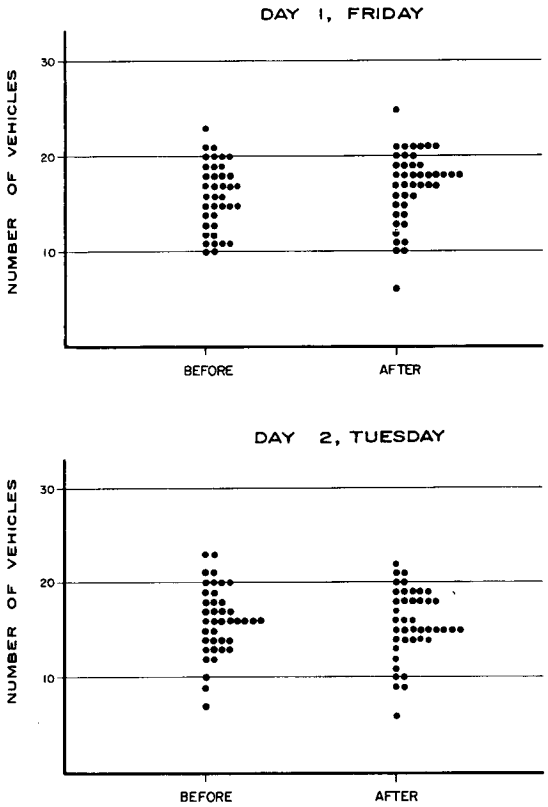


Figure G-81. Number of vehicles through Shelby Street southbound by 60-sec cycles (4:30 to 5:30 PM).

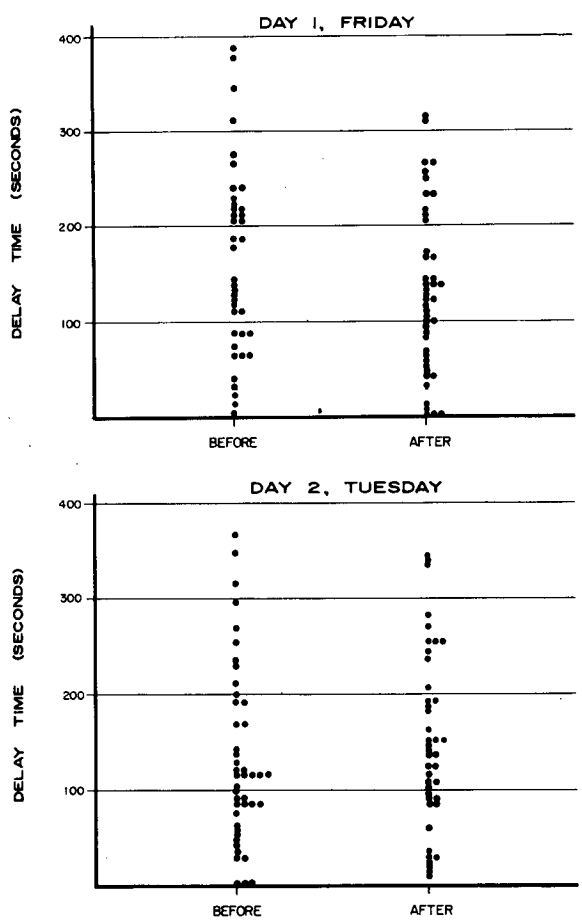


Figure G-83. Vehicle delay time at Shelby Street southbound by 60-sec cycles (4:30 to 5:30 PM).

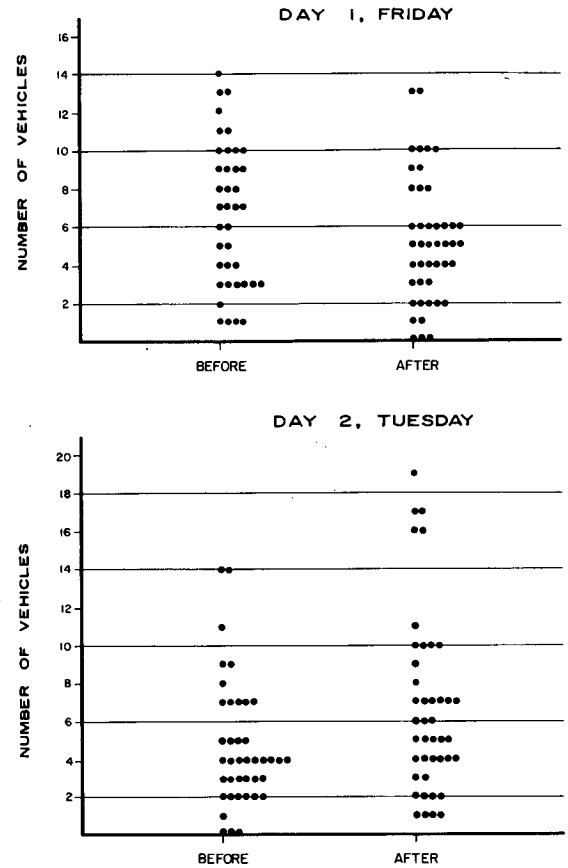


Figure G-82. Number of vehicles queued at Shelby Street southbound by 60-sec cycles (4:30 to 5:30 PM).

Conclusions

The results of this experiment show that the removal of curbside parking during peak traffic periods on the approaches to this intersection of two one-way streets does not necessarily improve the traffic flow characteristics of the intersection.

Oak Street Approach.—The volume through the Oak Street approach during the morning peak hour (7:30 to 8:30 AM) is low (360 vph); the evening volume (4:30 to 5:30 PM) is 1,100 vph. None of the measurements taken showed any significant change resulting from the restriction of parking on the north curbside.

There was no significant change in vehicles clearing each cycle, vehicles stopped each cycle, or total vehicle delay time per cycle during the evening peak hour on Oak Street. There was, however, a significant change in the delay per vehicle stopped and delay per vehicle through on Oak Street during the evening peak hour, both values increasing as a result of the parking revision. The average delay per vehicle stopped was 13.97 sec before and 17.86 sec after the restriction. The average delay per vehicle through was 2.51 sec before and 3.83 sec after the revision. The reason for the increased delay could be that vehicles would use all three lanes and increase their speed slightly, arriving at the signalized intersection a few seconds earlier when the curb lane was free of parked vehicles.

Shelby Street Approach.—There was no change in any of the variables measured before and after restricting parking on the entire east curbside on Shelby Street during the evening peak hour. The number of vehicles through, number of vehicles stopped, total delay time, average delay per vehicle through, and average delay per vehicle stopped did not change significantly at the 95-percent confidence level.

Peak Load Study.—A special study of each aproach was performed for the several cycles immediately following a train crossing, when traffic was severely backed up, as summarized in Table G-39. No conclusive results could be drawn from a Wilcoxon Rank Sum test of these data; one day showed a significant change for each approach; another day showed no change at the 95-percent level.

Conclusions.—A capacity study of each approach would indicate a 50-percent increase on Oak Street and a 20-percent increase on Shelby Street resulting from the parking restrictions. This experiment showed that this increase was not realized, even during peak loading conditions; this is probably due to a combination of existing lane markings, limited downstream capacity, and local driving habits.

It would not be reasonable to repaint the lane markings unless the parking restrictions were to be put into effect full-time rather than just during peak hours. The limited downstream capacity is probably the factor that eliminated any benefits from this experiment. Shelby Street south of the intersection of Oak Street is 36 ft wide, with parking permitted on both sides; Oak Street east of Shelby Street is only 26 ft wide.

Convenience and Safety.—The loss of free curbside parking always results in some inconvenience and discomfort to the abutting property owners. The parkers on both Oak and Shelby Streets were observed to be long-term parkers rather than customer parkers, so the total number of vehicles affected by the parking restrictions was rather small. There were no complaints from store owners or residents during the survey period.

The restriction of parking on the north side of Oak Street was proposed to eliminate the hazardous condition of maneuvering through a tight "S" curve in the 16-ft curb lane with parked vehicles. Police records list six collisions in 1967 at Oak and Shelby Streets and two accidents on Oak Street between Clay and Shelby Streets.

Parking Restrictions on Seventh Street North of Oak Street—Experiment D66

The Louisville Traffic Engineering Department had planned for the elimination of parking at the approaches to several intersections to provide separate lanes for turning vehicles. The intersection of Seventh Street and Oak Street is one of these locations. Although traffic conditions are not very severe at this location and only minor benefits were anticipated, the situation was accepted as an experiment because it offered an opportunity to extend the range of traffic flows under which this type of improvement is to be analyzed.

Experiment D66 examines the effect of providing an additional travel lane by prohibiting parking on tthe east side of Seventh Street north of Oak Street in Louisville. The additional lane was primarily intended to provide a separate lane for left turns from Seventh Street into Oak Street. Seventh Street is a one-way southbound street and Oak Street is one-way eastbound. The elimination of parking on the east side of Seventh Street provides a lane for traffic making a left turn into Oak Street, reducing the interference of turning traffic on the through lanes. The location of this intersection is shown in Figure G-84.

Experimental Area

Seventh Street is a 42-ft-wide arterial street marked for four travel lanes. However, only two lanes are available when parking is permitted on both sides of the street. Under conditions that prevailed prior to the experiment, parking was permitted on the west side of the street, except from 7 to 9 AM and from 4 to 6 PM on all days. Parking was permitted on the east side of the street, except from 4 to 6 PM on weekdays. Oak Street is a 36-ft-wide arterial street marked for three travel lanes. Parking was permitted on the north side of Oak Street at all times and prohibited on the south side from 7 to 9 AM and from 4 to 6 PM on weekdays.

The signal at the intersection is operated by a two-dial controller on a 60-sec cycle. Dial 1 is in operation from 5:45 PM to 2:15 PM the following afternoon, with 59 percent of the cycle allotted to Oak Street and 41 percent allotted to Seventh Street. Dial 2 is in operation from 2:15 PM to 5:45 PM, with 55 percent of the cycle allotted to Oak Street and 45 percent allotted to Seventh Street.

Figure G-85 shows the physical details of the area in which this experiment was conducted.

Design of Experiment

The physical improvement consisted of installing two signs reading NO STOPPING AT ANYTIME at the locations shown in Figure G-85. The installation of these signs was completed on November 8, 1968.

Surveys to measure the volumes through this intersection and travel time on Seventh Street between Oldham and Oak Streets were made between the hours of 7 and 11 AM of the following days (1968):

	DATE	
DAY	"BEFORE"	"AFTER"
Tuesday	10/29	11/19
Wednesday	10/30	11/20
Thursday	10/31	11/14

The average hourly traffic volumes for vehicles traveling through on Seventh Street and turning left into Oak Street are given in Table G-40. Detailed information recorded while making the volume counts included the number of vehicles through per cycle, number of vehicles stopped on red per cycle, and number of turns. Computations based on procedures outlined in the 1965 *Highway Capacity Manual* (*1*) indicate a level of service A for all of the

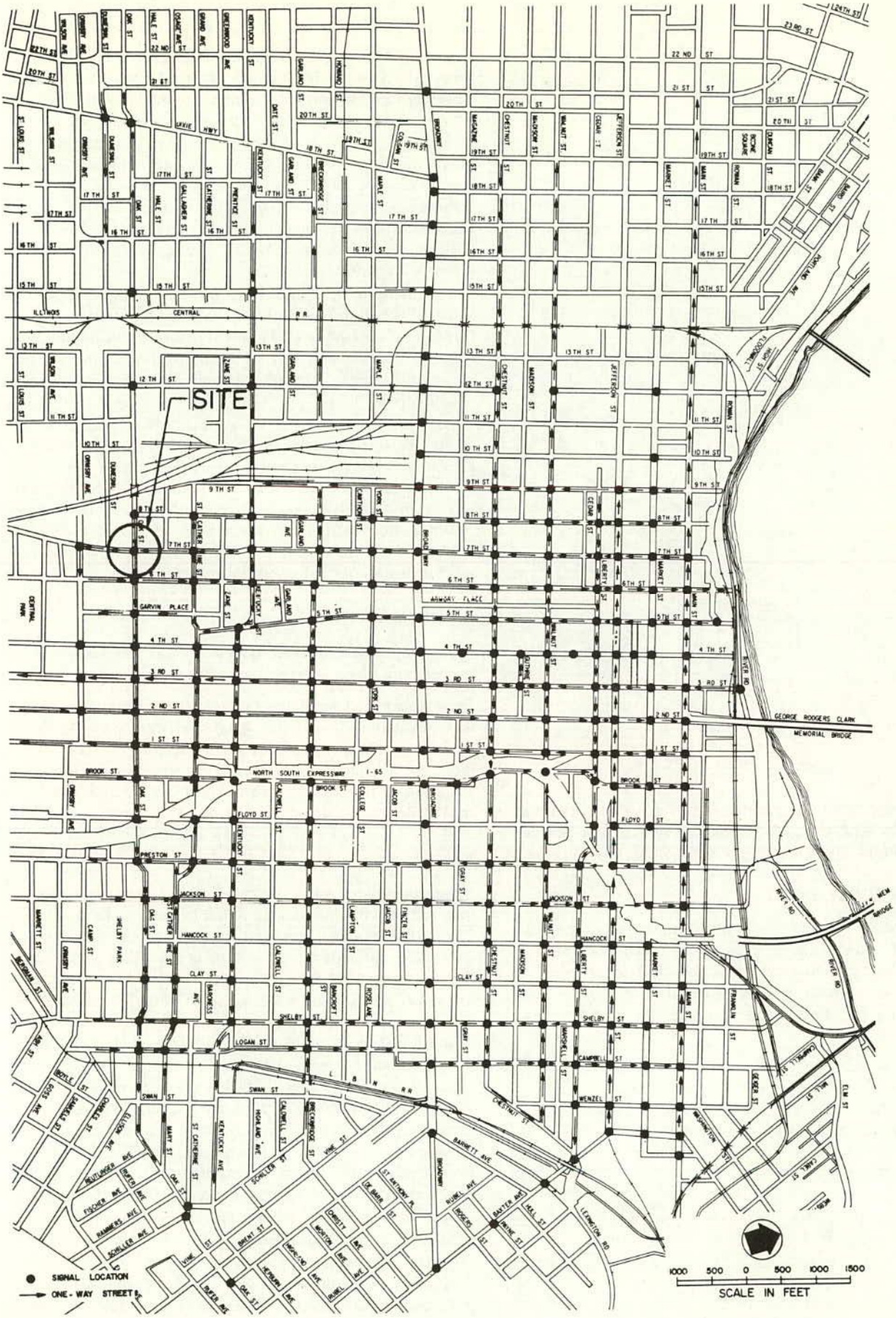


Figure G-84. Location map, Experiment D66.

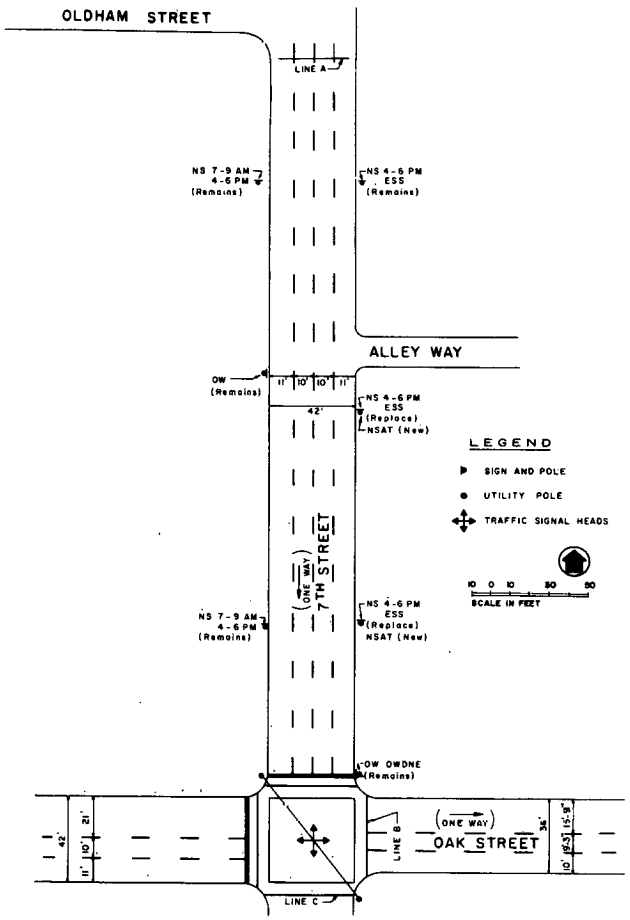


Figure G-85. Design plan.

"before" and "after" conditions surveyed. Travel times of automobiles and light trucks on Seventh Street between Oldham Street and Oak Street were randomly surveyed.

Analysis and Conclusions

Comparison of "before" and "after" measurements of the number of vehicles through per cycle and the number of vehicles stopped per cycle indicated that there were no significant differences between the means of "before" and "after" samples. Travel time for vehicles traveling from Oldham Street to Oak Street on Seventh Street were also analyzed. This analysis was performed both for vehicles traveling straight through the intersection at Oak Street (see movement from line A to line C in Fig. G-85) and those making a left turn onto Oak Street (see movement from line A to line B in Fig. G-85). In the initial analysis it became apparent that two distinct populations had been surveyed—a population for vehicles encountering signal delay and those not encountering signal delay. The comparison of "before" and "after" means of travel times is given in Table G-41.

The analysis of travel time for automobiles indicated some slight improvement for left-turning and straight-through movement, but no improvement was found in travel time for truck categories. The improvement in flow for automobiles took place only when signal delay was not encountered.

As previously mentioned, an analysis of the level of service in accordance with the procedures outlined in the *Highway Capacity Manual* indicated a level of service A for both "before" and "after" conditions. Therefore, it is not surprising that no improvement of any magnitude resulted from the parking restrictions. Field observations, however, confirmed that the use of the separate lane for left-turning traffic did result in smoother flow.

TABLE G-40

SEVENTH STREET AVERAGE HOURLY TRAFFIC VOLUMES

TIME	MOVEMENT	AVERAGE VEHICLES PER HOUR "BEFORE"	"AFTER"
AM	Left	93	95
	Through	677	713
Midday	Left	107	100
	Through	498	481

CHANNELIZATION

Pedestrian-Vehicle Conflict Control, Market Street at Washington Street—Experiment A33

Experiment A33 evaluates the effects of pedestrian crosswalk relocations and traffic signal operations on pedestrian-vehicle conflicts at an offset intersection. The work included obliteration of existing pavement markings, installation of new markings and signs, and modifications to the existing traffic signal installation.

Experimental Area

Experiment A33 was conducted on Market Street at its intersection with Washington Street in Newark (Fig. G-86). The existing signal installation is part of the master-controlled, interconnected system in the CBD area. During the "before" and "after" conditions of this experiment, the signal operated on a 90-sec cycle, providing a basically simultaneous offset relationship to adjacent signals. No changes in cycle length or offset arrangements were made during this experiment.

As Figure G-87 shows, Market Street has a clear curb-to-curb width of 59 ft on the east side of the intersection, expanding into 73+ ft on the west. Pavement markings provided for five traffic lanes on the westbound approach and six lanes on the eastbound approach. On the eastbound approach, three lanes were designated for each direction of movement during peak traffic periods. Parking was permitted in both curb lanes at all other times, resulting in only two lanes being available for each direction of movement. On the westbound approach during the AM peak traffic period, three lanes were used for eastbound movement and two were used for westbound movement.

TABLE G-41

"BEFORE" AND "AFTER" TRAVEL TIME MEANS

VEHICLE TYPE	MOVEMENT	NO. OF OBS. "BEFORE"	NO. OF OBS. "AFTER"	MEAN TRAVEL TIME (SEC) "BEFORE"	MEAN TRAVEL TIME (SEC) "AFTER"	SIG. % CHANGE
		(*a*) With signal delay				
Automobile	AB	229	219	32.8	32.0	—
	AC	41	36	34.3	33.8	—
Light truck	AB	48	41	36.7	34.0	—
	AC	7	16	33.5	37.9	—
Heavy truck	AB	47	33	32.1	31.5	—
	AC	—	—	Insufficient data		—
		(*b*) Without signal delay				
Automobile	AB	173	187	10.8	9.9	—8 3[a]
	AC	67	57	11.9	11.1	—6.7[a]
Light truck	AB	50	45	12.0	12.8	—
	AC	14	18	13.1	12.0	—
Heavy truck	AB	37	53	15.2	14.2	—
	AC	21	14	13.7	12.9	—

[a] Although an improvement was measured in 9 of the 11 comparisons that were made, in only two cases was the improvement significant, considering the variability of surveyed data.

During the PM peak period, the direction of movement in the center lane was reversed; three lanes were used for westbound movement, with the north curb lane being reserved for buses. In nonpeak traffic periods, parking was permitted in the south curb lane, resulting in two lanes being available for each direction of travel.

East of Washington Street during the "before" condition, the reversible flow in the center lane on Market Street operated without regulatory or guide signs and without the use of lane signals or traffic cones. Between 4:00 and 6:00 PM, the change occurred when the westbound curb lane was reserved for buses. This latter operation was effected by (1) portable, pedestal-mounted signs and chains placed on the lane line separating the curb and second lanes, and (2) by curbside signs mounted on light standards. The pedestal signs and chains were used to define a platform area for passengers waiting to board buses in the second lane.

Washington Street has a clear curb-to-curb width of 52 ft. North of its intersection with Market Street (Fig. G-87) the number of moving traffic lanes varied from two to four lanes, depending on the volume of traffic and the degree of parking occurring in each curb lane. South of the intersection, four lanes were used for the through and right-turn movements, and two lanes were used for left turns.

Market Street is a major east-west arterial, and Washington Street is a one-way northbound arterial. Their intersection lies on the western edge of the CBD. Government office buildings to the west and a municipal parking facility located in the southwest quadrant of the intersection contribute to high pedestrian volumes within the intersection, particularly during peak traffic periods. Figure G-88 shows the results of two peak-hour traffic surveys that indicate the magnitude of vehicular and pedestrian activity during the "before" condition.

Experimental Design

The "before" traffic phasing (Fig. G-89) did not provide sufficient clearance time for through vehicles to clear the intersection. As a result, eastbound vehicles frequently were required to stop prior to crossing the east crosswalk; and westbound vehicles frequently crossed the west crosswalk in conflict with pedestrians and north-to-west left-turning vehicles. The latter situation was a result of the lack of signal indications at the far side of the conflict area. In the former situation, eastbound vehicles stopped at the east crosswalk and delayed right-turning vehicles from Washington Street. Both these movements frequently blocked the intersection, resulting in congestion and delay to the through and right-turning movements of northbound and eastbound traffic.

The experiment was designed to reduce delays to northbound traffic caused by eastbound queues at the east crosswalk blocking the intersection. This purpose was accomplished by providing sufficient time for all eastbound and westbound vehicles entering the intersection to clear all points of conflict prior to the release of Washington Street traffic. To minimize the clearance time required and to compact the intersection area, the east crosswalk was relocated. The hazardous situation at the west crosswalk was treated similarly (Fig. G-90).

To ensure that sufficient clearance time was provided for east-west vehicles, the existing signal faces were relocated and additional signal faces were installed (Fig. G-90). In addition, the signal operations were revised to provide a

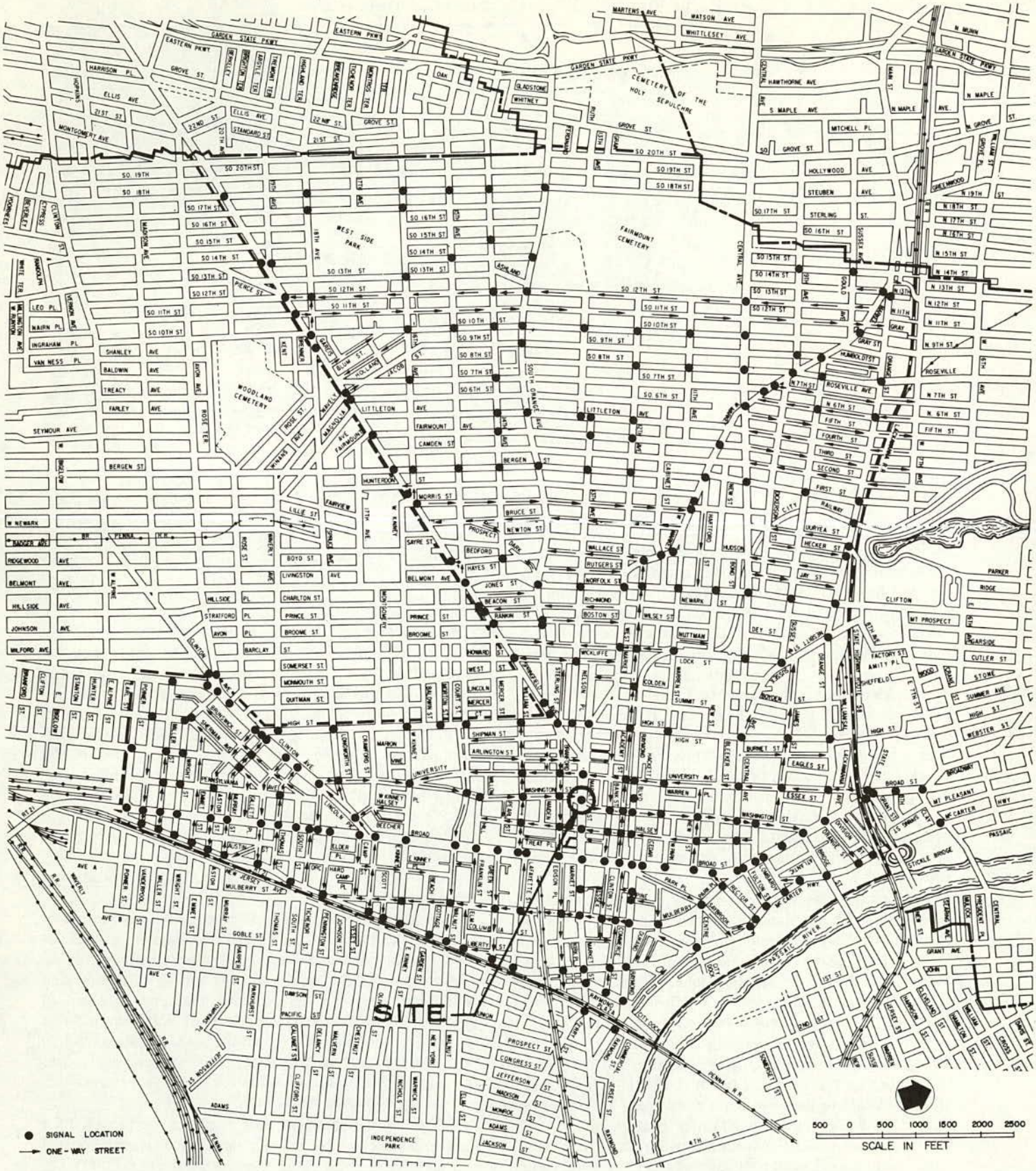


Figure G-86. Location map, Experiment A33.

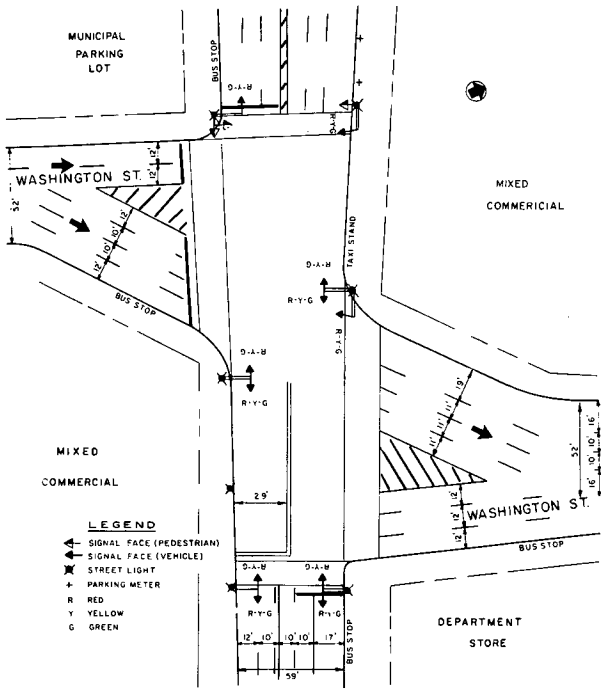


Figure G-87. Vicinity map, "before."

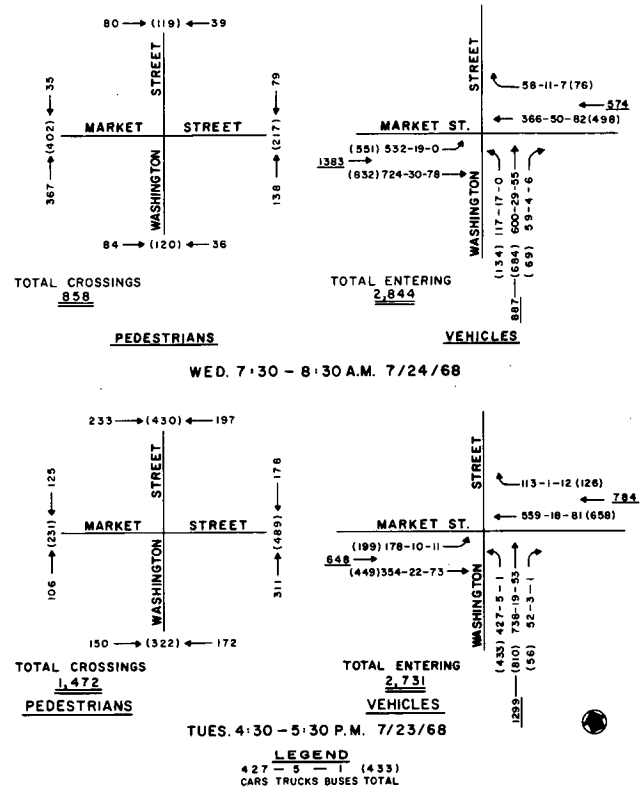


Figure G-88. Peak-hour traffic volumes.

second clearance interval for Market Street traffic (Fig. G-91). Pedestrian pipe barricades and portable concrete pedestals (Fig. G-90) were used to control pedestrian movement and delineate refuge areas.

Providing a second clearance interval for traffic proceeding successively through two closely spaced intersections is normally described as a double-clearance operation. Under ideal circumstances, separate signal indications are provided for each direction of aproaching traffic at each intersection. These indications are operated to stop most, if not all, traffic at the first approach and, after an interval of time (normally sufficient to drive through the intersections), stop any

PHASE	MARKET STREET / WASHINGTON ST.	INTERVAL	COLOR SEQUENCE E-W MARKET ST.	N.B'd. WASHINGTON ST.	E-W PEDS	N-S PEDS	INTERVAL TIMING (SEC.)
A		1	G	R	W	DW	45.6
		2	G	R	DW	DW	4.0
		3	Y	R	DW	DW	4.2
B		4	R	G	DW	W	32.8
		5	R	G	DW	DW	2.8
		6	R	Y	DW	DW	3.2
						TOTAL CYCLE	92.6

VEHICLE MOVEMENT
PEDESTRIAN MOVEMENT

Figure G-89. Traffic signal operations, "before."

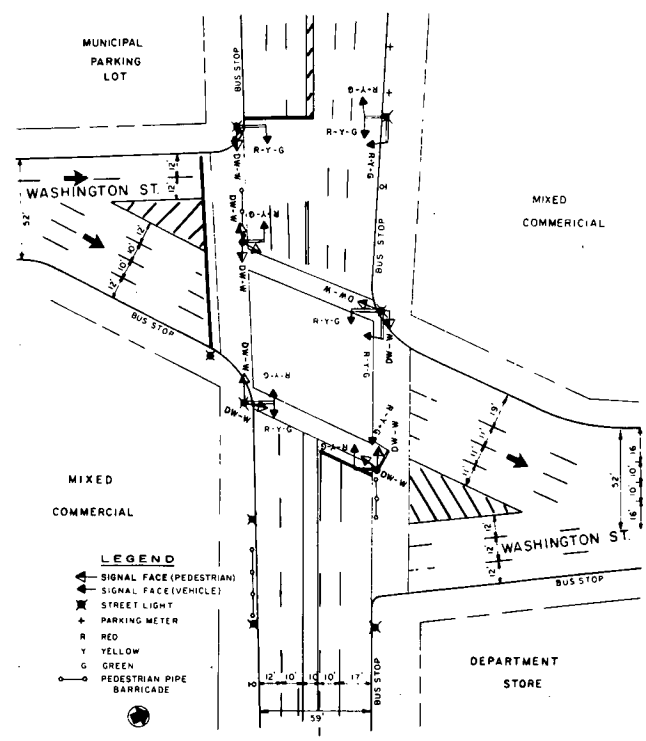


Figure G-90. Vicinity map, "after."

PHASE	MARKET STREET / WASHINGTON ST.	INTERVAL	COLOR SEQUENCE: E-W MARKET ST. FIRST SIGNALS	E-W MARKET ST. SECOND SIGNALS		N.B'd. WASHINGTON ST.	E-W PEDS	N-S PEDS	INTERVAL TIMING (SEC.)
A		1	G		G	R	W	DW	33.0
		2	G		G	R	DW	DW	12.0
		3	Y	Y	G	R	DW	DW	5.0
A_2		4	R	R	G	R	DW	DW	6.0
B		5	R	R		G	DW	W	16.6
		6	R	R		G	DW	DW	16.0
		7	R	R		Y	DW	DW	4.0
								TOTAL CYCLE	92.6

INDICATIONS ONLY VISIBLE TO STOP BAR

INDICATIONS ONLY VISIBLE BEYOND STOP BAR

VEHICLE MOVEMENT

PEDESTRIAN MOVEMENT

Figure G-91. Traffic signal operations, "after."

remaining traffic at the second approach. Approaching motorists view two sets of signal indications that simultaneously display green and then change to yellow on the first set, with green continuing to be displayed on the second set. Such a signal display can cause confusion, particularly if the area between the intersections is limited in length or is poorly defined. Sometimes motorists continue to proceed through the first intersection during the yellow indication. The amount of time during which the second set of signals continues to display a green indication after the first set has changed to yellow varies according to the time required for vehicles to clear the area between the two intersections. If the clearance time is equivalent to the time for the yellow signal indication at the first intersection, one additional interval must be provided by the control equipment for the yellow indication at the second intersection. However, if the clearance time is greater, two additional intervals are necessary: one for the clearance time in excess of the time for the yellow indication at the first intersection, and another for the time for the yellow indication at the second intersection.

For this experiment, the clearance time was greater than the time for the yellow indication at the first intersection, and the existing control equipment was not capable of providing two additional intervals. Because signal indications having the capability of being programmed to a definitive area of visibility were immediately available, a unique solution to the problem was possible. Program-type signals also afforded the opportunity to eliminate the potential confusion resulting from motorists' viewing two conflicting signal indications simultaneously, as previously described. Figure G-92 shows the programmed signals.

Programmed signal indications were used in the second sets of signals for both eastbound and westbound traffic. The red and yellow indications were programmed to be visible only to motorists approaching or stopped at the first

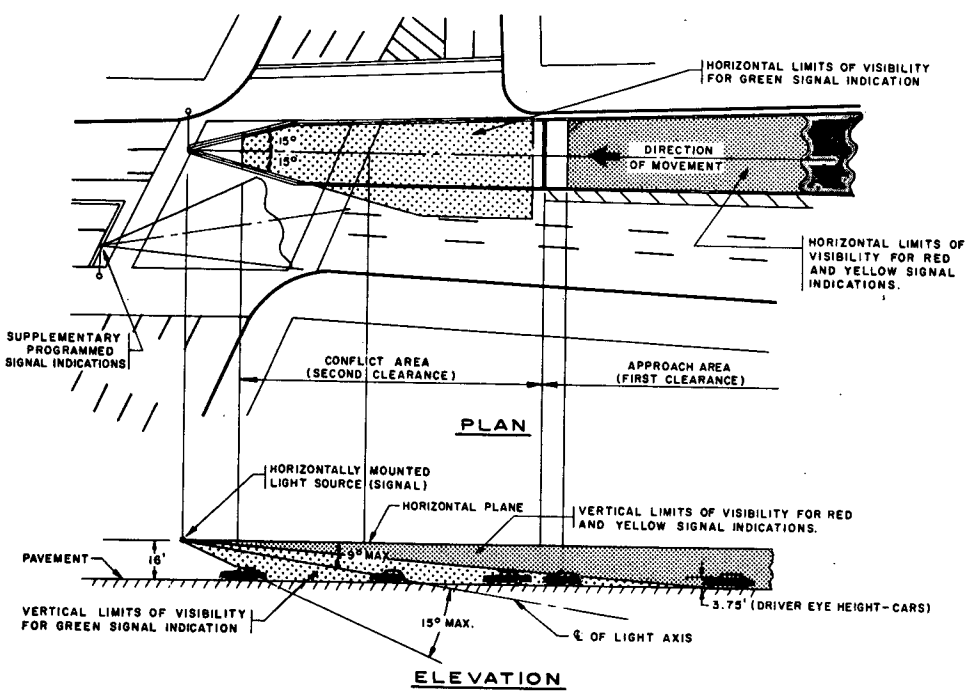


Figure G-92. Typical programmed signal indications.

stopping point and were operated in unison with the first sets of signal indications. The green indications of the second sets of signals were similarly programmed to be visible only to motorists who had proceeded past the first stopping points. These were illuminated at the same time the green signal indications were illuminated in the first sets of signals and continued for 6 sec after red signal indications were displayed by the first sets of signals (Fig. G-91). At the end of this interval (Interval 4), they were extinguished to prevent a conflicting display of right-of-way to motorists unable to clear the conflict area.

Observation of operations after the implementation indicated few instances when eastbound vehicles failed to clear the conflict area. Such instances were the result of eastbound vehicles stopped at Halsey Street and backed up across Washington Street. Motorists' acceptance of and obedience to the revised operations were immediate and without confusion.

Owing to this unique operation and the revised pedestrian crossing locations, pedestrian signal indications were provided at the far side of each crosswalk to properly advise pedestrians of the appropriate time to cross and to ensure adequate pedestrian clearance time. Generally, pedestrians did not observe the relocated crosswalks, probably owing to years of habit and the lack of physical islands defining the refuge area. Westbound motorists continued to stop at the former east crosswalk location, even though the markings and signals had been removed.

This experiment was implemented in conjunction with Experiment C110, Market Street Bus Operations. Operating conditions on the westbound approach were affected by the latter experiment through the elimination of second-lane bus passenger service operation. In addition, a far-side bus stop was created for westbound buses (Fig. G-90).

The surveillance for this experiment consisted of speed and delay runs and manual vehicle counts. Speed and delay runs were made between 7:00 and 9:00 AM and between 4:00 and 6:00 PM on Washington Street between William Street and Raymond Boulevard, a distance of 1,710 ft (0.324 mile). The number of vehicles through per cycle and the number of vehicles stopped per cycle were manually counted at all approaches during the morning peak period of 7:30 to 8:30 AM and the afternoon peak hour of 4:30 to 5:30 PM. Table G-42 gives the surveillance activity and dates of implementation of this experiment.

Analysis

The effects of the combined changes on all three approaches were determined from the manual count data by "before" and "after" comparison of:

1. The number of vehicles stopped on red per cycle.
2. The number of vehicles through per cycle.

Mean volumes are summarized in Tables G-43, G-44, and G-45.

The effects of combined changes on northbound Washington Street traffic were determined from speed and delay data by "before" and "after" comparisons of (1) travel time, (2) delay time, and (3) the frequency of stops at Market Street. Table G-44 summarizes the statistical analysis of and gives the mean values for the foregoing comparison.

For the AM time period, a comparison of the "before" and "after" conditions indicated the following to be statistically significant:

1. An increase of 4.0 vehicles stopped per cycle (66.7 percent) on the eastbound Market Street approach between 7:30 and 8:30 AM.
2. Decreases of 9.2 sec (9.3 percent) in travel time, 7.7 sec (18.6 percent) in delay time, and 77.4 percent in the frequency of stops at Market Street for northbound traffic.

In terms of miles of travel, the changes for northbound traffic are reductions of 0.5 min per mile in travel time and 0.4 min per mile in delay time, with an accompanying increase in average speed of from 11.85 to 13.07 MPH. The increase in the number of eastbound vehicles stopped may have resulted from the 5.1-percent decrease in the cycle time assigned to this approach during the "after" condition.

TABLE G-42

SUMMARY OF SURVEILLANCE PERIODS

CONDITION	SPEED AND DELAY RUNS	MANUAL VOLUME SURVEYS		
		DAY 1	DAY 2	DAY 3
	(*a*) AM time period			
		Tues.	Wed.	Thurs.
"Before"	8/2/68 to 12/18/68	5/6/69	5/7/69	5/8/69 [a]
"After" [c]	5/22, 23, and 26/69	5/27/69	5/28/69	5/22/69 [a]
	(*b*) PM time period			
		Thurs.	Fri.	Tues.
"Before"	11/28/67 to 12/12/68	5/1/69	5/2/69	5/6/69 [b]
"After" [c]	5/22, 23, and 26/69	5/22/69	5/23/69	5/27/69

[a] Day 3 eliminated in statistical analysis; however, summaries include these data.
[b] Day 3 eliminated in statistical analysis and summaries due to signal offset malfunction.
[c] Experiment implemented 5/20/69.

TABLE G-43

SUMMARY OF ANALYSIS, MARKET STREET

CONDITION	EB VEHICLES THROUGH LEFT TURN	EB VEHICLES THROUGH STRAIGHT	EB VEHICLES THROUGH ALL	EB VEHICLES STOPPED	WB VEHICLES THROUGH STRAIGHT	WB VEHICLES THROUGH RIGHT TURN	WB VEHICLES THROUGH ALL	WB VEHICLES STOPPED
(*a*) AM time period								
"Before"	529.3	843.7	1373.0	(6.0)[a] 288.7	532.7	64.7	597.4	245.0
"After"	526.3	819.3	1345.6	(10.0)[a] 374.0	532.3	71.0	603.3	237.3
Net change	−3.0	−24.4	−27.4	(+4.0)[a] +85.3	−0.4	+6.3	+5.9	−7.7
Percent change	−0.6	−2.9	−2.0	(+66.7)[a] +29.5	−0.1	+9.7	+1.0	−3.1
Sig. level	NS	NS	NS	0.05	NS	NS	NS	NS
(*b*) PM time period								
"Before"	204.0	510.5	714.5	(4.8) 193.5	782.3	133.3	915.6	(13.9) 553.7
"After"	197.3	517.7	715.0	(3.2) 132.7	774.7	128.7	903.4	(10.0) 399.3
Net change	−6.7	+7.2	+0.5	(−1.6) −60.8	−7.6	−4.6	−12.2	(−3.9) −154.4
Percent change	−3.3	+1.4	+0.1	(−33.3) −31.4	−1.0	−3.5	−1.3	(−28.1) −27.9
Sig. level	NS	NS	NS	0.05	NS	NS	NS	0.05

NS = not significant at $\alpha \leq 0.05$.
(3.2) = per cycle.
[a] = Day 3 eliminated in statistical analysis.

TABLE G-44

SUMMARY OF ANALYSIS, NORTHBOUND WASHINGTON STREET

CONDITION	SPEED AND DELAY NO. OF RUNS	TRAVEL TIME (SEC)	DELAY TIME (SEC)	STOPS AT MARKET ST.	TRIPS STOPPED (%)	VEHICLES (PEAK HOUR) THROUGH LEFT TURN	THROUGH STRAIGHT & RIGHT	THROUGH ALL	STOPPED LEFT TURN	STOPPED STRAIGHT & RIGHT	STOPPED ALL
(*a*) AM time period											
"Before"	18	98.4	41.4	7	38.9	190.0	852.3	1042.3	40.0	296.7	336.7
"After"	34	89.2	33.7	3	8.8	194.7	774.7	969.4	55.0	313.3	368.3
Net change	—	−9.2	−7.7	—	−30.1	+4.7	−77.6	−72.9	+15.0	+16.6	+31.6
Percent change	—	−9.3	−18.6	—	−77.4	+2.5	−9.1	−7.0	+37.5	+5.6	+9.4
Sig. level	—	0.05	0.05	—	0.05	NS	NS	NS	NS	NS	NS
(*b*) PM time period											
"Before"	14	100.1	41.1	5	35.7	522.3	1036.0	1558.3	304.0	664.0	(16.6)[a] 968.0
"After"	32	107.3	46.0	10	31.3	512.3	982.6	1494.9	204.0	480.7	(11.9)[a] 684.7
Net change	—	+7.2	+4.9	—	−4.4	−10.0	−53.4	−63.4	−100.0	−183.3	(−4.7)[a] −283.3
Percent change	—	+7.2	+11.9	—	−12.3	−1.9	−5.2	−4.1	−32.9	−27.6	(−28.3)[a] −29.3
Sig. level	—	NS	NS	—	NS	NS	NS	NS	0.05	0.05	0.05

NS = not significant at $\alpha \leq 0.05$.
[a] = Per cycle. Left-turn vehicles not included.

For the 4:30 to 5:30 PM time period, a comparison of the "before" and "after" mean values indicated the following to be statistically significant:

1. A decrease of 1.6 vehicles stopped per cycle (33.3 percent) on the eastbound Market Street approach.
2. A decrease of 3.9 vehicles stopped per cycle (28.1 percent) on the westbound Market Street approach.
3. A decrease of 4.7 vehicles stopped per cycle (28.3 percent) on the northbound Washington Street approach.

While some of the decreases in stopped vehicles for westbound traffic probably resulted from the elimination of bus passenger service operations in the second lane, the total reduction in stopped vehicles of 29.1 percent experienced for all approaches (Table G-45) reflects the improved intersection operations.

Although it was not included in the volumes analyzed, a decrease in the number of northbound left-turning vehicles stopped of 2.5 per cycle (32.9 percent) was also observed.

TABLE G-45

MEANS OF VOLUME SURVEYS

CONDITION	VEHICLES THROUGH: CARS	TRUCKS AND BUSES	TOTAL	TOTAL EQUIVALENT CARS [a]	VEHICLES STOPPED: ALL
	(*a*) AM period				
"Before"	2622.3	390.3	3012.6	3246.8	830.4
"After"	2500.7	417.7	2918.4	3169.0	979.6
Net change	−121.6	+27.4	−94.2	−77.8	+149.2
Percent change	−4.6	+7.0	−3.1	−2.4	+18.0
	(*b*) PM period				
"Before"	2833.3	355.2	3188.5	3401.6	1715.2
"After"	2750.7	362.7	3113.4	3331.0	1216.7
Net change	−82.6	+7.5	−75.1	−70.6	−498.5
Percent change	−2.9	+2.1	−2.4	−2.1	−29.1

[a] One truck or bus = 1.6 cars.

Conclusions

The relocation of pedestrian crosswalks and the addition of a double-clearance interval for eastbound and westbound traffic at the intersection of Market Street with Washington Street resulted in safer and more efficient operation.

In the AM time period, northbound traffic experienced reductions of 0.5 min per mile in travel time and 0.4 min per mile in delay time. During the PM time period, the number of vehicles stopped was significantly reduced by 29.1 percent for all approaches combined, with each approach experiencing reductions.

Channelization and Lane Control Devices on Jefferson Street and Ninth Street—Experiment D8

Experiment D8 investigates changes in channelization, lane markings, and signing on Jefferson Street and Ninth Street in Louisville. The effectiveness of these devices in the control of traffic was determined for wrong-way movements on Jefferson Street at its intersection with Ninth Street and for lane use of vehicles on Ninth Street at Liberty Street. Jefferson Street is a one-way westbound street between Baxter Avenue and Ninth Street, and a two-way street between Ninth Street and 30th Street (Fig. G-93). Liberty Street is a one-way eastbound street between Ninth Street and Baxter Avenue, forming a one-way pair with Jefferson Street. Liberty Street is designated as U.S. 60-Truck, and serves as a major access route for truck movements into and through the Louisville CBD. Ninth Street is one-way in a southbound direction and is designated as U.S. 60-Truck between Jefferson Street and Liberty Street.

Experimental Area

Jefferson Street, west of its intersection with Ninth Street, is 60 ft wide; parking is regulated at both curbs by meters (Fig. G-94). All eastbound traffic must turn right into southbound Ninth Street. This movement is regulated by a traffic signal at the intersection and a sign reading RIGHT TURN ON RED WHEN CLEAR. Jefferson Street, east of this intersection, is also 60 ft wide, with parking meters on both curbs. Ninth Street, between Jefferson Street and Liberty Street, is 36 ft wide, with stopping not permitted adjacent to the west curb and no signed restriction for the east curb.

The traffic signal at Jefferson Street and Ninth Street is part of the progressive systems on these streets. The signal is set so that 57 percent of the cycle time is allotted to Ninth Street. There is no traffic signal at Ninth Street and Liberty Street.

Traffic volumes for the morning and afternoon peak hours of an average weekday (AWDT) in 1968 are shown in Figure G-95.

Design of Experiment

Observations of traffic approaching Ninth Street from both directions of Jefferson Street indicated that many vehicles were traveling in the wrong direction east of Ninth Street or on the wrong side of the road west of Ninth Street. This dangerous condition is often encountered where a one-way roadway becomes two-way. Observations of traffic on Ninth Street approaching Liberty Street indicated that most vehicles were using the west curb lane or were straddling the two most westerly lanes.

Changes to improve these conditions were made in two phases. Phase I was installed on October 16, 1968, and consisted of the pavement markings and curb-mounted signs shown in Figure G-94. Phase II was completed on November 2, 1968, and included erection of the overhead signs on Ninth Street and both approaches to Jefferson Street.

The following data groups of measurements were taken for at least three different days:

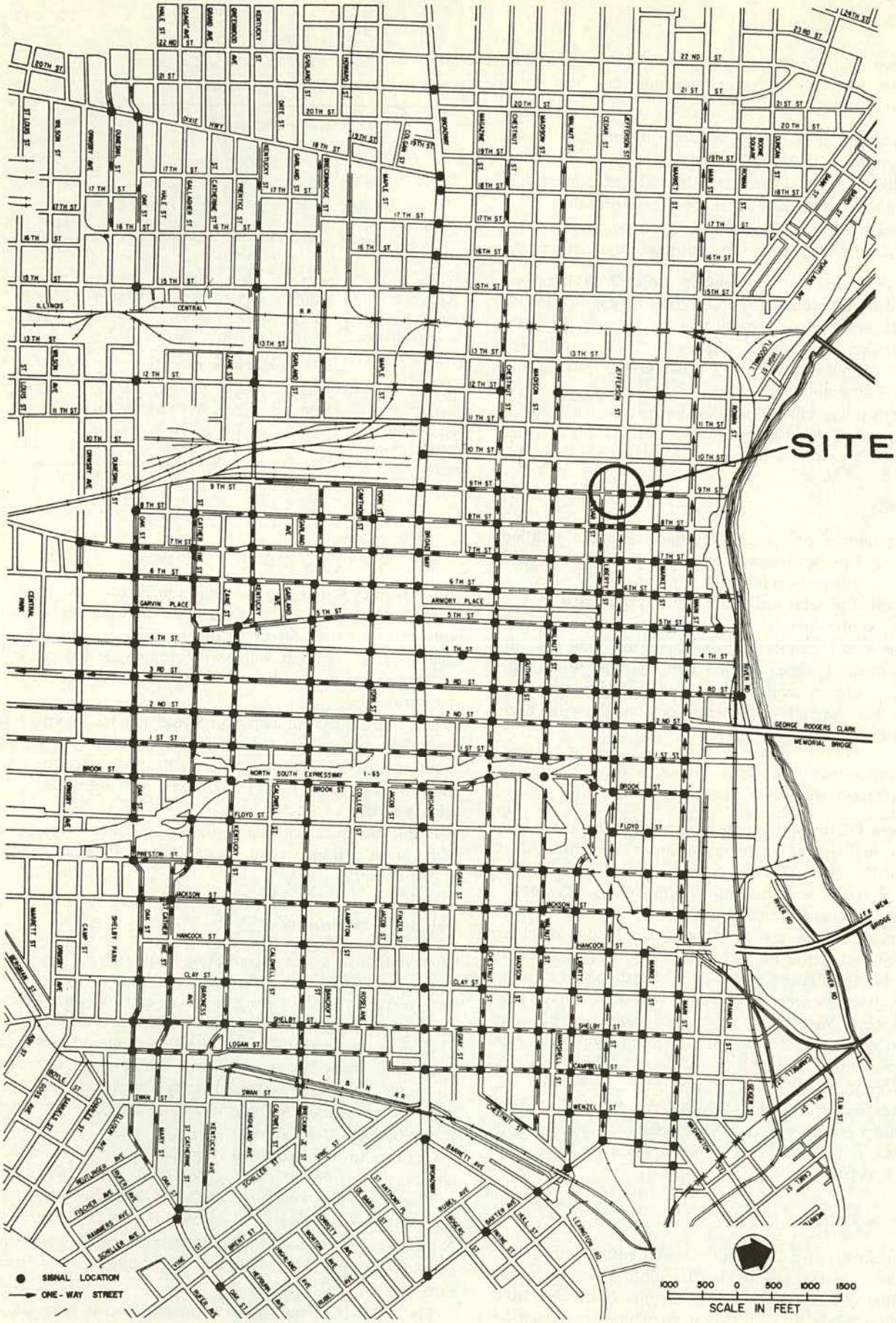


Figure G-93. Location map, Experiment D8.

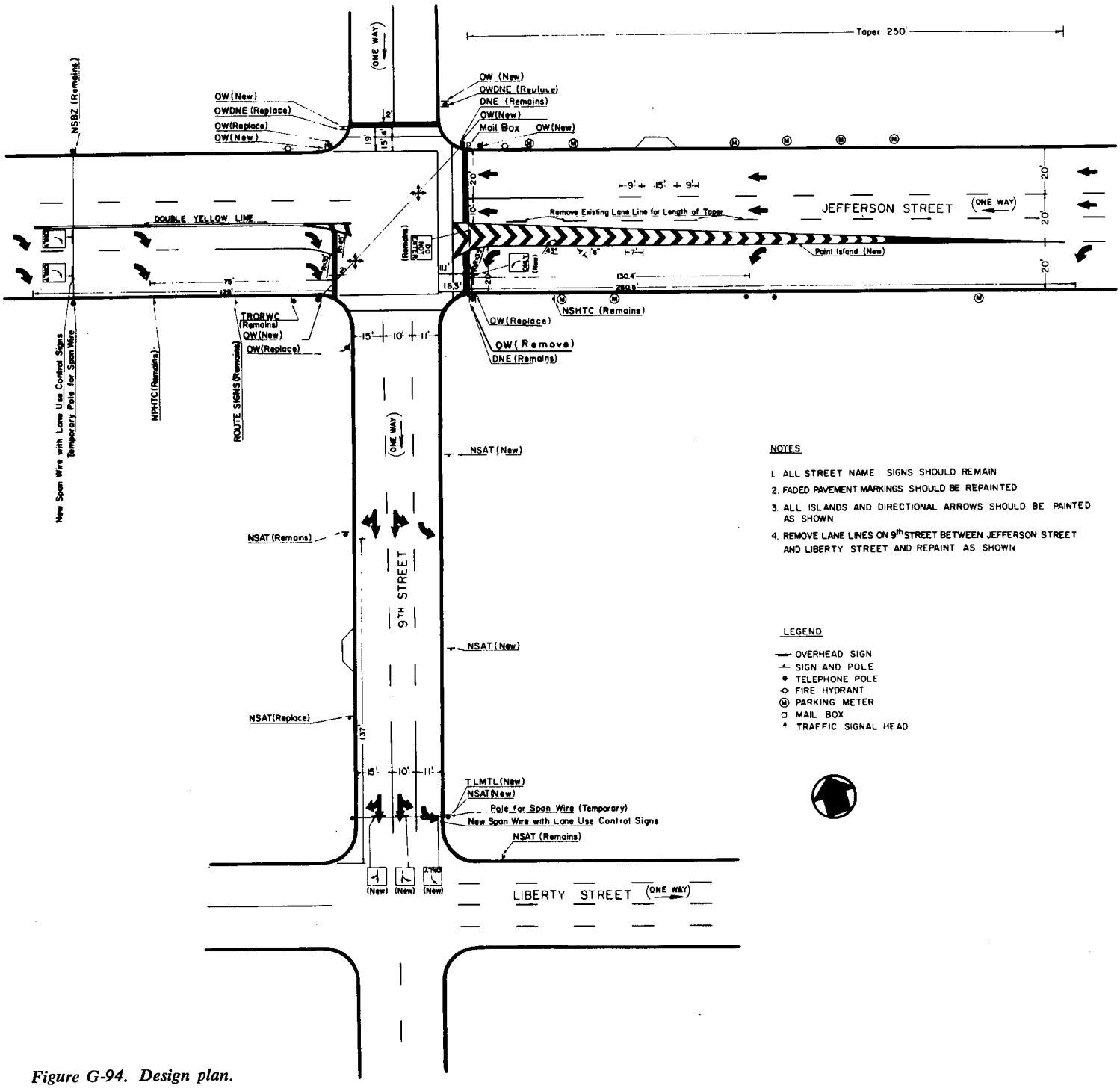


Figure G-94. Design plan.

DATA GROUP	DATES COUNTED	PHASES
1	9/16/68 through 9/20/68, and 9/24/68	Before Phase I
2	10/23/68 through 10/25/68, and 10/28/68	Between Phases I and II
3	11/13/68, 11/15/68, and 11/18/68	After Phase II

The measurements included:

1. The travel time, in seconds, for automobiles, light trucks, and heavy trucks to proceed from line A to line C, line A to line D, line B to line C, and line B to line D, as shown in Figure G-95. These were recorded manually by stopwatch for the morning peak hour from 7:30 to 9:00 AM, the midday period of 9:00 AM to 4:00 PM, and the evening peak period from 4:30 to 5:30 PM. The vehicle types were defined as follows:

TABLE G-46

MEAN TRAVEL TIME

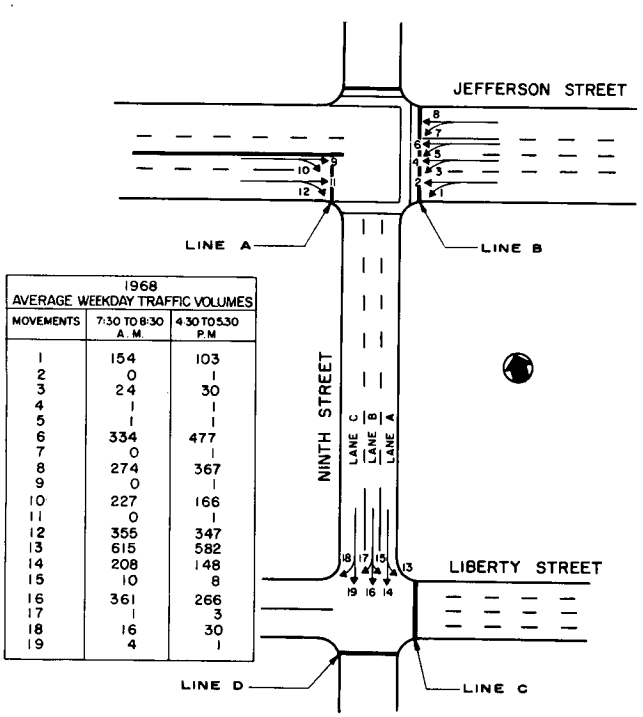
MOVEMENT	AUTOMOBILES MEAN TIME (SEC)	AUTOMOBILES NO. OF OBS.	LIGHT TRUCKS MEAN TIME (SEC)	LIGHT TRUCKS NO. OF OBS.	HEAVY TRUCKS MEAN TIME (SEC)	HEAVY TRUCKS NO. OF OBS.
A to C	13.4	1410	15.3	380	17.9	416
A to D	11.2	309	13.5	100	16.5	36
B to C	13.3	375	15.1	39	15.1	7
B to D	11.6	822	12.6	109	14.8	17

TABLE G-47

PERCENTAGE INCREASE OF TRUCK TRAVEL TIME OVER AUTOMOBILES

MOVEMENT	INCREASE OF TRAVEL TIME (%) LIGHT TRUCKS	INCREASE OF TRAVEL TIME (%) HEAVY TRUCKS
A to C	14.2	33.6
A to D	20.5	47.3
B to C	13.5	13.5
B to D	8.6	27.6

a. Automobile: any vehicle with two axles and a total of four tires.
b. Light trucks: any vehicle with two or three axles and a total of six tires.



1968 AVERAGE WEEKDAY TRAFFIC VOLUMES

MOVEMENTS	7:30 TO 8:30 A.M.	4:30 TO 5:30 P.M.
1	154	103
2	0	1
3	24	30
4	1	1
5	1	1
6	334	477
7	0	1
8	274	367
9	0	1
10	227	166
11	0	1
12	355	347
13	615	582
14	208	148
15	10	8
16	361	266
17	1	3
18	16	30
19	4	1

Figure G-95. Traffic measurements.

c. Heavy trucks: multiaxled vehicles with more than six tires.

2. The number of vehicles stopped and the vehicular movements (Fig. G-95) by lane on both Jefferson Street approaches were recorded by cycle. The vehicular movements on Ninth Street as shown in Figure G-95 were also recorded. These measurements were taken from 7:30 to 8:30 AM, 1:30 to 2:30 PM, and 4:30 to 5:30 PM.

Analysis

Comparison of the travel times between data groups for each vehicle type indicated no significant difference between data groups but indicated a significant difference between vehicle types (Table G-46).

As would be expected, travel times for trucks are greater than for autos. Table G-47 gives the percentage of increase in travel time of trucks over automobiles.

The variabilities of observations within each time period for each movement were compared between data groups, with no significant differences being detected.

Vehicular movement data were summarized for 3 hr per day and for three days for each data group, providing data for 9 hr, or 540 traffic-signal cycles. Table G-48 gives the number of vehicles per lane on Ninth Street for each data group.

A chi square analysis of the row totals for each data group indicated a significant increase in volume (4.7 percent) from data group 1 to data group 3. Table G-49 gives the lane distributions in percentage form. As indicated, there was a significant change in lane distribution from data group 1 to 2. Use of lane A decreased, whereas use of lanes B and C increased.

Table G-50 gives the individual vehicular movements shown in Figure G-95 for each data group. Of particular importance are the lane shifts that have taken place for various movements. In data group 1 only 2 percent of the left-turning vehicles made movement 15, contrasted with 23 percent in data group 3. Straight-traveling vehicles shifted from movement 14 to 19. This latter change was anticipated, because movement 14 is illegal under the improved system.

The hazardous wrong-way movements on Jefferson Street are shown in Figure G-95 as movements 2 and 4 for westbound traffic and movements 9 and 11 for eastbound traffic. Vehicles recorded making these movements are given in

TABLE G-48

NUMBER OF VEHICLES ON NINTH STREET

DATA GROUP	VEHICLES, BY LANE A	B	C	TOTAL
1	5627	2289	202	8118
2	3535	2864	1928	8327
3	3500	3036	1965	8501

TABLE G-49

LANE DISTRIBUTION OF VEHICLES ON NINTH STREET

DATA GROUP	DISTRIBUTION OF VEHICLES (%), BY LANE A	B	C	ALL
1	69.3	28.2	2.5	100.0
2	42.5	34.4	23.1	100.0
3	41.2	35.7	23.1	100.0

Tables G-51 and G-52. They indicate a significant decrease in these movements due to the improvements.

The right-turn movements from the eastbound Jefferson Street approach were given further analysis to determine the effect of the improvements on (1) observance of the RIGHT TURN ON RED WHEN CLEAR sign at this approach and (2) lane distribution. The number of vehicles turning right during each data group is given in Table G-52. As indicated, the number of right-turning vehicles was the same for each data group.

The number of vehicles turning during the red and green phases is given in Table G-53. It shows a significant reduction in the number of vehicles turning during the red signal indication and a corresponding increase in the number of vehicles turning during the green signal indication, due to the improvements. There was no significant change in lane distribution.

The number of vehicles stopped per cycle and the number of vehicles through per cycle were analyzed for both Jefferson Street approaches to Ninth Street. Inasmuch as counts were made on a Monday, Wednesday, and Friday for all data groups, only these measurements were used in this analysis. An ANOVA of the measurements for data group 1 indicated that Friday was different from Monday and Wednesday. Therefore, a factor of days was included in all subsequent analysis, with Mondays and Wednesdays being combined and Friday being analyzed separately. No significant difference was determined for the number of vehicles through per cycle between data groups for either approach (Table G-54).

The means for the number of vehicles stopped per cycle are given in Table G-54. Significant reductions in the number of vehicles stopped on the eastbound approach were determined for both days during the AM period and for Friday during the PM period. In many cases a significant difference was determined between data groups 1 and 2, with no difference between data groups 1 and 3. This condition may be due to the longer "shakedown" period between data groups 1 and 3 and/or the regulatory effect of the overhead signs that were installed during Phase II.

TABLE G-50

VEHICULAR MOVEMENTS ON NINTH STREET

DATA GROUP	VEHICLES, BY MOVEMENT LEFT TURN 13	15	STRAIGHT 14	16	19	RIGHT TURN 17	18	ALL
1	4221	106	1406	2163	20	20	182	8118
2	3449	965	86	1899	1742	0	186	8327
3	3433	1024	67	2012	1729	0	236	8501

TABLE G-51

WRONG-WAY MOVEMENTS ON WESTBOUND JEFFERSON STREET APPROACH

DATA GROUP	VEHICLES, BY MOVEMENT WRONG WAY 2	4	LEFT TURN 1 & 3	ALL
1	2	14	1162	1178
2	0	0	1052	1052
3	1	0	1126	1127

TABLE G-52

WRONG-WAY MOVEMENTS ON EASTBOUND JEFFERSON STREET APPROACH

DATA GROUP	VEHICLES, BY MOVEMENT WRONG WAY 9	11	RIGHT TURN 10 & 12	ALL
1	5	4	4607	4616
2	0	1	4662	4663
3	1	1	4648	4650

Conclusions

Based on the foregoing analysis, it is concluded that:

1. Travel time did not change due to improvements.
2. Significant changes in lane distribution were made with pavement markings and lane control signs.
3. The major change in lane distribution occurred before installation of the lane control signs.
4. The hazardous wrong-way movements on Jefferson Street were reduced by 78 percent on the eastbound approach and by 94 percent on the westbound approach.

TABLE G-53

RIGHT-TURN MOVEMENTS FROM EASTBOUND JEFFERSON STREET APPROACH

	VEHICLES, BY MOVEMENT						
	10			12			
DATA GROUP	DURING GREEN	DURING RED	SUB-TOTAL	DURING GREEN	DURING RED	SUB-TOTAL	ALL
1	1151	311	1462	2510	635	3145	4607
2	1312	184	1496	2765	401	3166	4662
3	1341	98	1439	2882	327	3209	4648

TABLE G-54

VEHICLES THROUGH AND STOPPED PER CYCLE ON JEFFERSON STREET

	EB			WB		
DATA GROUP	AM	MIDDAY	PM	AM	MIDDAY	PM
	(*a*) Vehicles through per cycle					
1	10.4	5.8	9.6	11.3	8.7	15.7
2	11.1	6.2	9.0	12.4	9.6	15.5
3	10.6	6.0	9.0	13.1	8.9	15.9

	(*b*) Vehicles stopped per cycle											
	MON. & WED.	FRI.	MON. & WED.	FRI.	MON. & WED.	FRI.	MON. & WED.	FRI.	MON. & WED.	FRI.	MON. & WED.	FRI.
1	3.7	5.7	0.4	1.8	0.7	2.3	1.6	1.6	0.7	1.1	1.7	2.0
2	1.8	3.9	1.3	1.7	0.8	3.0	1.5	1.1	1.3	2.3	3.5	3.3
3	0.6	2.7	0.5	1.0	1.1	1.2	1.2	1.3	1.3	1.5	2.5	1.9

5. The number of vehicles turning right during the red interval decreased by 55 percent on the eastbound Jefferson Street approach.

Channelization at St. Catherine Street and Floyd Street—Experiment D10

Channelization has been a useful method for organization of traffic flows through complex areas by separating or controlling conflicting movements.

Experiment D10 deals with the use of a channelizing island to eliminate certain conflicting movements at the intersection of St. Catherine and Floyd Streets in Louisville. The experiment is concerned with the westbound St. Catherine Street approach where an off-ramp from I-65 Expressway joins St. Catherine Street 100 ft east of the intersection. The channelizing island and turn prohibitions were used to eliminate weaving of off-ramp traffic across the three lanes of traffic on St. Catherine Street within the short distance to the Floyd Street intersection. Ramp vehicles attempting this weave often waited for an opportunity at the nose of the off-ramp, causing sudden stops and backups on the ramp.

Experimental Area

Experiment D10 was concentrated at the intersection of Floyd Street and St. Catherine Street (Fig. G-96). Because the traffic signal timing was not changed, and the possible effect of the chanelizing island and turn prohibitions on network movements was assumed to be minimal, no additional intersections were monitored.

St. Catherine Street is an arterial street permitting one-way travel in a westbound direction. It is 36 ft wide east of the off-ramp junction, 46 ft wide between the ramp junction and Floyd Street, tapers from 46 ft to 42 ft for a distance of 150 ft west of the intersection, and then continues west at the 42-ft width. Stopping is not permitted at any time on St. Catherine Street for a distance of 190 ft on the south side and a distance of 103 ft on the north side, east of the intersection. A near-side bus stop is located on the north side of St. Catherine Street east of the intersection. Floyd Street is a 42-ft-wide, two-way street with parking permitted on both sides. A two-dial controller operates the traffic signal at this intersection on a 60-sec cycle, with 58 percent of the cycle for green time for St. Catherine Street and 33 percent of the cycle for green time for Floyd Street. The timing is approximately the

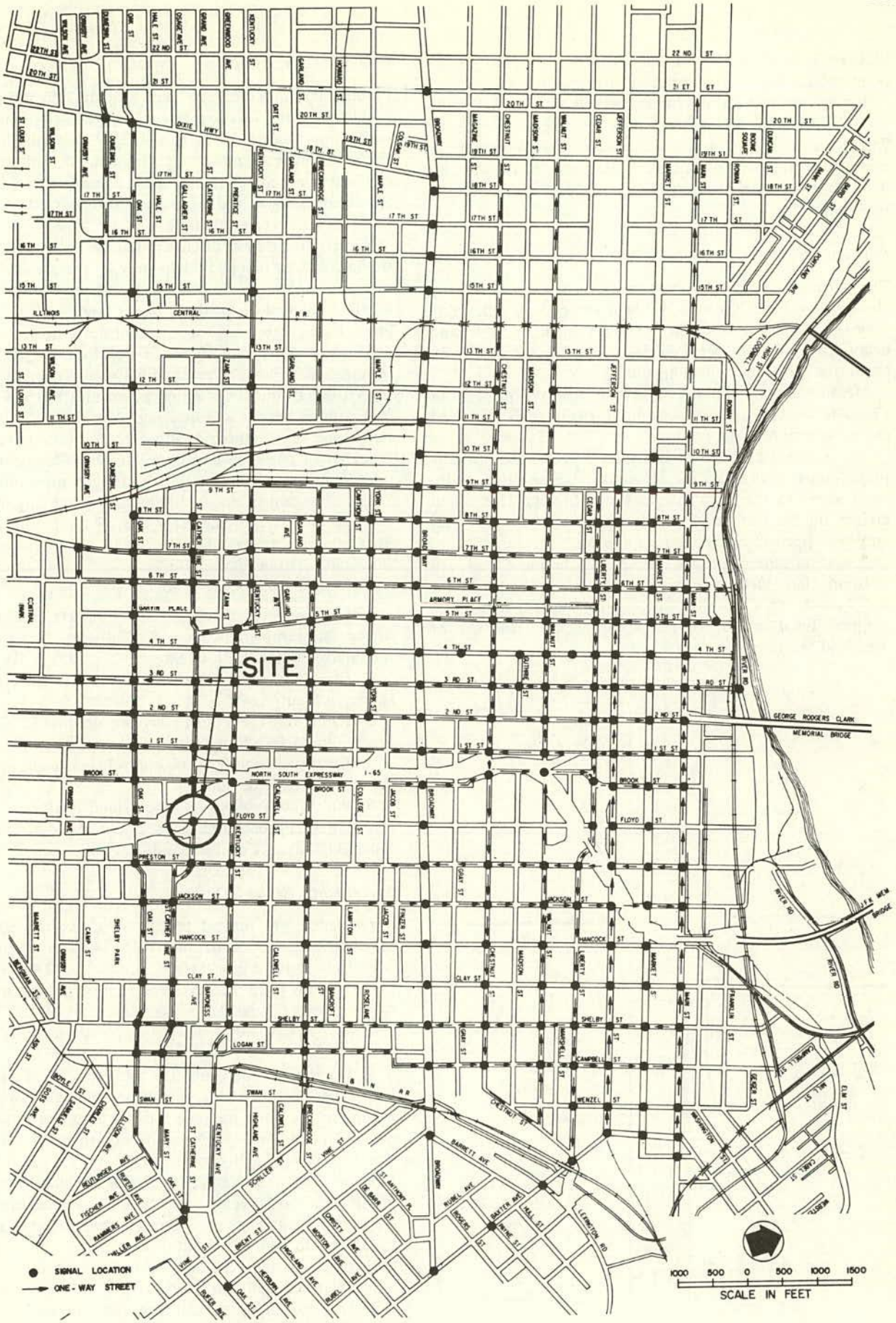


Figure G-96. Location map, Experiment D10.

same on both dials. The St. Catherine Street signal system is interconnected and operates in progression.

Traffic volumes during August 1968 on St. Catherine Street east of the intersection were recorded to be about 750 vehicles during both the morning and evening peak hours. The ramp volumes during August were recorded to be 450 vehicles in the morning and 300 in the evening peak hour.

Purpose and Scope

The purpose of this experiment was to determine any change in vehicles through, vehicles stopped on red, delay time on red, delay time on red per vehicle through, and delay time on red per vehicle stopped on red for St. Catherine Street and off-ramp traffic.

Measurements were taken during the AM peak hour (7:30 to 8:30), the PM peak hour (4:30 to 5:30), and during several off-peak hours.

The scope of work included a narrow channelizing island constructed from the off-ramp junction at St. Catherine Street to the crosswalk at Floyd Street. This island divides the St. Catherine Street approach to Floyd Street into two approaches: a one-lane approach for the off-ramp and a three-lane approach for St. Catherine Street. In addition, left turns are prohibited at Floyd Street for traffic on the St. Catherine Street approach, and right turns are prohibited on the off-ramp approach by the use of overhead and roadside signs.

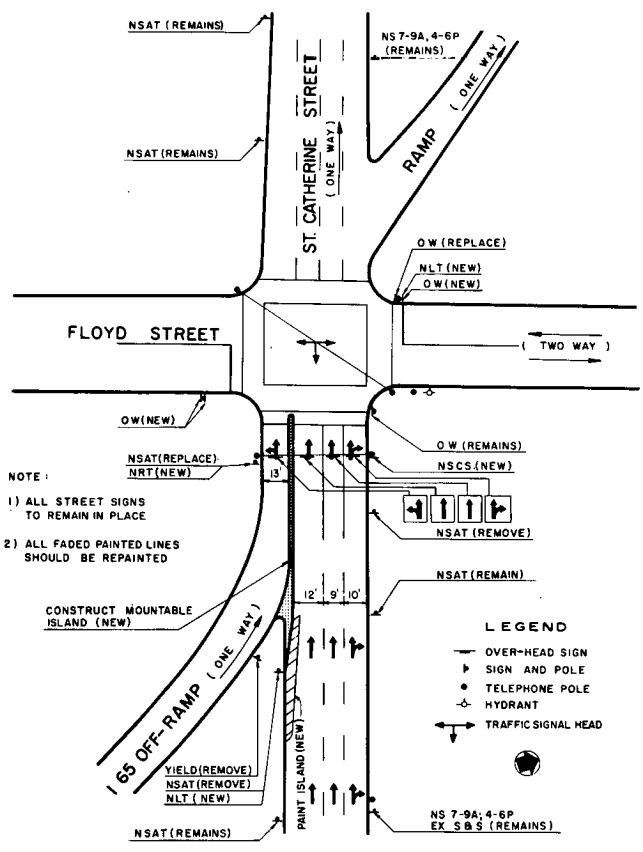


Figure G-97. Design plan.

Design of Experiment

Preliminary observations had brought attention to the problem at this location caused by cars stopping on the ramp and waiting for a gap in the St. Catherine Street traffic before merging or weaving into that flow, and by conflicts between right turns from the off-ramp into Floyd Street northbound and left turns from St. Catherine Street into Floyd Street southbound.

A capacity analysis disclosed that the volume of off-ramp traffic could be handled adequately by one travel lane, and that the volume of St. Catherine Street traffic could be handled adequately by three travel lanes with the existing green time. The peak-hour turning movement counts also disclosed that between 10 and 20 vph turned right from the off-ramp into Floyd Street, and between 20 and 30 vehicles turned left from St. Catherine Street into Floyd Street.

An improvement was designed that channelized the off-ramp and St. Catherine Street traffic into separate approaches of one lane and three lanes at Floyd Street, and prohibited the conflicting left and right turns into Floyd Street. The improvement eliminated the merging maneuver and the yield requirements at the off-ramp junction with St. Catherine Street. The design of the improvement is shown in detail in Figure G-97.

The improvements are as follows:

1. Construct a channelizing island on St. Catherine Street to separate ramp traffic from St. Catherine Street traffic at the intersection with Floyd Street.
2. Erect NO LEFT TURN and NO RIGHT TURN signs, as shown in Figure G-97.
3. Paint lane-use control arrows on the St. Catherine Street approach to Floyd Street.
4. Erect overhead lane-use control signs using span wire. Arrows will indicate lane use.
5. Paint both edges of the island yellow. Diagonal obstruction approach markings should be painted white and tapered in advance of the island.

Surveillance System Design

Surveillance was limited to the St. Catherine Street and off-ramp approaches, because there were no changes that would affect the Floyd Street approaches. Four measurements of flow were recorded by signal cycle for comparison of the "before" and "after" conditions: vehicles stopped, vehicles through, total delay time, and queueing on the ramp.

It was desired to record the data for the off-ramp and St. Catherine Street approaches separately; however, the mixing of the flows between the off-ramp and Floyd Street during the "before" phase of the experiment made tabulation of the data by approach origin very difficult. Therefore, no attempt was made to separate traffic by approach origin. Two observers were stationed at the intersection stop line on St. Catherine Street. One observer recorded traffic in the two left (south) lanes, including any traffic backed up on the off-ramp behind the ramp nose. The other recorded traffic in the two right (north) lanes.

The approach-leg arrival rate was measured by two observers stationed near the nose of the off-ramp to provide

data by approach origin. One observer recorded arrivals at an imaginary line (the arrival rate line) located across the off-ramp approach before its junction with St. Catherine Street. The other observer recorded arrival at a similar line located perpendicular to St. Catherine Street at its junction with the off-ramp. The same lines were used in both "before" and "after" measurements. Observers recorded arrivals at the respective arrival rate lines unless a backup went past the line, in which case observers recorded arrivals at the end of the queue in the usual manner.

The "before" measurements were scheduled for August 1, 2, and 5, 1968, from 7:30 to 9:30 AM, 10 to 11:30 AM, 1:30 to 3 PM, and 3:30 to 5:30 PM. The "after" measurements were scheduled for August 22, 23, and 26, 1968, during the same hours as the "before" measurements. ATR counters recorded the number of vehicles on each approach leg by 15-min intervals for a 24-hr period on July 31, 1968, and again for a 24-hr period on August 22, 1968.

Implementation of Improvement

The improvement was implemented during the period from August 19 to August 22, 1968. Initially, cones were placed to mark the location of the island, and the necessary signs were installed. On August 21 the mountable island was installed, and on August 22 the improvement was completed when the edges of the island were painted yellow. The construction was in accordance with the plan shown in Figure G-97. The cost of the improvement was as follows:

ITEM	COST ($)
Island	220
Span wire and signs	155
Pavement markings, signs, etc.	240
	615
25% for engineering and supervision (city personnel)	154
Total	769

Because the basic consequence of the improvement on traffic flow was effective with the first day of implementation, and because data collected on Thursday, August 22, were not used in analysis due to implementation still being under way that day, there were, in effect, four weekdays for traffic to adjust to the improvement. The field crew reported considerably fewer illegal turns on August 23, the first day for which data were analyzed, than on the previous day. The shakedown period is believed to have been adequate. It was imperative that all "after" measurements be completed before Labor Day, because the change in season would probably result in a change in traffic patterns.

Surveys

Four men were assigned to the field crew to collect data. They carried out their assignment as planned, except that the arrivals on the end of a queue backed up on the ramp beyond the arrival rate line were not recorded until those vehicles actually passed this line.

Surveys on August 1 and 2 were interrupted by bad weather and other circumstances beyond the researchers' control, necessitating discontinuance of survey work at 8:30 AM on August 1, and 12 noon on August 2. This left only one day of complete afternoon data in the "before" measurements, August 5, when the measurements were taken without incident.

The first "after" measurements were taken on Thursday, August 22, 1968. Because the island was painted during mid-morning on August 22, 1968, an additional day of measurements was taken on the following Thursday, August 29. The measurements were also taken as planned on Friday, August 23, and Monday, August 26.

ATR counts were taken for a minimum of 24 hr on all approaches to the intersection as planned and without incident. Volume counts derived from survey data showed that peak-hour traffic on St. Catherine Street was higher on the last day of observations, August 29, than on previous days. This may indicate that volumes were low on St. Catherine Street for most of the time of the experiment due to vacations.

First Level Analysis

The purpose of the First Level Analysis is to evaluate the significance of local changes in traffic flow caused by the improvement. This was accomplished through the statistical comparison of "before" and "after" measurements of certain quantities that were judged to be indicators of traffic flow improvement. The following quantities were studied for both off-ramp and St. Catherine Street approaches:

1. Vehicles stopped on red per cycle.
2. Vehicles through per cycle.
3. Total delay time on red per cycle.
4. Backup on the ramp per cycle.
5. Delay time on red divided by vehicles stopped on red per cycle.
6. Delay time on red divided by vehicles through per cycle.

The analysis was performed for the peak-hour volumes recorded during the periods from 7:30 to 8:30 AM and from 4:30 to 5:30 PM. The off-peak hours were not analyzed, because inspection indicated very little delay at this location during off-peak hours. Analysis was performed for the off-ramp and St. Catherine Street approaches for data recorded on the following dates (1968):

DAY	TIME	DAY CODE	"BEFORE"	"AFTER"
Monday	AM	3	Aug. 5	Aug. 26
Monday	PM	3	Aug. 5	Aug. 26
Thursday	AM	1	Aug. 1	Aug. 29
Friday	AM	2	Aug. 2	Aug. 23

Summary of Field Data.—The field sheets were summarized by cycle in preparation for the First Level Analysis. The number of vehicles through and number of vehicles stopped on red were totaled and recorded. The total vehicles arriving per cycle and the number arriving during the red interval, with adjustments being made for the distance from the stop line, were also totaled.

The delay time on red by cycle was calculated from the number of arrivals during each 3-sec period, multiplied by the average time these vehicles were stopped. If vehicles were stopped longer than the initial red interval, the delay on green was added to the delay on red to give the total delay time for each cycle. Backup on the ramp was recorded directly on the field sheets during the "before" measurements. For the purpose of calculating ramp backup during the "after" measurements, the fifth vehicle and those behind it in the queue on the off-ramp lane were considered to be ramp backup. The ratios of delay time on red per vehicle through and delay time on red per vehicle stopped on red were calculated for each respective cycle.

Statistical Analysis of Data.—Because the amounts of data that were recorded were not the same for the morning and afternoon peak hours, the methods of analysis were different. The morning peak-hour data were analyzed for the most part by the ANOVA technique; the median or t tests were performed on the afternoon peak-hour data.

Analysis of AM Data.—The greatest traffic volumes during the day occurred in the morning peak hour. Individual observations of all variables recorded in the morning were plotted. Several "outliers" were eliminated from the data, and Bartlett's test and the appropriate ANOVA were performed on remaining values. The results of these tests are given in Table G-55. As indicated, only the variable "vehicles through" on the St. Catherine Street approach exhibited a sufficiently low coefficient of variation and did not reject the null hypothesis of equal variance. The ANOVA for this variable indicated that none of the three factors—"before" and "after," days, and interaction—was significant at the $\alpha = 0.05$ level.

All variables except "vehicles through" on the St. Catherine Street approach were transformed and subjected to Bartlett's test and ANOVA. The transform took the form of $10\sqrt{x+1}$. Table G-56 gives the results of ANOVA on transformed data. Based on these tests, the ANOVA for the following variables were accepted:

APPROACH	VARIABLE
Ramp	1. Vehicles stopped on red
	2. Vehicles through
	3. Delay time on red
	4. Delay time on red per vehicle through
St. Catherine St.	1. Vehicles stopped on red
	2. Delay time on red per vehicle stopped on red

F ratios were determined for the original "before" and "after" measurements, by day, for each of the variables that did not have equal variances. These are given in Table G-57.

The F ratios were compared with critical values of the F distribution at the $\alpha = 0.05$ level. Those ratios, which were greater than the critical values, have been indicated

TABLE G-55

BARTLETT'S TEST AND ANOVA, ORIGINAL AM DATA, ST. CATHERINE STREET AND FLOYD STREET

APPROACH	VARIABLE	NO. OF OBS.	EQUAL VARIANCE	COEF. OF VARIATION	STANDARD DEVIATION	GRAND MEAN	SIG. @ $\alpha =$ 0.05 LEVEL
Ramp	Vehicles stopped on red	304	No	71.24	2.25	3.2	NA
	Vehicles through	304	Yes	40.57	2.96	7.3	NA
	Delay time on red	301	No	80.41	28.24	35.1	NA
	Delay time on red/vehicle stopped on red	282	No	38.07	4.31	11.3	NA
	Delay time on red/vehicle through	301	No	70.30	3.31	4.7	NA
St. Catherine St.	Vehicles stopped on red	305	No	138.22	0.94	0.7	NA
	Vehicles through	305	Yes	37.91	4.81	12.7	No
	Delay time on red	302	No	160.36	11.67	7.3	NA
	Delay time on red/vehicle stopped on red	139	Yes	57.70	6.49	11.3	NA
	Delay time on red/vehicle through	300	No	168.51	0.99	0.6	NA

NA = not applicable.

TABLE G-56

BARTLETT'S TEST AND ANOVA, TRANSFORMED AM DATA, ST. CATHERINE STREET AND FLOYD STREET

APPROACH	VARIABLE	NO. OF OBS.	EQUAL VARIANCE	COEF. OF VARIATION	STANDARD DEVIATION	GRAND MEAN	SIG. @ $\alpha=0.05$ LEVEL
Ramp	Vehicles stopped on red	304	Yes	26.48	5.20	19.6	"Before" and "after"
	Vehicles through	304	Yes	18.72	5.29	28.3	No
	Delay time on red (sec)	301	Yes	41.78	23.05	55.2	"Before" and "after"
	Delay time on red/vehicle stopped on red	282	No	19.38	6.67	34.4	N/A
	Delay time on red/vehicle through	301	Yes	29.58	6.75	22.8	"Before" and "after"
St. Catherine St.	Vehicles stopped on red	305	Yes	25.98	3.26	12.6	No
	Delay time on red (sec)	302	No	75.36	17.29	22.9	NA
	Delay time on red/vehicle stopped on red	139	Yes	29.47	9.90	33.6	No
	Delay time on red/vehicle through	300	No	26.86	3.26	12.1	NA

NA = not applicable.

as being significant at the $\alpha = 0.05$ level in Table G-57, the results indicating that the null hypothesis of equal variance has been rejected. In all cases where the null hypothesis was rejected, the "before" variance was higher than the "after" variance.

A median test was performed on data that had unequal variance between the "before" and "after" conditions; t tests were performed on the means of the individual days that exhibited equal variance (Table G-58).

The tests indicate a significant reduction in delay time on the ramp approach between the "before" and "after" conditions, and no significant change in any variable on the St. Catherine Street approach. The results of the median test were not reported as a net difference between the mean values of the "before" and "after" conditions because the median test tested the null hypothesis that the number of observations in the "before" and "after" conditions above and below the median are independent; hence, the samples came from the same population. A rejection of the null hypothesis indicates the conditions to be dependent.

Table G-59 is a summary of the results of statistical tests that were performed on the three days of "before" and "after" data recorded during the morning.

Generally, the results indicate that the number of vehicles stopped on red, the delay time on red, the delay time on red per vehicle stopped on red, and the delay time on red per vehicle through showed significant reductions between the "before" and "after" conditions for the ramp approach. The number of vehicles through on the ramp and all variables for the St. Catherine Street approach showed no significant change between the "before" and "after" conditions. The results for the ramp variables that showed significant reductions would show greater reduc-

TABLE G-57

F RATIOS, AM DATA, ST. CATHERINE STREET AND FLOYD STREET

APPROACH	VARIABLE	DAY	NO. OF OBS. "BEFORE"	NO. OF OBS. "AFTER"	F RATIO	SIG. @ $\alpha=0.05$ LEVEL
Ramp	Delay time per vehicle stopped	1	50	54	1.70	Yes
		2	44	54	2.11	Yes
		3	52	28	1.94	Yes
St. Catherine St.	Delay time on red	1	52	60	1.41	No
		2	43	59	1.66	Yes
		3	58	30	3.50	Yes
	Delay time per vehicle through	1	52	60	1.49	No
		2	44	60	1.69	Yes
		3	54	30	1.68	No

TABLE G-58

OTHER TESTS, AM DATA, ST. CATHERINE STREET AND FLOYD STREET

APPROACH	VARIABLE	DAY	TEST USED	AVERAGE[a] "BEFORE"	AVERAGE[a] "AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$ LEVEL
Ramp	Delay time on red/vehicle stopped on red	1	Median	11.5	9.0	−2.5	Yes
		2	Median	13.0	11.5	−1.5	Yes
		3	Median	11.6	9.4	−2.2	Yes
St. Catherine St.	Delay time on red	1	t	9.0	8.5	−0.5	No
		2	Median	0.0	0.0	0.0	No
		3	Median	2.5	0.0	−2.5	No
	Delay time on red/vehicle through	1	t	0.8	0.6	−0.2	No
		2	Median	0.0	0.0	0.0	No
		3	t	0.6	0.4	−0.2	No

[a] Median values are listed where the median test was used; mean values are listed where the t test was used.

tions if tests were performed including the three extreme values that were rejected from the "before" data.

The data for ramp backup, taken in the AM surveys, were not analyzed due to the many zero observations that were recorded. However, it is evident that the maximum number of vehicles backed up has been reduced in two out of three days between the "before" and "after" condition.

Analysis of PM Data.—Because it was possible to analyze only one day of PM data, the ANOVA technique was not used. The individual observations were plotted and investigated for extreme values. The values that were identified as outliers by Dixon's test were rejected. The variances of each group of data were then computed. Table G-60 indicates the results of an F test on the "before" and "after" variances.

As indicated, the ramp variables of vehicles stopped on red, delay time on red, delay time on red per vehicle stopped on red, and delay time on red per vehicle through did not have the same variability for the "before" and "after" measurements. In fact, in all cases, the "before" variability was significantly greater than the "after" variability, except for the "before" variance of the number of vehicles through on the St. Catherine Street approach.

Differences between the means or medians of each of the "before" and "after" variables were tested and are given in Table G-61. The number of vehicles stopped, delay time on red, delay time on red per vehicle stopped, and delay time per vehicle through showed significant reductions at the $\alpha = 0.05$ level between the "before" and "after" conditions on the ramp approach. The number of vehicles through on the ramp approach and all variables for the St. Catherine Street approach did not show a significant change at the $\alpha = 0.05$ level between the "before" and "after" conditions.

TABLE G-59

SUMMARY OF TESTS ON AM DATA, ST. CATHERINE STREET AND FLOYD STREET

APPROACH	VARIABLE	UNIT	AVERAGE "BEFORE"	AVERAGE "AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$ LEVEL
Ramp	Vehicles stopped on red	Veh	3.2[a]	2.6[a]	−0.6	Yes
	Vehicles through	Veh	6.9[a]	7.1[a]	+0.2	No
	Delay time on red	Sec	35.8[a]	24.3[a]	−11.5	Yes
	Delay time on red/vehicle stopped on red	Sec/veh	11.5 to 13.0[b]	9.0 to 11.5[b]	−1.5 to −2.5	Yes
	Delay time on red/vehicle through	Sec/veh	5.0[a]	3.5[a]	−1.5	Yes
St. Catherine St.	Vehicles stopped on red	Veh	0.6[a]	0.5[a]	−0.1	No
	Vehicles through	Veh	11.9[a]	12.2[a]	+0.3	No
	Delay time on red	Sec	0.0[a] to 9.0[b]	0.0[a] to 8.5[b]	0.0 to −2.5	No
	Delay time on red/vehicle stopped on red	Sec/veh	10.6[a]	10.0[a]	−0.6	No
	Delay time on red/vehicle through	Sec/veh	0.0[a] to 0.8[b]	0.0[a] to 0.6[b]	0.0 to −0.2	No

[a] Median values.
[b] Mean values.

TABLE G-60

F RATIOS, PM DATA, ST. CATHERINE STREET AND FLOYD STREET

APPROACH	VARIABLE	NO. OF OBS. "BEFORE"	NO. OF OBS. "AFTER"	*F* RATIO	SIG. @ $\alpha=0.05$ LEVEL
Ramp	Vehicles stopped on red	56	57	3.06	Yes
	Vehicles through	57	57	1.25	No
	Delay time on red	57	57	65.94	Yes
	Delay time on red/vehicle stopped on red	51	49	4.16	Yes
	Delay time on red/vehicle through	57	55	36.10	Yes
St. Catherine St.	Vehicles stopped on red	57	53	1.15	No
	Vehicles through	57	53	1.07	Yes
	Delay time on red	57	53	1.45	No
	Delay time on red/vehicle stopped on red	46	45	1.34	No
	Delay time on red/vehicle through	57	53	1.32	No

Conclusions

The results of Experiment D10 indicate that significant reductions in the following measurements were realized for ramp traffic with the aforementioned improvements and without a substantial change in volume:

1. The number of vehicles stopped on red.
2. The delay time on red.
3. The delay time on red per vehicle stopped on red.
4. Delay time on red per vehicle through.

Table G-62 gives a summary of results.

Analysis of all variables for the St. Catherine Street approach indicated no significant differences between the "before" and "after" measurements.

Convenience and Safety

Immediately visible beneficial effects of this experiment were seen in the reduction of delays on the off-ramp so that no cars were detained by backups through more than one red signal period. Traffic was observed to move smoothly; and the danger from rear-end collisions on the off-ramp, as well as sideswipe accidents from merging and weaving maneuvers on St. Catherine Street, should be materially reduced.

Many illegal turns occurred during the first few days of the implementation period, consisting of left turns from St. Catherine Street into Floyd Street southbound and right turns from the ramp to Floyd Street northbound. The number of illegal turns has been reduced, although some were still observed on September 17, 1968, approxi-

TABLE G-61

OTHER TESTS, PM DATA, ST. CATHERINE STREET AND FLOYD STREET

APPROACH	VARIABLE	UNIT	TEST USED	AVERAGE [a] "BEFORE"	AVERAGE [a] "AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$ LEVEL
Ramp	Vehicles stopped on red	Veh	Median	2.0	1.0	−1.0	Yes
	Vehicles through	Veh	*t*	4.8	4.4	−0.4	No
	Delay time on red	Sec	Median	29.0	11.0	−14.0	Yes
	Delay time on red/vehicle stopped on red	Sec/veh	Median	13.8	9.0	−4.8	Yes
	Delay time on red/vehicle through	Sec/veh	Median	6.0	2.7	−3.3	Yes
St. Catherine St.	Vehicles stopped on red	Veh	*t*	1.6	1.7	+0.1	No
	Vehicles through	Veh	*t*	11.0	10.0	−1.0	No
	Delay time on red	Sec	*t*	17.9	17.1	−0.8	No
	Delay time on red/vehicle stopped on red	Sec/veh	*t*	11.8	10.6	−0.2	No
	Delay time on red/vehicle through	Sec/veh	*t*	1.7	1.7	0.0	No

[a] Median values listed where the median test was used; mean values listed where *t* test was used.

TABLE G-62

SUMMARY OF RESULTS, RAMP APPROACH, ST. CATHERINE STREET AND FLOYD STREET

VARIABLE	TIME PERIOD	UNIT	AVERAGE "BEFORE"	AVERAGE "AFTER"	DIFFERENCE	CHANGE (%)	SIG. @ $\alpha=0.05$
Vehicles stopped on red	AM	Veh	3.2 [a]	2.6 [a]	−0.6	−18.8	Yes
	PM	Veh	2.0 [b]	1.0 [b]	−1.0	−50.0	Yes
Vehicles through	AM	Veh	6.9 [a]	7.1 [a]	+0.2	+2.9	No
	PM	Veh	4.8 [a]	4.4 [a]	−0.4	−8.3	No
Delay time on red	AM	Sec	35.8 [a]	24.3 [a]	−11.5	−32.2	Yes
	PM	Sec	29.0 [b]	11.0 [b]	−14.0	−48.3	Yes
Delay time on red/vehicle stopped on red	AM	Sec/veh	11.5 to 13.0 [b]	9.0 to 11.5 [b]	−1.5 to −2.5	−11.5 to −21.8	Yes
	PM	Sec/veh	13.8 [b]	9.0 [b]	−4.8	−34.8	Yes
Delay time on red/vehicle through	AM	Sec/veh	5.0 [a]	3.5 [a]	−1.5	−30.0	Yes
	PM	Sec/veh	6.0 [b]	2.7 [b]	−3.3	−55.0	Yes

[a] Mean values.
[b] Median values.

mately 3½ weeks following the completion of "after" measurements.

Channelization and Lane Control Devices on Brook Street, Jacob Street, Broadway—Experiment D13

Experiment D13 studies the effect of signing and lane marking revisions on traffic congestion occurring in the vicinity of the I-65 off-ramp to Brook Street in Louisville (Fig. G-98). Both the off-ramp and Brook Street are major traffic carriers during the morning peak hours, providing for inbound trips to the downtown area.

Experimental Area

Brook Street is 42 ft wide south of Jacob Street and 52 ft wide at Jacob Street, tapering to 42 ft wide at Broadway. Parking was prohibited on the west side of Brook Street between Jacob Street and Broadway and controlled by meters on the east side. The I-65 off-ramp is 16 ft wide and, although designed for one-lane operation, frequently carries two lanes of traffic during peak traffic hours. Jacob Street crosses Brook Street in an east-west direction immediately north of the junction of Brook Street with the I-65 ramp. Jacob Street provides for two-directional traffic movement (Fig. G-99).

The traffic signals at Broadway and Brook Street are part of the interconnected PR system on Broadway. The timing is set on a 90-sec cycle and allots 44 percent of the time to Brook Street.

Traffic volumes for the morning and afternoon peak hours of an average weekday (AWDT) in 1968 are shown in Figure G-100.

Design of Experiment

Observations of off-ramp traffic during the AM peak period indicated that vehicles were backing up on Brook Street from the traffic signal at Broadway onto the off-ramp and the expressway. This dangerous condition is often encountered when a traffic signal is located near the foot of an expressway off-ramp. Queued traffic was stored in a single lane except for about a 15-min period when vehicles traveled two abreast on the 16-ft-wide ramp. The backup was caused by the following factors:

1. The number of vehicles turning left at Broadway was greater than could be accommodated by a single turning lane.
2. Vehicles on Brook Street made an illegal left turn into Jacob Street.
3. Ramp vehicles made an illegal right turn into Jacob Street.
4. Ramp vehicles wanting to make a right turn at Broadway were often stopped at the foot of the ramp before weaving with Brook Street traffic.

Changes to improve or eliminate these conditions were made in four phases. Phase 1 consisted of the following:

1. Painted islands were installed on Brook Street (Fig. G-99), with the exception that the painted island did not cross Jacob Street.
2. Pavement arrows were painted and curb-mounted signs were installed to provide optional turn lanes on Brook Street at Broadway.
3. The parking meters on the east side of Brook Street between Jacob Street and Broadway were removed. Stopping was prohibited at all times at this location.
4. The four travel lanes on Brook Street immediately south of Jacob Street were reduced to two travel lanes.

Phase 2 consisted of the erection of overhead lane control signs over Brook Street.

Phase 3 involved no-stopping restrictions at the following two locations:

1. North side of Jacob Street from its intersection with Brook Street to a point 50 ft to the west.
2. North side of Broadway from its intersection with Brook Street to a point approximately 230 ft to the west.

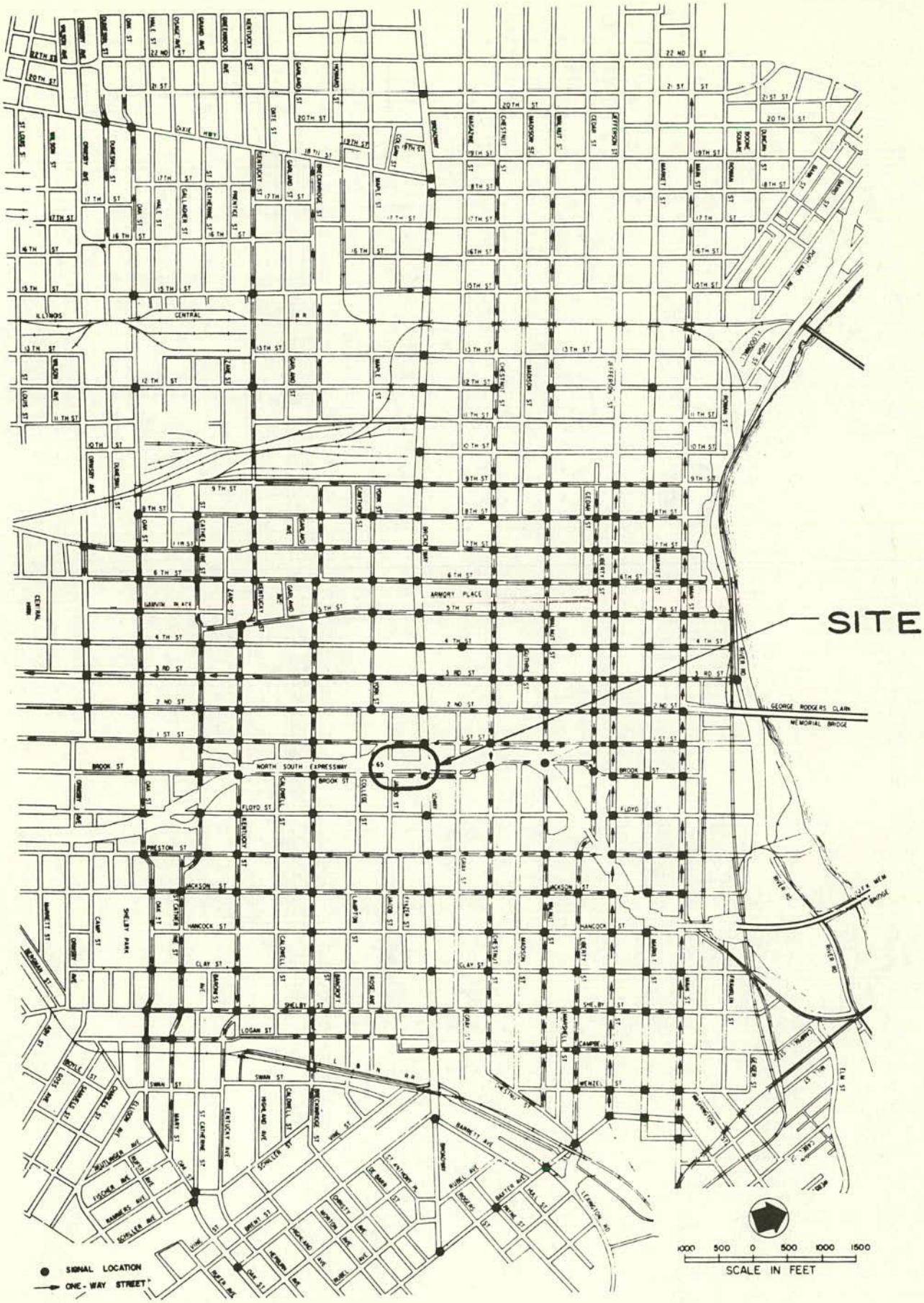


Figure G-98. Location map, Experiment D13.

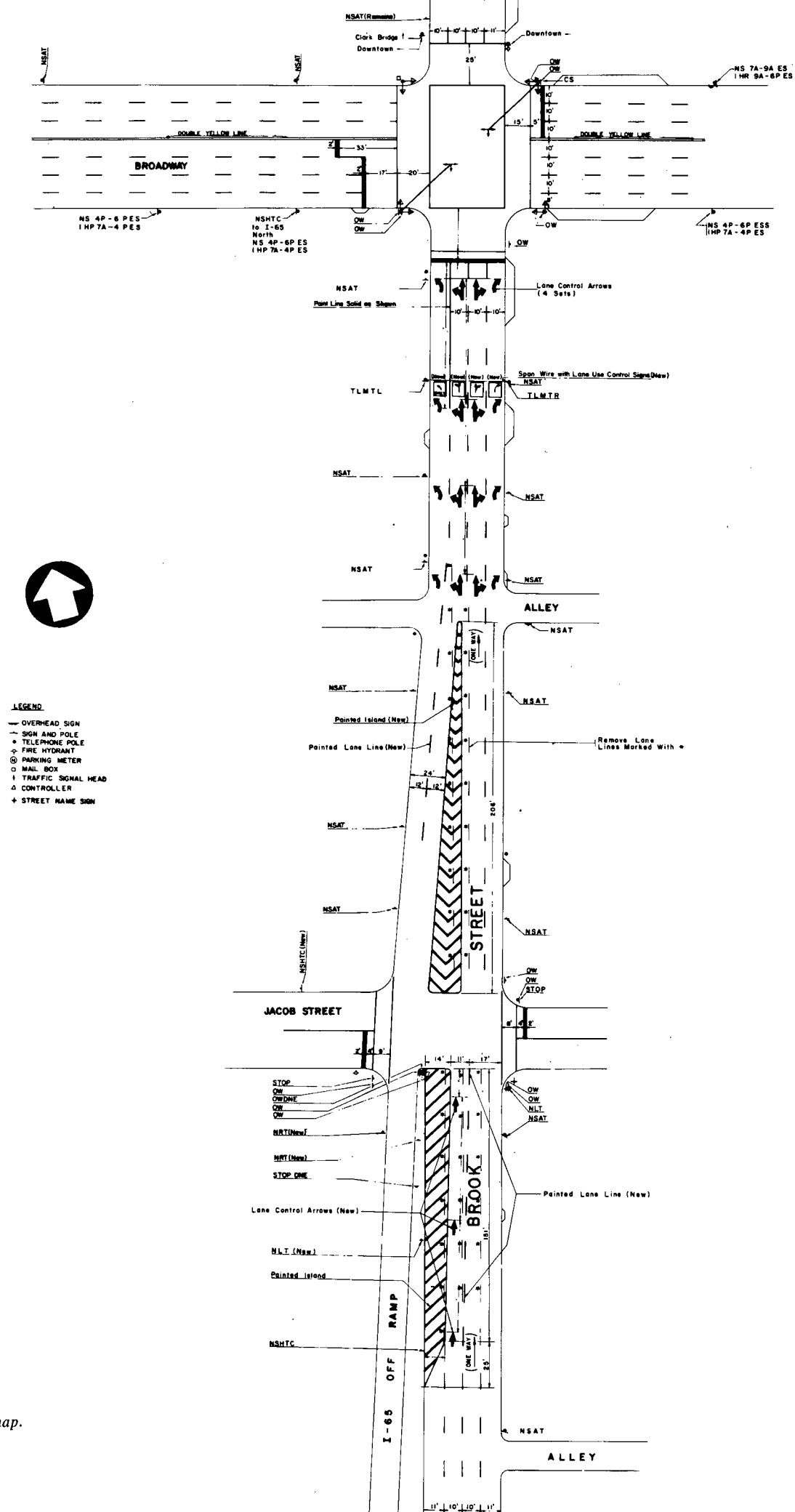


Figure G-99. Vicinity map.

Phase 4 consisted of making Jacob Street one-way westbound between Brook Street and First Street and extending the Brook Street island installed in Phase 1 across Jacob Street. Figure G-101 shows newspaper articles concerning these changes.

Dates of implementation of the various phases and data group measurements are:

ITEM	DATES
Data group 1	9/26/68, 9/27/68, 9/30/68, 10/2/68
Phase 1	Week of 11/17/68
Data group 2	12/2/68, 12/4/68, 12/5/68, 12/6/68
Phase 2	12/7/68
Data group 3	12/13/68, 12/16/68, 12/19/68
Phase 3	12/20/68
Data group 4	1/9/69, 1/10/69, 1/13/69
Phase 4	Week of 1/19/69
Data group 5	2/3/69, 2/7/69, 2/13/69

Measurements for each of the data groups included:

1. The travel time in seconds for automobiles, light trucks, and heavy trucks to proceed from line A to line C, line A to line D, line A to line E, line B to line C, line B to line D, and line B to line E, as shown in Figure G-100. These were recorded manually by stopwatch for the morning peak period of 7:15 to 9 AM, the midday period of 1:30 to 2:30 PM, and the evening peak period of 4:30 to 5:30 PM. The vehicle types were defined as follows:
   a. Automobile: any vehicle with two axles and a total of four tires.
   b. Light truck: any vehicle with two or three axles and a total of six tires.
   c. Heavy trucks: multiaxled vehicles with more than six tires.
2. The number of vehicles stopped at the end of the red signal interval and the vehicular movements during the green interval by lane were recorded by cycle at two approaches (Fig. G-100):
   a. The I-65 off-ramp terminus at Jacob Street.
   b. The Brook Street approach to Broadway.

The Brook Street indications at Broadway were used to control the counts at both locations. These measurements were taken from 7:15 to 8:15 AM, 1:30 to 2:30 PM, and 4:30 to 5:30 PM.

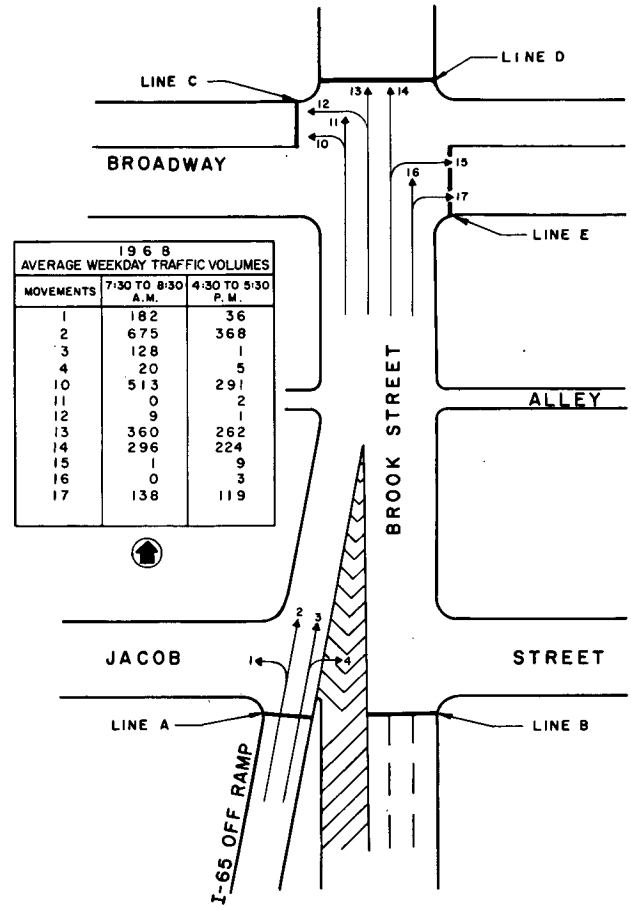


Figure G-100. Traffic measurements.

Analysis and Conclusions

Analysis.—Travel times for automobiles were analyzed by time of day, movement, and data group. Comparison of the travel times during all time periods for ramp vehicles indicated a significant ($\alpha = 0.05$) decrease in the mean travel time between data group 1 and data groups 4 and 5 for automobiles and light trucks turning left (movement A to C) at Broadway (Table G-63).

The reduction in travel time was even more pronounced for automobiles during the morning peak hour (Table G-64).

No differences in travel time between data groups were observed for movements A to D and A to E by vehicle type (Table G-65).

It would seem that there should be differences between the data groups for light and heavy trucks. However, the large variability determined and the small number of samples observed did not produce a statistically significant difference.

Comparison of the Brook Street travel times indicated no significant differences between data groups, except for a significant ($\alpha = 0.05$) difference between data group 1 and data group 2 for automobiles in movement B to C (Table G-66).

Vehicular movement data were summarized for 3 hr per day and for three days, providing data for 9 hr, or 360 traffic signal cycles per data group. Table G-67 gives the number of vehicles in each movement on the Brook Street approach to Broadway. The numbers in parentheses are the percentages of the data group total for each movement(s). As indicated, there was a substantial increase in left turns (movement 12) from the optional turn lane after Phase 1 and an accompanying reduction in the straight-through movement. The net effect was about a 3- to 4-percent change in type of movement (i.e., straight to turn).

Table G-68 gives the vehicular movements at the foot of

TABLE G-63

MEAN TRAVEL TIME FOR RAMP VEHICLES TURNING LEFT AT BROADWAY (MOVEMENT A TO C), ALL TIME PERIODS

VEHICLE TYPE	MEAN TRAVEL TIME (SEC), BY DATA GROUP				
	1	2	3	4	5
Automobiles	48.5	43.6	40.1	38.2	38.7
Light trucks	48.6	46.4	46.0	42.6	47.7
Heavy trucks	60.2	47.6	48.2	46.6	43.7

TABLE G-64

MEAN TRAVEL TIME FOR RAMP VEHICLES TURNING LEFT AT BROADWAY, MORNING PEAK HOUR

VEHICLE TYPE	MEAN TRAVEL TIME (SEC), BY DATA GROUP				
	1	2	3	4	5
Automobiles	73.4	43.0	42.3	41.2	38.5

TABLE G-65

MEAN TRAVEL TIME, ALL TRAVEL TIMES

MOVEMENT	VEHICLE TYPE	MEAN TRAVEL TIME (SEC), BY DATA GROUP				
		1	2	3	4	5
A to D	Automobiles	38.4	39.0	37.7	39.7	34.6
	Light trucks	42.5	49.9	49.7	38.7	31.8
	Heavy trucks	38.2	47.6	35.4	38.0	54.0
A to E	Automobiles	36.2	41.8	40.9	37.1	39.3
	Light trucks	38.3	41.5	43.4	39.3	49.9
	Heavy trucks	51.6	49.2	52.9	42.8	55.3

TABLE G-66

MEAN TRAVEL TIME, ALL TIME PERIODS

MOVEMENT	VEHICLE TYPE	MEAN TRAVEL TIME (SEC), BY DATA GROUP				
		1	2	3	4	5
B to C	Automobiles	47.0	39.8	36.5	39.0	33.8
	Light trucks	33.1	35.5	33.7	46.6	45.8
	Heavy trucks	—[a]	—[a]	—[a]	—[a]	—[a]
B to D	Automobiles	37.9	37.1	38.6	36.1	36.0
	Light trucks	38.0	43.7	40.4	39.9	35.6
	Heavy trucks	46.7	40.7	42.4	46.7	42.4
B to E	Automobiles	35.7	41.3	39.6	35.6	36.2
	Light trucks	45.1	36.6	39.5	35 4	44.1
	Heavy trucks	30.4	45.7	44.1	34.6	34.5

[a] Insufficient observations.

TABLE G-67

VEHICULAR MOVEMENTS ON BROOK STREET

DATA GROUP	VEHICLES, BY MOVEMENT					
	LEFT TURN		STRAIGHT	RIGHT TURN		
	10	12	11, 13, 14, 16	15	17	ALL
1	3,567(36%)	51(1%)	4,750(49%)	85(1%)	1,287(13%)	9,740(100%)
2	3,603(36%)	440(4%)	4,567(46%)	22(0%)	1,405(14%)	10,037(100%)
3	3,293(35%)	527(6%)	4,219(46%)	40(0%)	1,240(13%)	9,319(100%)
4	3,312(36%)	493(5%)	4,163(45%)	19(0%)	1,311(14%)	9,298(100%)
5	3,355(34%)	611(6%)	4,358(45%)	20(0%)	1,517(15%)	9,861(100%)
All	17,130	2,122	22,057	186	6,760	48,255

TABLE G-68

VEHICULAR MOVEMENTS ON I-65 OFF-RAMP

DATA GROUP	VEHICLES, BY MOVEMENT			
	1	2 & 3	4	ALL
1	815(13%)	5,141(85%)	114(2%)	6,070(100%)
2	854(13%)	5,669(86%)	45(1%)	6,568(100%)
3	694(12%)	5,129(87%)	43(1%)	5,866(100%)
4	739(12%)	5,161(87%)	36(1%)	5,936(100%)
5	725(12%)	5,152(88%)	0(0%)	5,877(100%)
All	3,827	26,252	238	30,317

the I-65 off-ramp. No change occurred between the data groups.

Analysis of the number of vehicles stored on the ramp at the end of the Brook Street red interval at Broadway indicated a significant reduction by data group during the AM period in the number of cycles when vehicles were stored (Table G-69).

A significant ($\alpha = 0.05$) reduction in the number of vehicles stopped per cycle was observed between data group 1 and all other data groups for the Brook Street approach to Broadway (Table G-70). The difference was

Brook Section Traffic Flow Being Changed

The Louisville-Jefferson County Traffic Engineering Department is changing the traffic pattern on Brook between Jacob and Broadway to speed up movement of vehicles, particularly those coming off the North-South Expressway.

Department crews yesterday began painting signs on the pavement for the new traffic-control system. Overhead signs will be hung later.

Under the new system, northbound traffic coming off the expressway onto Brook at Jacob will funnel into two lanes, either of which could be used to turn left onto Broadway. Traffic in the curb lane will have to turn at Broadway, while that in the adjoining lane can either turn left or continue northward on Brook.

In the past, vehicles turning onto Broadway used the curb lane only

Adjoining Lane Has Option

Traffic from the south on Brook will be funneled into two lanes on the east side of the street just south of Jacob. Vehicles in the right curb lane will have to turn right at Broadway, while those in the adjoining lane can turn right or continue on Brook.

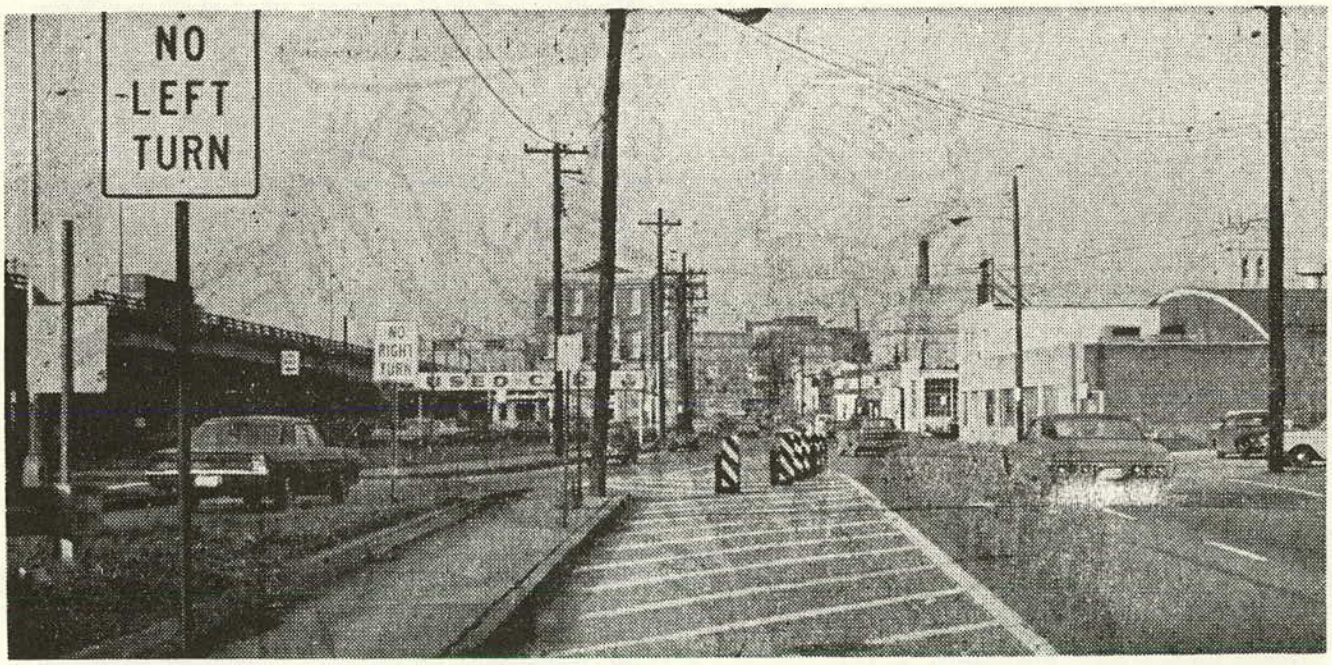
Vehicles coming from the south on Brook and wanting to turn left on Broadway will have to work their way into the center lane on the west side of the street. Those coming off the expressway and wanting to turn right onto Broadway will have to get into the center lane on the east side of Brook.

Parking meters are being removed and parking will be banned on both sides of Brook from a point just south of Jacob to Broadway.

Traffic Engineer Arthur R. Daniel Jr., said relatively little revenue will be lost by removal of the meters. They have been used less than 40 per cent of the time, he explained.

Daniel said that Jacob will be made one way west between Brook and Second under the new system.

"THE LOUISVILLE TIMES," February 6, 1969



Staff Photo

Restrainer

A row of barricades erected on Brook at Jacob is expected to reduce the traffic hazard at the intersection. The obstruction is located where northbound traffic exits from the North-South Expressway, and it will prevent drivers from cutting sharply east across lanes carrying northbound traffic on Brook. Jacob is a block south of Broadway.

Figure G-101. Newspaper publicity, Phase 4 changes.

greater during the AM period than in the midday and PM periods.

Analysis of the vehicles through per cycle at Brook Street indicated a significant difference between data group 3 and all other data groups (Table G-71). This condition is due to a reduction in the number of vehicles through per cycle on December 13, 1968, during the AM period.

Table G-71 also gives the number of vehicles through per cycle on the ramp. Data group 2 is statistically higher than all other data groups, but the difference may not be meaningful.

Conclusions.—Based on the foregoing analysis, it is concluded that:

1. Travel time was significantly reduced for ramp vehicles and Brook Street vehicles turning left at Broadway.
2. The pavement markings and signs caused a reduction in the straight-through movement and an increase in the turning movements on Brook Street at Broadway.

TABLE G-69

NUMBER OF CYCLES WHEN VEHICLES WERE STORED ON THE RAMP, AM PERIOD

DATA GROUP	NO. OF CYCLES
1	52
2	3
3	2
4	1
5	0

TABLE G-70

VEHICLES STOPPED PER CYCLE ON BROOK STREET

	VEHICLES STOPPED PER CYCLE, BY TIME		
DATA GROUP	AM	MIDDAY	PM
1	27.4	15.2	14.8
2	21.6	12.7	12.8
3	19.5	12.1	12.5
4	18.6	12.1	12.0
5	20.3	13.0	12.5

TABLE G-71

VEHICLES THROUGH PER CYCLE

	VEHICLES THROUGH PER CYCLE, BY APPROACH	
DATA GROUP	BROOK ST.	RAMP
1	27.2	16.8
2	28.2	18.1
3	25.6	16.4
4	27.1	16.4
5	28.1	16.7

3. The number of cycles with vehicles stored on the ramp was substantially reduced.
4. The number of vehicles stopped per cycle on Brook Street at Broadway was significantly reduced.

Channelization and Lane Control Devices on Brook Street, Jefferson Street, I-65 Off-Ramps—Experiment D68

Experiment D68 evaluates the effects of changes in the channelization of the I-65 off-ramps to Jefferson Street and Brook Street in Louisville (Fig. G-102). The off-ramps are major traffic carriers during the morning peak hour for inbound trips to the downtown area.

Experimental Area

Jefferson Street is a 60-ft-wide one-way arterial street permitting travel in a westbound direction. East of Brook Street, 1-hr parking is permitted between the hours of 7:00 AM and 6:00 PM on the south side of the street; stopping is not permitted at any time on the north side of the street. West of Brook Street, stopping is not permitted at any time on the south side of the street; metered parking is permitted on the north side of the street. Brook Street is a 42-ft-wide one-way street permitting travel in a northbound direction. South of Jefferson Street, stopping is prohibited on both sides of the street. North of Jefferson Street, metered parking is permitted on both sides of the street except from 7:00 to 9:00 AM and 4:00 to 6:00 PM on the east side of the street. These regulations are shown in Figure G-103.

The traffic signals at the intersections of Jefferson Street with Brook Street and Jefferson Street with First Street are part of the interconnected synchronous system north of Broadway. The signals are set to provide Jefferson Street with 55 percent of the cycle time at its intersection with Brook Street and 57 percent of the cycle time at its intersection with First Street. No changes were made in signal timing for this experiment.

Traffic volumes for an average weekday in 1968 are shown in Figure G-104. As indicated, the heaviest traffic hour is during the morning peak hours.

Design of Experiment

Observations of ramp traffic during the morning peak hours indicated considerable backup due to vehicles stopping at the ramp terminus before merging with Jefferson Street traffic. Also, traffic frequently backed up on Jefferson Street from First Street to Brook Street. To alleviate these conditions, ramp traffic was physically separated from Brook Street and Jefferson Street traffic by means of painted islands and movable barrels. Double left-turn indications were also installed on the Jefferson Street approach to First Street. The improvement plan is shown in Figure G-103. These improvements were installed on March 18, 1969, as a cooperative effort of the Louisville and Jefferson County Department of Traffic Engineering and the Kentucky Department of Highways.

The surveillance system for this experiment consisted of:

1. Automatic Traffic Recorder (ATR equipment counts).

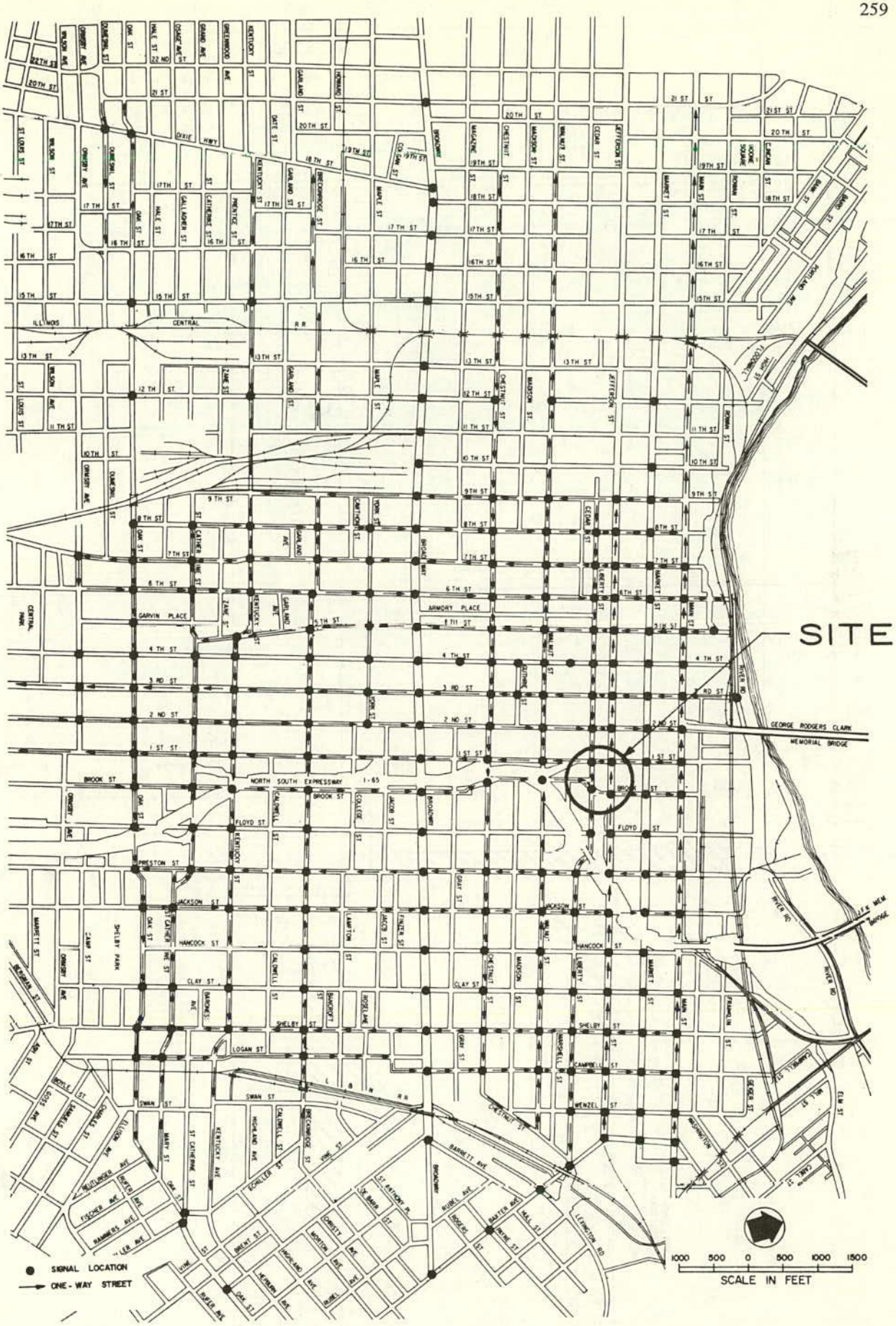


Figure G-102. Location map, Experiment D68.

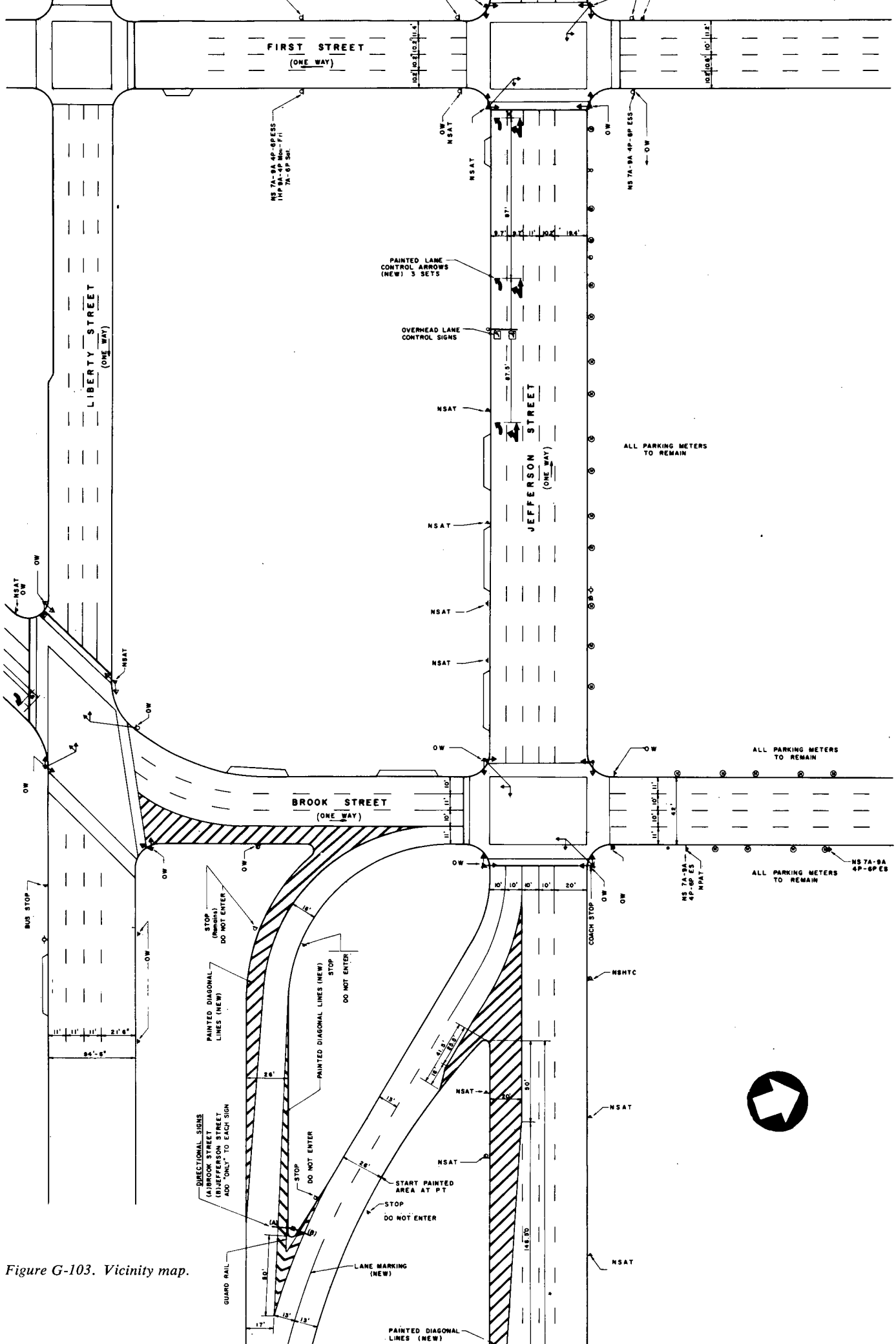
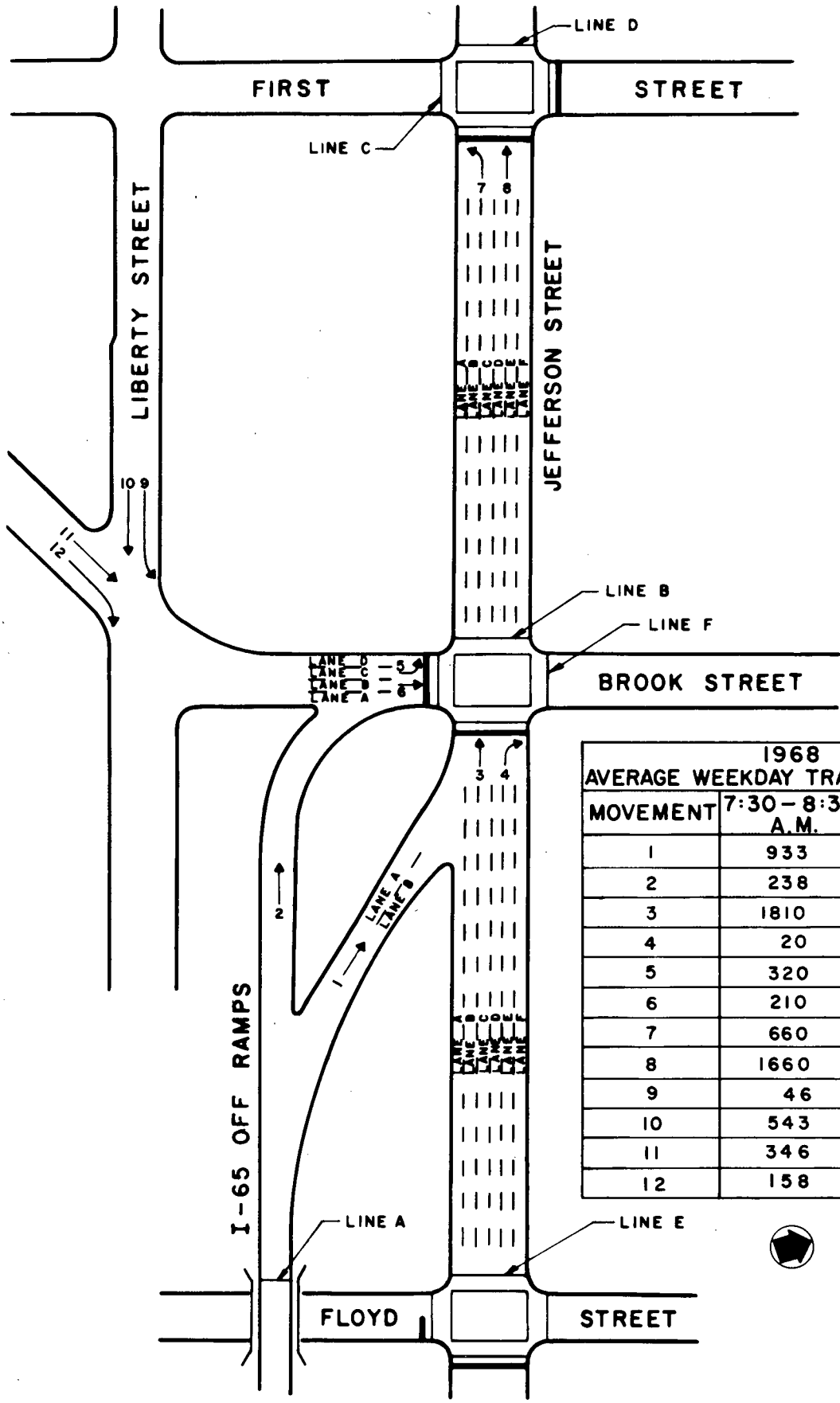


Figure G-103. Vicinity map.



1968 AVERAGE WEEKDAY TRAFFIC VOLUME		
MOVEMENT	7:30 – 8:30 A.M.	4:30 – 5:30 P.M.
1	933	425
2	238	83
3	1810	905
4	20	288
5	320	270
6	210	441
7	660	383
8	1660	960
9	46	68
10	543	1214
11	346	560
12	158	618

Figure G-104. Traffic measurements.

ATR counts were taken for 24-hr periods on both off-ramps and on Jefferson Street between Floyd Street and Brook Street.

2. Speed and delay runs. Runs were made on Jefferson Street between Baxter Avenue and Ninth Street, a distance of approximately 9,345 ft.

3. Stopwatch travel time measurements. Travel time for automobiles, light trucks, and heavy trucks was recorded by stopwatch measurements for the following movements, as shown in Figure G-104:

a. Line A to lines B or F.
b. Line E to lines B or F.
c. Line B to lines C or D from line E.
d. Line B to lines C or D from line A.

4. Vehicle counts. The number of vehicles stopped per cycle and the number of vehicles through per cycle were counted at the following approaches:

a. Jefferson Street approach to Brook Street.
b. Brook Street approach to Jefferson Street.
c. Jefferson Street approach to First Street.

The number of vehicles backed up on the I-65 off-ramp to Jefferson Street were also counted.

The dates when the various measurements were taken are given in Table G-72.

Analysis

Analysis of the ATR counts (Table G-73) indicated a significant increase in ramp traffic of 670 vehicles (8.6 percent) for a 24-hr period and an increase of 128 vehicles (8.3 percent) for the 7:30 to 8:30 AM period. It should be noted that Jefferson Street ramp traffic increased, whereas Brook Street ramp traffic decreased.

Analysis of the speed and delay run data indicated no significant changes between the "before" and "after" mean values of travel time, delay time, or number of stops. The mean values and variances for these data are given in Tables G-74 and G-75.

Analysis of the stopwatch travel time measurements indicated no significant difference between the "before" and "after" conditions for the midday or PM time periods for any vehicle type. During the AM time period, only the movements for automobiles from line A to line B and line E to line F indicated significant differences at the $\alpha \leq 0.10$ level (Table G-76). Changes in the variances are given in Table G-77.

The distributions on the number of vehicles stopped on the Jefferson Street off-ramp are summarized in Table G-78. Analysis of these data for the AM time period indicated a significant shift of vehicles to lane B, resulting in a more even distribution of vehicles stopped on the ramp. The number of vehicles through from each lane on Jefferson

TABLE G-72

DATES AND TIME OF TRAFFIC MEASUREMENTS

	"BEFORE"							"AFTER"								TIME			
	FEB.		MARCH					APRIL								MAY		7:30-9:00 AM 1:30-3:00 PM 4:00-5:30 PM	7:30-8:30 AM 1:30-2:30 PM 4:30-5:30 PM
MEASUREMENT	24	28	3	4	5	6	7	3	7	8	21	22	28	29	30	9	24-HR		
ATR counts				X	X	X	X		X	X	X	X					X		
Speed and delay runs	X			X		X		X		X	X		X	X	X			X	
Stopwatch travel time measurements	X			X		X		X		X	X							X	
Vehicle counts		X	X		X		X		X			X				X			X
Classification counts	X	X																X	

TABLE G-73

SUMMARY OF ATR COUNTS, MEAN VALUES

ATR LOCATION	24-HR VOLUME "BEFORE"	24-HR VOLUME "AFTER"	24-HR VOLUME DIFFERENCE	7:30-8:30 AM VOLUME "BEFORE"	7:30-8:30 AM VOLUME "AFTER"	7:30-8:30 AM VOLUME DIFFERENCE
Jefferson St. ramp	6,006	6,878	+872(+14.5%)	1,079	1,331	+252(+23.4%)
Brook St. ramp	1,797	1,595	−202(−11.2%)	462	338	−124(−26.8%)
Total off I-65	7,803	8,473	+670(+ 8.6%)	1,541	1,669	+128(+ 8.3%)
Jefferson St. between Brook St. and First St.	8,747	9,201	+454(+ 5.2%)	879	877	—

TABLE G-74

SUMMARY OF SPEED AND DELAY ANALYSIS, MEAN VALUES

TIME PERIOD	VARIABLE	MEAN VALUES "BEFORE"	MEAN VALUES "AFTER"	DIFFERENCE
AM	Travel time	290.7	284.9	−5.8 (−2.0%)
	Delay time	20.8	18.1	−2.7(−13.0%)
	Number of stops	0.8	0.7	−0.1(−12.5%)
Midday	Travel time	263.5	272.7	9.2 (3.5%)
	Delay time	8.5	11.5	3.0 (35.3%)
	Number of stops	0.4	0.4	0.0 (0.0%)
PM	Travel time	270.1	271.7	1.6 (0.6%)
	Delay time	11.1	10.9	−0.2 (−1.8%)
	Number of stops	0.5	0.4	−0.1(−20.0%)

TABLE G-75

SUMMARY OF SPEED AND DELAY ANALYSIS, VARIANCES

TIME PERIOD	NO. OF OBS. "BEFORE"	NO. OF OBS. "AFTER"	VARIABLE	VARIANCES "BEFORE"	VARIANCES "AFTER"	F RATIO	LEVEL OF SIG.
AM	50	21	Travel time	1870.7	1277.9	1.46	NS
			Delay time	537.4	340.6	1.58	NS
			Number of stops	0.7	0.4	1.75	0.10
Midday	45	23	Travel time	609.2	739.5	1.21	NS
			Delay time	157.0	198.1	1.26	NS
			Number of stops	0.3	0.3	1.00	NS
PM	52	24	Travel time	773.5	829.6	1.07	NS
			Delay time	297.9	268.9	1.11	NS
			Number of stops	0.6	0.3	2.00	0.10

NS = not significant at $\alpha \leq 0.10$.

TABLE G-76

SUMMARY OF TRAVEL TIME ANALYSIS, AM AUTOS

MOVEMENT	CONDITION	MEAN VALUES "BEFORE"	MEAN VALUES "AFTER"	DIFFERENCE	LEVEL OF SIG.
A-B	Not stopped	18.6	20.3	+1.7 (+9.1%)	0.10
	Stopped	44.4	43.1	−1.3 (−2.9%)	NS
E-F	Not stopped	18.3	14.9	−3.4(−18.6%)	0.10
	Stopped	38.1	41.0	+2.9 (+7.6%)	NS

NS = not significant at $\alpha \leq 0.10$.

TABLE G-77

SUMMARY OF TRAVEL TIME ANALYSIS, AM AUTOS

MOVEMENT	CONDITION	NO. OF OBS. "BEFORE"	NO. OF OBS. "AFTER"	VARIANCES "BEFORE"	VARIANCES "AFTER"	F RATIO	LEVEL OF SIG.
A-B	Not stopped	36	51	14.7	47.5	3.23	0.005
	Stopped	159	147	388.9	255.0	1.53	0.005
E-F	Not stopped	12	14	43.7	5.0	8.74	0.005
	Stopped	7	6	75.1	33.2	2.26	NS

NS = not significant at $\alpha \leq 0.10$.

TABLE G-78

VEHICLES STOPPED PER HOUR, JEFFERSON STREET OFF-RAMP

TIME PERIOD	CONDITION	VEHICLES STOPPED PER HOUR, BY LANE A	B	ALL
AM	"Before"	373	147	520
	"After"	217	334	551
Midday	"Before"	4	1	5
	"After"	3	0	3
PM	"Before"	2	1	3
	"After"	3	2	5

Street at Brook Street is given in Table G-79. Analysis of this table determined that considerably more vehicles are using lane B, indicating that the two ramp lanes are being used more equally. This change also reflects the increased volume on the ramp. Analysis of the number of vehicles stopped per hour on Jefferson Street at Brook Street (Table G-80) indicated that the number of vehicles stopped in each lane was more evenly distributed in the "after" condition. Also, the number of vehicles stopped during the AM time period decreased. Table G-81 gives the vehicles through per hour on Brook Street at Jefferson Street. It indicates a reduction in the number of left turns from lane A during the AM period. This reduction was also reflected in the ATR counts on the Brook Street ramp during the 7:30 to 8:30 AM period. The number of vehicles stopped per hour on Brook Street at Jefferson Street was significantly reduced during the AM period (Table G-82). Analysis of the vehicles through per hour on Jefferson Street at First Street (Table G-83) indicates practically no change in lane distribution. However, the number of vehicles stopped per hour was significantly reduced in the "after" conditions during the AM period (Table G-84).

The classification of vehicles counted on Jefferson Street between Brook Street and First Street and on the Jefferson Street off-ramp is given in Table G-85.

Conclusions

Based on the foregoing analysis it can be concluded that:

1. Ramp traffic increased by 670 vehicles (8.6 percent) during a 24-hr period and by 128 vehicles (8.3 percent) during the 7:30 to 8:30 AM period.
2. Travel time, delay time, and number of stops did not change between the "before" and "after" condition measured by speed and delay runs on Jefferson Street.
3. No changes were made in the stopwatch travel time measurements at the $\alpha \leq 0.10$ level.
4. The number of vehicles stopped per hour on the Jefferson Street off-ramp was more evenly distributed by lane during the "after" conditions.
5. During the AM time period the number of vehicles stopped per hour decreased significantly on the:

TABLE G-79

VEHICLES THROUGH PER HOUR ON JEFFERSON STREET AT BROOK STREET

TIME PERIOD	CONDITION	VEHICLES THROUGH PER HOUR, BY LANE A	B	C	D	E STRAIGHT	E RIGHT	F STRAIGHT	F RIGHT	ALL
AM	"Before"	658	463	425	383	218	66	0	1	2,214
	"After"	673	671	373	324	196	43	0	17	2,297
Midday	"Before"	222	179	187	191	121	38	0	0	938
	"After"	183	139	295	191	110	33	0	5	956
PM	"Before"	202	218	203	242	122	46	0	0	1,033
	"After"	196	146	348	246	127	26	0	3	1,092

TABLE G-80

VEHICLES STOPPED PER HOUR
ON JEFFERSON STREET AT BROOK STREET

TIME PERIOD	CONDITION	VEHICLES STOPPED PER HOUR, BY LANE A	B	C	D	E	F	ALL
AM	"Before"	336	197	120	91	61	1	806
	"After"	274	274	98	50	53	3	752
Midday	"Before"	117	71	45	32	49	0	314
	"After"	80	67	81	41	44	3	316
PM	"Before"	96	74	45	42	51	0	308
	"After"	89	82	104	44	44	3	366

TABLE G-82

VEHICLES STOPPED PER HOUR
ON BROOK STREET AT JEFFERSON STREET

TIME PERIOD	CONDITION	VEHICLES STOPPED PER HOUR, BY LANE A	B	C	D	ALL
AM	"Before"	140	85	54	127	406
	"After"	40	30	38	154	262
Midday	"Before"	45	23	44	25	137
	"After"	31	20	31	42	124
PM	"Before"	33	23	29	63	148
	"After"	31	20	39	42	132

TABLE G-81

VEHICLES THROUGH PER HOUR ON BROOK STREET
AT JEFFERSON STREET

TIME PERIOD	CONDITION	VEHICLES THROUGH PER HOUR, BY LANE A LEFT	A STRAIGHT	B LEFT	B STRAIGHT	C	D	ALL
AM	"Before"	389	13	92	277	183	312	1,266
	"After"	288	12	19	273	225	260	1,077
Midday	"Before"	217	15	11	155	180	78	656
	"After"	205	23	13	165	176	83	665
PM	"Before"	221	15	10	198	102	179	725
	"After"	212	22	9	208	174	95	720

TABLE G-83

VEHICLES THROUGH PER HOUR ON JEFFERSON STREET AT FIRST STREET

TIME PERIOD	CONDITION	VEHICLES THROUGH PER HOUR, BY LANE A LEFT	A STRAIGHT	B LEFT	B STRAIGHT	C	D	E	F	ALL
AM	"Before"	713	12	124	485	564	495	296	0	2,689
	"After"	657	5	140	476	546	470	275	0	2,569
Midday	"Before"	293	5	31	170	214	192	153	0	1,058
	"After"	275	1	49	182	210	249	148	0	1,114
PM	"Before"	318	6	17	183	232	312	165	0	1,233
	"After"	321	3	37	181	242	304	173	0	1,261

TABLE G-84

VEHICLES STOPPED PER HOUR
ON JEFFERSON STREET AT FIRST STREET

TIME PERIOD	CONDITION	VEHICLES STOPPED PER HOUR, BY LANE A	B	C	D	E	F	ALL
AM	"Before"	369	232	107	115	67	0	890
	"After"	186	157	98	102	63	0	606
Midday	"Before"	38	56	54	77	35	0	260
	"After"	49	71	41	66	33	0	260
PM	"Before"	43	45	58	92	40	0	278
	"After"	38	57	45	76	42	0	258

TABLE G-85

CLASSIFICATION OF VEHICLES COUNTED

	CLASSIFICATION OF VEHICLES (%)							
	JEFFERSON ST. BETWEEN BROOK ST. AND NINTH ST.				JEFFERSON ST. OFF-RAMP			
TIME PERIOD	AUTO-MOBILES	LIGHT TRUCKS	HEAVY TRUCKS	BUSES	AUTO-MOBILES	LIGHT TRUCKS	HEAVY TRUCKS	BUSES
AM	98.3	1.3	0.1	0.3	99.4	0.3	0.3	0.0
Midday	91.6	6.5	1.2	0.7	96.3	2.8	0.7	0.2
PM	94.0	3.5	0.7	1.8	97.7	1.5	0.4	0.4
All	96.5	2.6	0.4	0.5	98.6	0.9	0.4	0.1

a. Jefferson Street approach to Brook Street.
b. Brook Street approach to Jefferson Street.
c. Jefferson Street approach to First Street.

Signal Retiming at First Street and Central Avenue—Experiment A53

Experiment A53 consists of a minor change of the existing signal split at First Street and Central Avenue in Newark. The green time on First Street was increased from 27 to 35 percent of the 90-sec cycle in an attempt to reduce the rather excessive queues on First Street during the peak periods of traffic. Many unsaturated cycles were noted on Central Avenue during the same peak periods.

Experimental Area

The experiment site, the intersection of First Street and Central Avenue, is shown in Figure G-105. Each adjacent signalized intersection (West Market Street and Central Avenue, Norfolk Street and Central Avenue, Sussex Avenue and First Street, West Market Street and First Street) was also monitored to measure any network effect that might occur as a result of the proposed change of signal split.

Central Avenue is a major arterial, 48 ft curb to curb, with parking prohibited from 7:00 to 9:00 AM on the south side and parking prohibited from 4 to 6 PM on the north side. First Street is a collector street 40 ft wide curb to curb, with parking permitted at all times. The signal controller has a single dial operating on a 90-sec cycle with 63 percent green on Central Avenue and 27 percent green on First Street. The roadway dimensions and traffic volumes are shown in Figure G-106.

Purpose and Scope

This experiment determined changes in volume, queue length, vehicles stopped and delay time at all of the approaches to the intersection of Central Avenue and First Street due to a change of the signal split. The effect of the signal split change was measured during the morning peak hour (7:30 to 8:30 AM), evening peak hour (4:30 to 5:30 PM), and during a midday hour (3:30 to 4:30 PM).

Design of Experiment

A summary of speed and delay runs of the Newark area displayed a high percentage of stops at Central Avenue for all runs on First Street. Preliminary observations indicated excessive queues on First Street during the evening rush hours; the Central Avenue approaches appeared to have some unused green time on most cycles. Traffic counts were obtained for the design of the revision of the signal split, using a ratio of the critical equivalent passenger vehicles for each phase, as follows:

		EQUIVALENT PASSENGER VEHICLES PER LANE		
APPROACH	PHASE	AM HOUR	MIDDAY HOUR	PM HOUR
Central WB	1	480	440	600
Central EB	1	720	520	640
First NB	2	260	360	360
First SB	2	400	250	280

Assuming 8 percent for amber, the percentage of green time required is:

$$\text{AM: Phase 1} = \frac{720}{720 + 400} \times (92\%) = 59\%;$$
$$\text{Phase 2} = 92 - 59 = 33\%$$

$$\text{Midday: Phase 1} = \frac{520}{520 + 360} \times (92\%) = 54\%;$$
$$\text{Phase 2} = 92 - 54 = 38\%$$

$$\text{PM: Phase 1} = \frac{640}{640 + 360} \times (92\%) = 59\%;$$
$$\text{Phase 2} = 92 - 59 = 33\%$$

$$\text{Average period} = \text{Phase 1} = 57\%, \text{Phase 2} = 35\%$$

Because this is a single dial controller, an average percentage of green of the three time periods was used as the setting for each phase, or 57 percent for Central Avenue and 35 percent for First Street. The revised timing that was

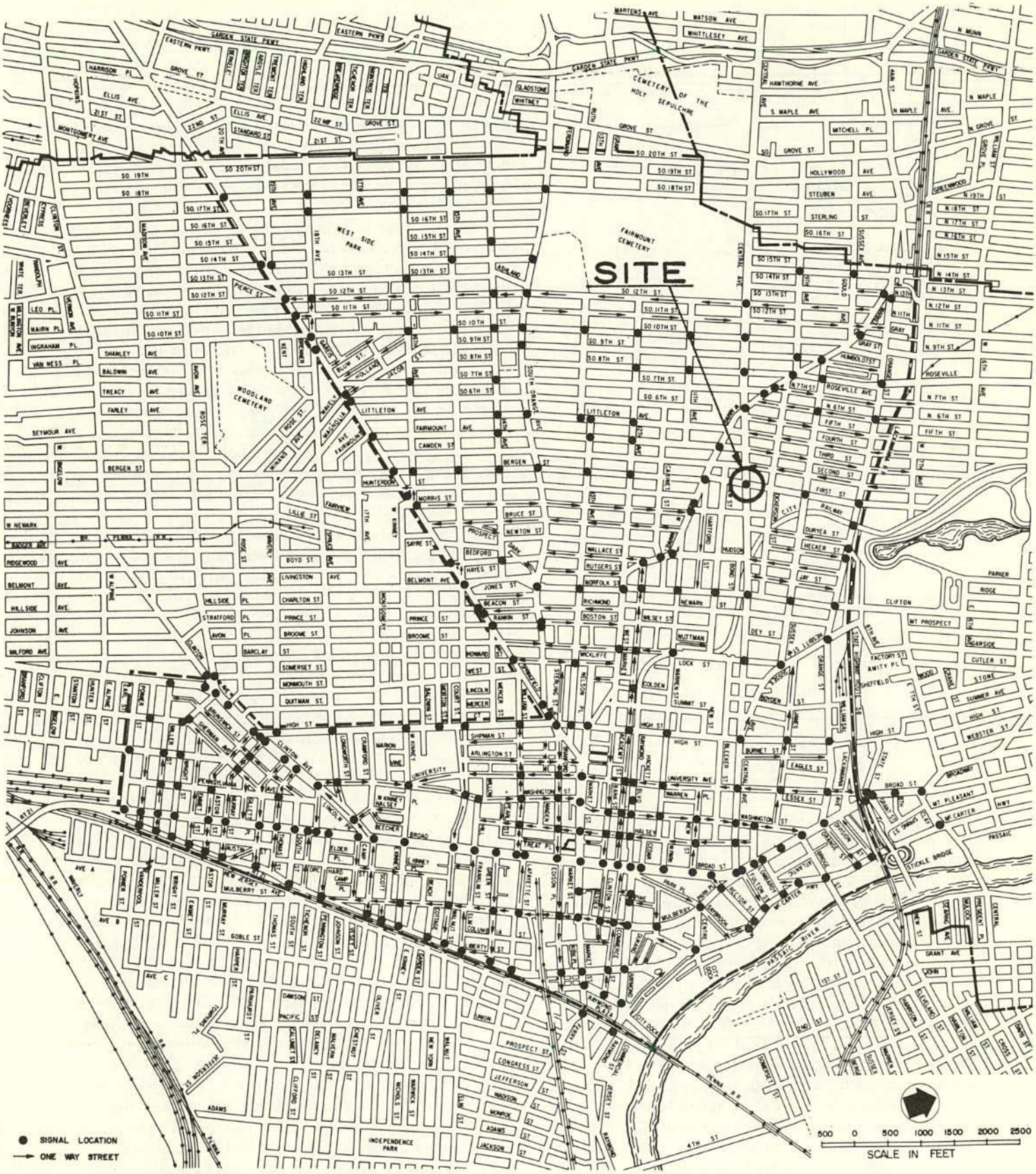


Figure G-105. Location map, Experiment A53.

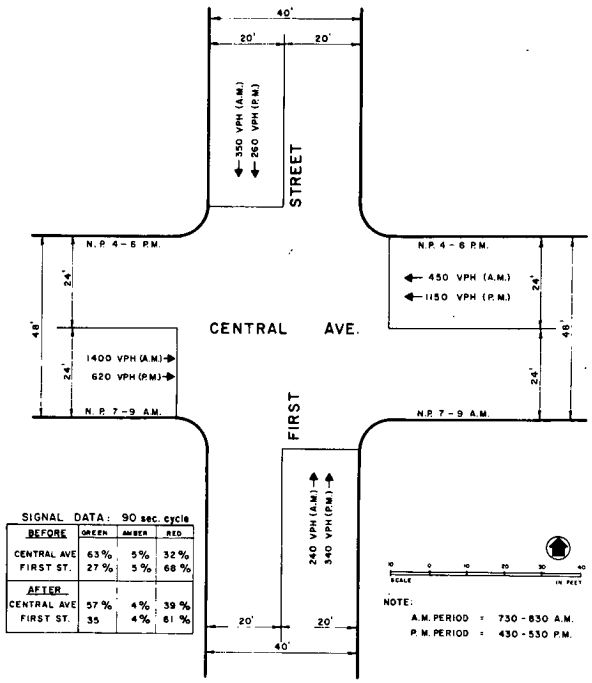


Figure G-106. Vicinity map.

implemented by the Essex County Traffic Engineer is as follows:

	CENTRAL AVE. AND FIRST ST. SIGNAL TIMING (SEC)					
	GREEN		AMBER		RED	
PHASE	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
Central Ave.	56.7	51.3	4.5	3.6	28.8	35.1
First St.	24.3	31.5	4.5	3.6	61.2	54.9

Surveillance System Design.—The variables to be measured for comparison consist of volume on each approach leg per unit of time, the number of vehicles stopped on each approach (or the average queue length) per cycle, the total delay time at each approach (or average delay per vehicle) per cycle, and the number of vehicles stopped for more than one cycle at each approach.

The "before" measurements were scheduled for 3:30 to 5:30 PM May 13, 7:30 to 8:30 AM and 3:30 to 5:30 PM May 14, and 7:30 to 8:30 AM May 15, 1968. The "after" measurements were scheduled for 3:30 to 5:30 PM May 20, 7:30 to 8:30 AM and 3:30 to 5:30 PM May 21, and 7:30 to 8:30 AM May 22. Automatic traffic recorders (ATR) were placed to measure the number of vehicles at each approach leg by 15-min intervals for a 48-hr period between May 13 and May 15, and between May 20 and May 22, 1968.

Implementation of Improvement.—The revised signal timing was implemented by the Essex County Traffic Engineer on Thursday, May 16, 1968. Because the change was rather small, and probably not noticed by the average motorist, only two weekdays were provided for adjustment prior to performing "after" measurements. In fact, visual observation during the PM peak hours of May 16 indicated an immediate reduction of queues on First Street, with no apparent change on Central Avenue. The "after" measurements for May 20, 21, and 22 displayed no noticeable changes of volume or queue length from day to day, showing that the adjustment period was satisfactory. Volume data from ATR control counting stations revealed no unusual variations within the study area during the period of the experiment.

Surveys.—Four men, one at each of the four approaches, counted the vehicles stored during the red intervals and the vehicle movement during the green intervals during each period of measurement. A fifth man counted the approach leg rate of arrival during the red interval for 15 min at each approach leg during each period of measurement. The sixth man alternately monitored the downstream vehicles at each of the four adjacent signalized intersections. Owing to time lost in moving from one location to another, only eight 15-min periods of "before" and "after" monitor counts were obtained during similar time periods for comparison.

A total of 440 cycles was measured for vehicles stopped and vehicles cleared at each approach. One hundred and ten cycles were measured for the arrival rate during the red interval.

Weather conditions were similar on all days of measurements, and, except for a stalled vehicle in the intersection at 3:30 PM on May 20, no unusual events were observed.

First Level Analysis

The purpose of the First Level Analysis is to evaluate the significance of the effects produced in the traffic flow by the revised signal split through a statistical comparison of "before" and "after" measurements of volume, number of vehicles stopped, queue length, and delay time. The analyses were performed for the periods of survey (two days of "before" and two days of "after" measurement) from 7:30 to 8:30 AM, 3:30 to 4:30 PM, and 4:30 to 5:30 PM.

Summary of Field Data.—The field sheets were summarized in preparation for the First Level Analysis. The summary consisted of accumulating the total vehicles stored during each red interval, and the total vehicles clearing each green interval for each approach measured. Fifteen-minute summaries of this data contained the total vehicles clearing, the number of vehicles stored, the number stopped, the maximum and the average queue, and the number of cycles with the vehicles stopped greater than the number of vehicles through at each approach. Hourly summaries included the percentage of turns and percentage of trucks, in addition to the previous data summarized by 15-min periods.

In addition, a summary of the arrival rate during the red interval was performed by 15-min periods for the approaches actually measured. The number of vehicles stored during each 3-sec period was multiplied by the average time stopped during each period to obtain the total vehicle delay during the red interval, and additional time (green plus

amber) was added for vehicles held more than one cycle. The measured average delay time per vehicle stopped per cycle was then used to estimate the total vehicle delay time for the entire hour.

Some extreme values that could not be related to unusual happenings (such as a vehicle stalling in the intersection, causing some congestion for several minutes) were eliminated prior to analysis. Conservative estimates were made to complete measurements for periods that were either eliminated or incomplete, as indicated on the plots of data.

Volume Comparison.—A summary of the manual counts at each approach leg shows very little change of hourly counts for the three periods for either day measured or for "before" and "after" counts (Table G-86). An analysis of the volume change was performed as a four-way ANOVA using the average number of vehicles clearing each cycle during 15-min intervals, comparing Day 1 or Day 2, approach leg, hour period, and "before" and "after" values. A plot of the input data (Fig. G-107) shows very little difference between the two days of "before" measurements or the two days of "after" measurements, and, in fact, very small change between "before" and "after" situations.

As expected, the results of the ANOVA show a significant difference at the 95-percent confidence level for a comparison of time period volumes, and for the interaction of approach leg with time period. There is no difference at the 95-percent level for a comparison of Day 1 or Day 2 volumes on all approach legs, nor for "before" and "after" volumes on all approaches (an average of 57.2 vehicles per cycle before the change and 55.7 vehicles per cycle after). The interactions of approach leg with "before" and "after" volumes, time period with "before" and "after" volumes, and approach leg with time period and "before" and "after" volumes were all found to be not significant. The plot of the interaction between approach leg with "before" and "after" vehicles clearing each cycle (Fig. G-109) shows a slight increase on both First Street approaches and a decrease on both Central Avenue approaches.

Further ANOVA analysis of each individual approach leg showed no significant difference between volumes for Day 1 or 2 on any approach. The comparison of "before" and "after" volumes resulted in no significant difference on three of the four approach legs but did show that the differences on the Central Avenue eastbound approach were significant at the 95-percent level, from 21.9 vehicles per cycle "before" to 21.1 vehicles per cycle "after" the signal revision.

Comparison of Average Queue Length.—The average queue length per cycle during 15-min periods (Fig. G-108) shows very little difference on any approach between Day 1 or Day 2, and only a slight difference between "before" and "after" values. A three-way ANOVA, comparing days, hour period, and "before" and "after," proved no significant difference between Day 1 and Day 2 for any approach leg at the 95-percent level. Therefore, Day 1 and Day 2 were combined for a three-way ANOVA, comparing approach leg, hour period, and "before" and "after" 15-min average queue lengths. This analysis indicated a significant difference between "before" and "after" mean queue lengths on the total of all approach legs and all time periods, from 33.7 to 32.2 vehicles per cycle. Figure G-109 shows a plot of the marginal means for the interaction effect of approach leg with the "before" and "after" values.

A series of three-factor ANOVA of days, time of day, and "before" and "after" was performed on the data for each approach separately. They indicated a significant decrease ($\alpha = 0.05$) in the marginal means between the "before" and "after" conditions for both First Street approaches, and a significant increase ($\alpha = 0.05$) in the marginal means between the "before" and "after" conditions for the eastbound Central Avenue approach.

Comparison of Number of Vehicles Stopped.—The average number of vehicles stopped during each cycle is similar to the average queue per cycle, which equals the average queue minus the number of vehicles held over one cycle. The difference on all approaches for all three time periods was 31.9 vehicles stopped per cycle "before" and 32.1 vehicles "after" the signal revision. There were, however, more vehicles stopped on the Central Avenue approaches and fewer stopped on the First Street approaches.

Comparison of Estimated Vehicle Delay Time.—Because vehicle delay time was not measured for the entire surveillance period, the values were estimated from a 15-min

TABLE G-86

APPROACH LEG HOURLY VOLUMES

			MANUAL COUNT SUMMARY					
	24-HR VOLUME		7:30-8:30 AM		3:30-4:30 PM		4:30-5:30 PM	
APPROACH LEG	(VEH)	DAY	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
Central Ave. EB	10600	1	1356	1329	—	585	665	583
		2	1405	1381	616	534	617	620
Central Ave. WB	10760	1	470	450	—	749	1157	1082
		2	425	469	831	859	1180	1142
First St. NB	3820	1	238	236	—	321	331	332
		2	235	242	310	351	327	369
First St. SB	4530	1	332	350	—	260	270	253
		2	352	350	243	246	252	265
All	29710	1	2396	2365	—	1915	2423	2250
		2	2417	2442	2000	1990	2376	2396

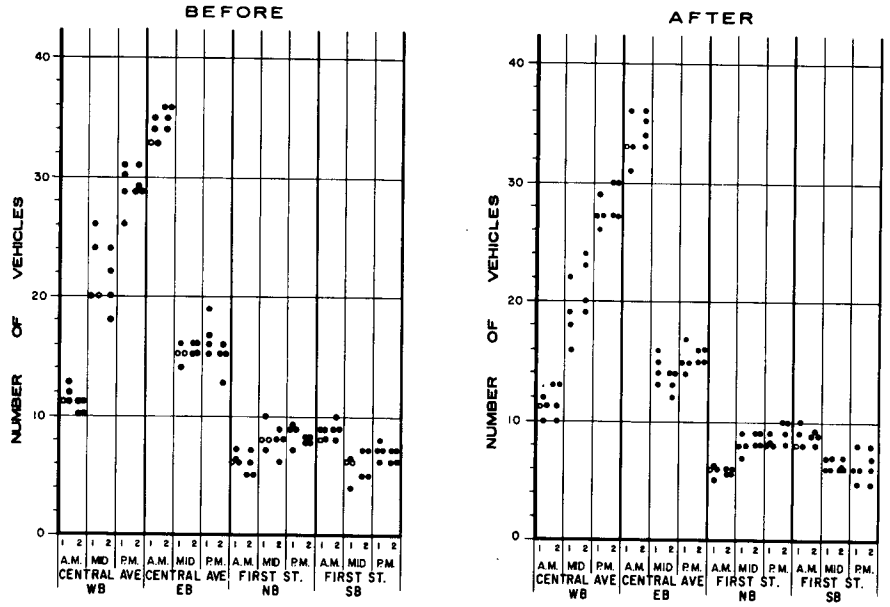


Figure G-107. Average number of vehicles clearing each cycle, by 15-min periods.

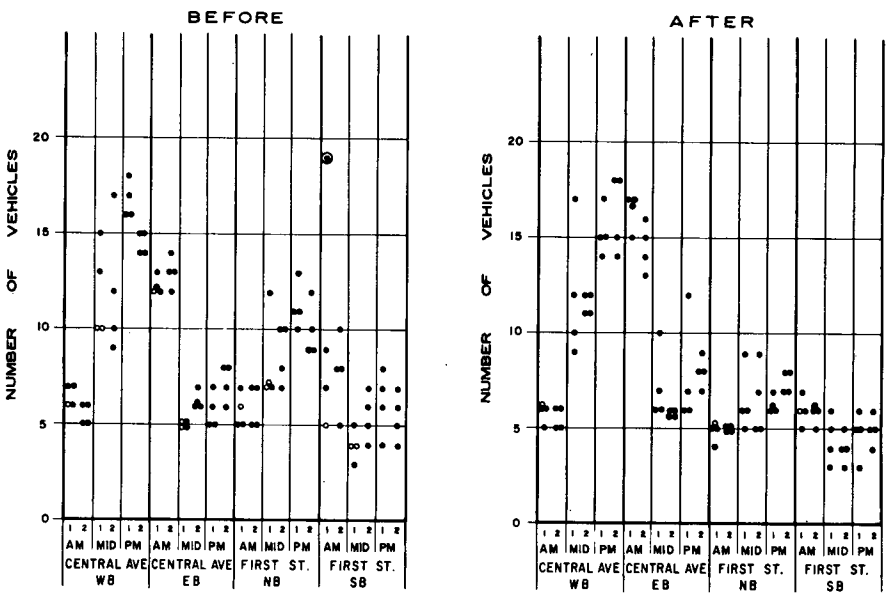


Figure G-108. Average queue length per cycle, by 15-min periods.

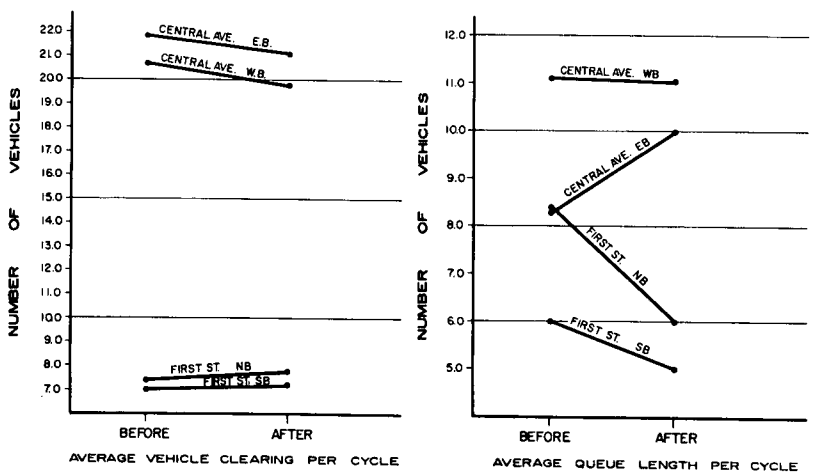


Figure G-109. Interaction between approach leg and "before" or "after."

measurement at each approach leg during each time period surveyed. The results of a three-way ANOVA, comparing the estimated delay time for each approach leg, hour period, and "before" and "after," showed a significant difference between "before" and "after" delay time on all approaches, from 28.4 to 21.6 veh-hr for the combined time periods.

Summary of Comparisons.—Table G-86 summarizes the mean values for each time period of both days of "before" and "after" measurements by approach leg. The most significant differences are in the "before" and "after" measurements on the First Street approaches, especially on First Street northbound during the evening peak hour.

A graph of the number of vehicles clearing and the queue length during each cycle of the evening peak hour for the two critical approaches, First Street northbound and Central Avenue westbound, clearly shows the improvement resulting from the signal split revision (Figs. G-110 and G-111). All the vehicles waiting at the beginning of a green interval did not clear the intersection on 25 of the 40 cycles at First Street northbound prior to the revision of the signal

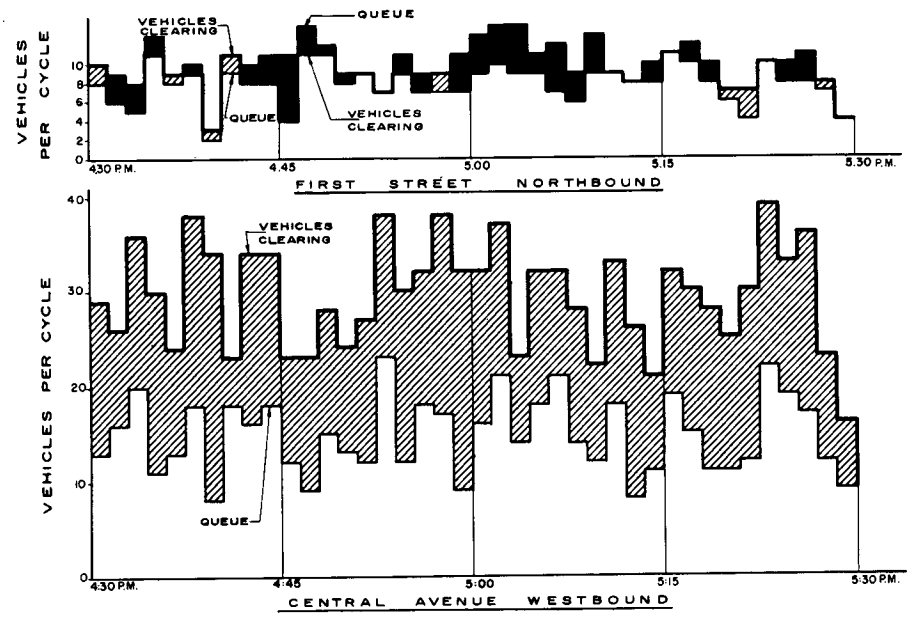


Figure G-110. "Before" conditions.

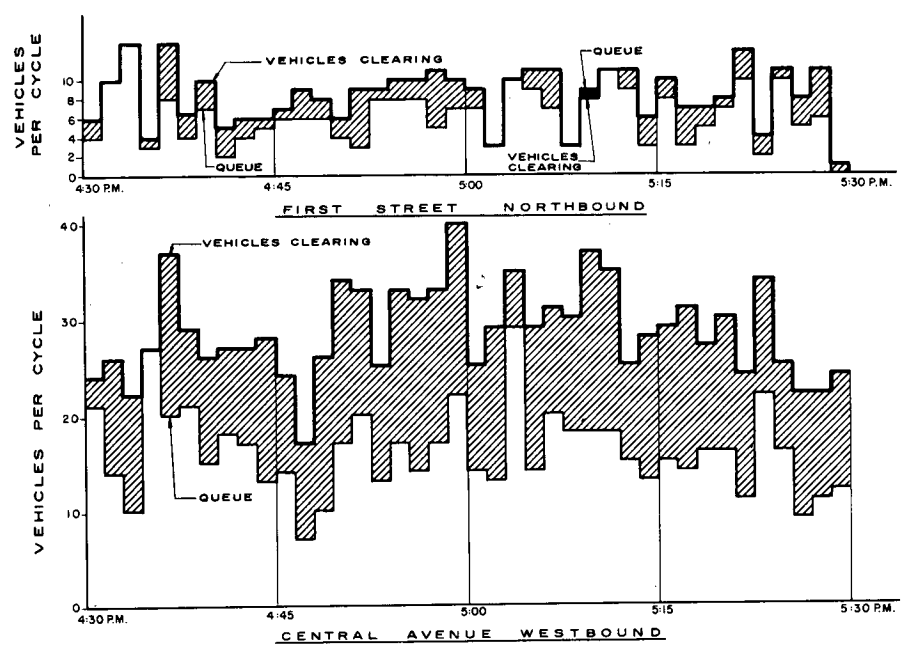


Figure G-111. "After" conditions.

split. This was reduced to only 1 of the 40 cycles not clearing during green after the revision. The increased queue length on Central Avenue westbound was not sufficient to cause vehicles to be held over one green interval.

Although the following comparison of adjacent intersection monitor counts indicates no change between "before" and "after" counts, the data are obviously too sparse to be conclusive:

APPROACH	TIME PERIOD	AVERAGE THROUGH "BEFORE"	AVERAGE THROUGH "AFTER"	AVERAGE QUEUE "BEFORE"	AVERAGE QUEUE "AFTER"
First St. SB at	7:45-8:00 AM	6	7	2	2
West Market	8:00-8:15 AM	5	5	1	3
Central Ave. WB	3:45-4:00 PM	19	18	23	20
at W. Market	5:15-5:30 PM	28	28	40	33

Inasmuch as the volume through each leg of the intersection did not change, it can reasonably be assumed that the effect of the experiment to the downtown intersections was negligible. The maximum queue length at any approach was not sufficient to affect the adjacent intersections. Therefore, it can be concluded that the signal split revision produced no noticeable network changes.

Summary and Conclusions

The results of Experiment A53 indicate that substantial gains can be obtained by a minor revision of the signal split at an intersection when there are excessive queues at one approach and available "green" time during the opposing phase. An analysis of "before" and "after" measurements shows that the 8-percent (7 sec per cycle) increase to the First Street phase at the expense of the Central Avenue phase produced no significant volume change through the intersection, resulted in no change of total vehicles stopped on all approaches, and displayed a slight reduction in the average queue length on all approaches, while substantially reducing the total vehicle delay time and the number of vehicles not clearing within one cycle on all approaches.

The important measurements for judging the efficiency of operation at this intersection are the total vehicle delay time and the number of vehicles held over one cycle. Analysis involves comparison of "before" and "after" measurements of these variables. The results of these two comparisons are similar, as indicated in Table G-87, indicating that the major portion of the decrease in vehicle delay time is the result of the decrease of vehicles held over one cycle (not clearing during the first green interval after arrival).

Both the delay time and vehicles held over one cycle increased substantially by time of day during the "before" period. However, the total volume through the intersection did not display a similar trend. Inspection of data for each approach shows that most of the delay time can be traced to First Street northbound, as indicated in Table G-88. All variables (vehicles clearing, average queue, vehicles not clearing one cycle, and total vehicle delay) display similar increases with time period for the "before" measurement on First Street northbound.

The experiment resulted in a 36-percent decrease in vehicle delay time during the evening rush hours (from 13.2 to 8.5 veh-hr), representing a 7-sec saving for each vehicle through the intersection. A 72-percent decrease

TABLE G-87

SUMMARY OF INTERSECTION "BEFORE" AND "AFTER" VALUES (TOTAL OF ALL FOUR APPROACH LEGS)

VARIABLE	TIME PERIOD	AVERAGE VALUE PER CYCLE "BEFORE"	"AFTER"	DIFFERENCE	CHANGE (%)	SIGNIFICANT (95% LEVEL)
Vehicles clearing	AM	60.4	60.0	−0.4	0.7	
	Midday	50.9	48.7	−2.2	4.3	
	PM	60.2	58.4	−1.7	2.8	
	Average	57.2	55.7	−1.5	2.6	No
Vehicles stopped	AM	30.9	31.9	+1.0	3.2	
	Midday	28.4	29.1	+0.7	2.5	
	PM	35.7	35.4	−0.3	0.8	
	Average	31.7	32.1	+0.4	1.3	No
Queue length	AM	31.9	31.9	0.0	0.0	
	Midday	30.5	29.2	−1.3	4.3	
	PM	38.6	35.5	−3.1	8.0	
	Average	33.7	32.2	−1.5	4.5	Yes
Vehicles held one cycle	AM	0.95	0.05	−0.90	94.7	
	Midday	2.12	0.14	−1.98	93.4	
	PM	2.90	0.03	−2.87	99.0	
	Average	1.99	0.07	−1.92	96.5	Yes
Vehicle-minutes delay	AM	10.8	9.6	−1.2	11.1	
	Midday	11.9	10.2	−1.7	14.3	
	PM	19.8	12.7	−7.1	35.9	
	Average	14.2	10.8	−3.4	23.9	Yes

TABLE G-88

SUMMARY OF APPROACH LEG "BEFORE" AND "AFTER" VALUES

APPROACH LEG	TIME PERIOD	AVERAGE VARIABLE VALUE PER CYCLE							
		VEHICLE CLEAR		QUEUE		VEHICLE HELD OVER		EST. DELAY (VEH-MIN)	
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
Central Ave. WB	AM	11.1	11.4	6.0	5.6	0	0	1.40	1.80
	Midday	21.8	20.1	11.6	11.8	0	0	2.75	3.73
	PM	29.3	27.9	15.6	15.8	0.025	0	3.83	5.30
	Average	20.7	19.8	11.1	11.1	0.008	0	2.66	3.61
Central Ave. EB	AM	34.5	33.9	12.6	15.5	0	0	2.40	3.08
	Midday	15.3	13.9	5.6	6.6	0	0	1.12	1.40
	PM	15.8	15.4	6.5	7.9	0.050	0	1.25	1.90
	Average	21.9	21.1	8.2	10.0	0.017	0	1.59	2.13
First St. NB	AM	6.0	5.9	5.9	4.9	0.438	0.050	2.55	1.82
	Midday	8.0	8.3	8.5	6.5	1.562	0.125	5.53	3.18
	PM	8.3	8.8	10.6	6.9	2.425	0.013	11.25	3.20
	Average	7.4	7.7	8.3	6.1	1.475	0.063	6.44	2.73
First St. SB	AM	8.8	8.8	7.4	5.9	0.512	0	4.43	2.90
	Midday	5.8	6.4	4.8	4.3	0.562	0.013	2.53	1.90
	PM	6.8	6.4	5.9	4.9	0.400	0.013	3.50	2.32
	Average	7.1	7.2	6.0	5.0	0.491	0.009	3.49	2.39

was realized on the First Street northbound approach between 4:30 and 5:30 PM (from 7.5 to 2.1 veh-hr); this represented an average saving of 59 sec for each vehicle.

Although this saving was not sufficient to produce any volume changes by re-routing trips or effect any other measurable network changes, the accumulation of several similar improvements on an artery could possibly result in substantial network changes.

Convenience and Safety.—Both field observation and summary of the data collected indicate the most substantial change due to the revised signal split was a reduction of the average delay time during the evening rush hours. The number of vehicles stopped on all approaches did not change, although there was a decrease on the First Street approaches and an increase on the Central Avenue approaches. Since First Street has poorer visibility than Central Avenue (because of the slight curve on the approaches), the decrease in vehicles stopped on First Street may imply a reduction in rear-end collisions at the intersection. However, the accident study reveals that this location was involved in only three accidents during 1967—one right-angle and two rear-end collisions. Therefore, the effect of these changes is probably negligible with regard to accidents.

The most important effect of the signal revisions appears to be the driver convenience produced by the substantial decrease in the number of vehicles stopped for longer than one cycle, especially during the evening rush hour. The total number of vehicles not clearing one cycle before the revision was 232 on both days of measurement during the PM peak hours, which was reduced to only two vehicles not clearing one cycle during the same time period after implementing the revision. The elimination of the frustrating experience of waiting through a portion of a red interval, not clearing the intersection during the green interval, and being forced to wait through another red interval is important in consideration of driver discomfort. Driver impatience under these conditions is indicated by the large number of signal violations observed at this location.

Revision of Signal Timing at Central Avenue and West Market Street—Experiment A69

Experiment A69 investigates the engineer's ability to control a driver's choice of route by giving preferential treatment to one of two competing traffic movements. In addition, the work that is done in this experiment prepares for further improvement in a later experiment. Experiment A69 proposes a revision of signal timing at Central Avenue and West Market Street in an attempt to attract more drivers to selection of the Central Avenue route for trips to and from downtown Newark. The experiment to follow investigates the benefits of establishing a signal progression for eight signals on Central Avenue. Inasmuch as this intersection is critical in establishing a "green" band for Central Avenue, it would be highly desirable to have drivers use Central Avenue for their trip, thereby becoming part of the platoons passing through this intersection. Even though the through movement on Central Avenue is quite heavy, often developing queues at this intersection, additional through traffic on Central Avenue is to be preferred to having this traffic use West Market Street. A large portion of the West Market Street movement makes a left turn to Central Avenue eastbound, further complicating movement at this congested intersection. Because West Market Street turns to almost parallel Central Avenue within three or four blocks on each side of this intersection, it offers a unique opportunity to test the ability of the engineer to divert traffic in such a situation by allocation of "green" time to each approach, in this case favoring Central Avenue and hindering movement on West Market Street. Of course, a mandatory NO LEFT TURN sign could have been used, but it was considered more desirable to test a situation where the choice could be made by the driver.

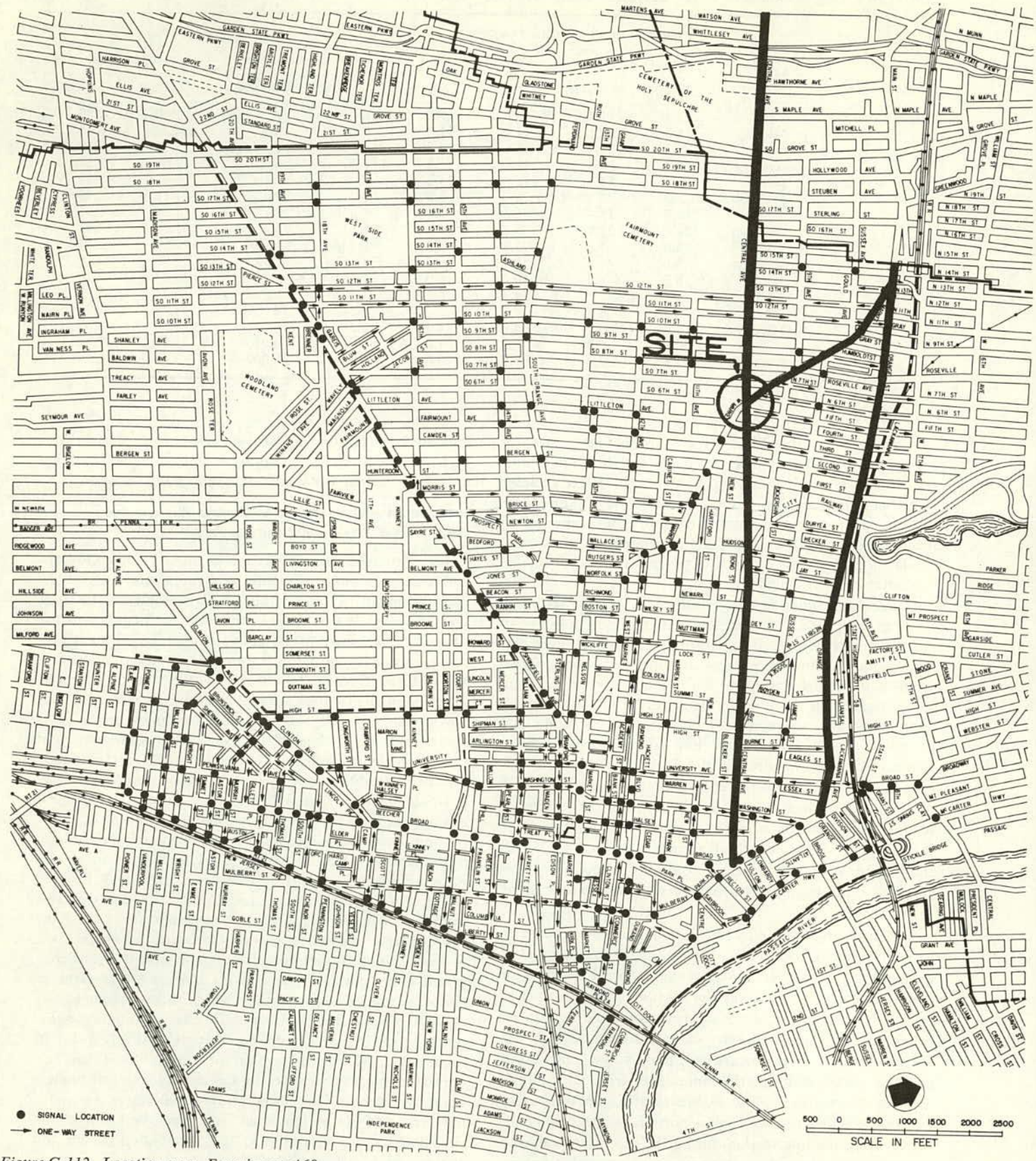


Figure G-112. Location map, Experiment A69.

Eastbound traffic within the traffic corridor has a choice of three arterial streets: Central Avenue, Orange Street, and West Market Street (Fig. G-112). Construction of an expressway parallel to Orange Street has periodically restricted the flow of traffic on Orange Street, diverting additional traffic to West Market Street and Central Avenue. Most of the temporarily diverted traffic moves along West Market Street to Central Avenue and then turns left onto Central Avenue eastbound.

Experimental Area

Central Avenue, west of the intersection with West Market Street, varies from 60 to 68 ft in width and is marked for three lanes in each direction. East of the intersection the street is 50 ft wide and is marked for two lanes in each direction. Parking is prohibited on the southerly side from 7 to 9 AM, providing three eastbound lanes for moving vehicles west of the intersection and two lanes east of the intersection.

West Market Street is 40 ft wide north of the intersection and marked for four lanes of traffic. South of the intersection the street is of irregular width, varying from 55 to 72 ft, and marked for three lanes northbound and two lanes southbound. Parking is prohibited on the west side of the street during the morning peak hour, providing two lanes for moving vehicles (Figs. G-113 and G-114).

The traffic signal is under the jurisdiction of the Essex County Highway Department. The controller has a single dial, operating on a 90-sec cycle. Central Avenue has 52 percent of the cycle and West Market Street has 48 percent of the cycle (Fig. G-115).

Experimental Design

A preliminary intersection count for a morning peak period from 7 to 9 AM indicated that the peak 1-hr volume occurred between 7:30 and 8:30 AM (Fig. G-116).

The "green" time required for each phase at 95-percent acceptance is as follows:

PHASE	CRITICAL APPROACH	EQUIVALENT VOLUME	REQUIRED SECONDS	GREEN TIME (%)
A	Central Ave. EB	1954	65.7	74
B	West Market St. SB	1273	70.2	78

A new signal timing of 70 percent for Central Avenue and 30 percent for West Market Street was proposed to the Essex County Highway Department for morning peak-period operation. A second dial and a time clock were subsequently installed by the county.

As a measure of the effect of the change in signal split, the number of vehicles stopped during the red interval and the number of vehicles clearing the green interval were to be measured on each approach to the intersection.

The "before" counts were made on March 4, 5, and 6; the "after" survey was conducted on March 18, 19, and 20.

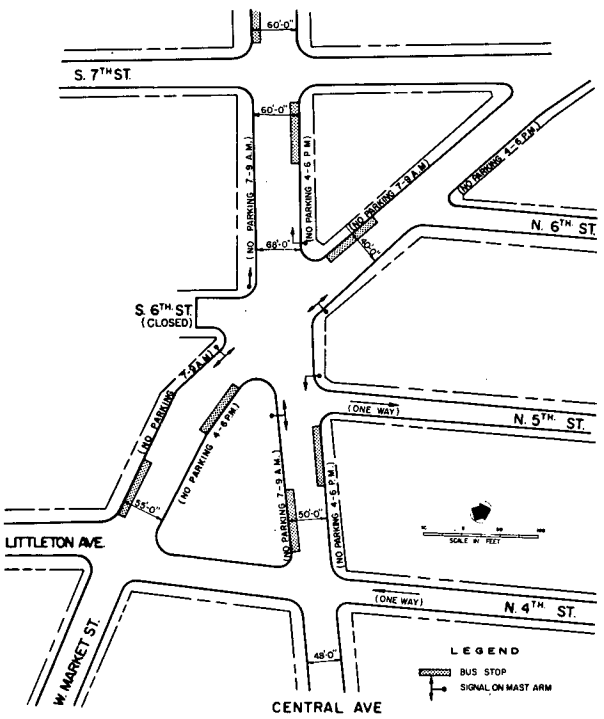


Figure G-113. Traffic controls, Central Avenue and West Market Street.

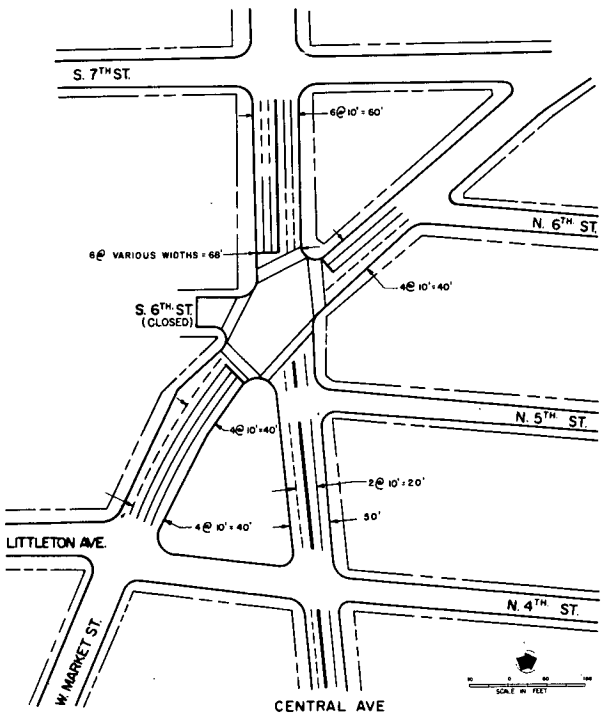


Figure G-114. Lane markings, Central Avenue and West Market Street.

TRAFFIC SIGNAL TIMING

CENTRAL AVENUE	G	A	R	R	R
WEST MARKET ST.	R	R	R	G	A
TIME-"BEFORE" (%)	44	4	4	44	4
TIME-"AFTER" (%)	62	4	4	26	4

Cycle = 90 Seconds

TRAFFIC SIGNAL OPERATION

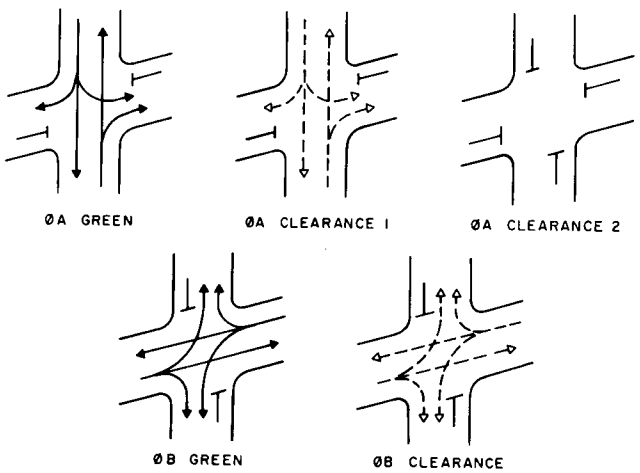


Figure G-115. Traffic signal timing and operation, Central Avenue and West Market Street.

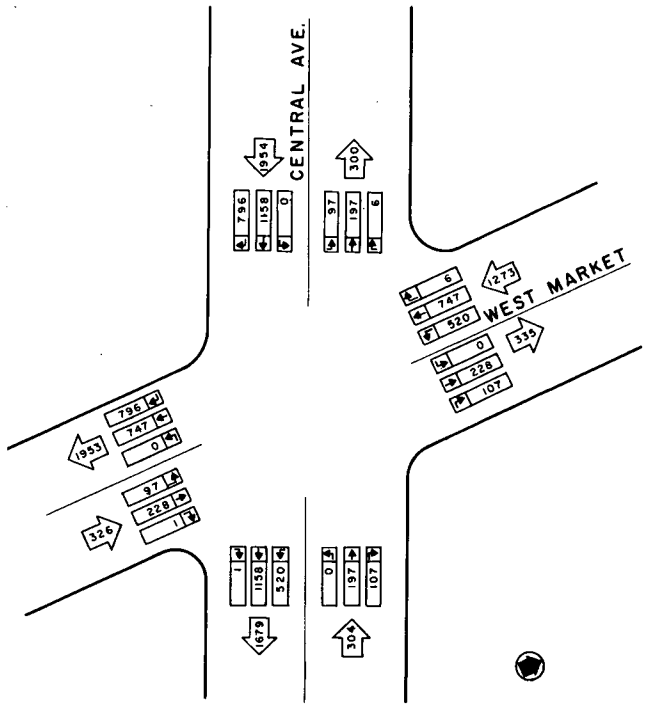


Figure G-116. Peak-hour flow, Central Avenue and West Market Street.

Analysis and Conclusions

The variables analyzed from the intersection data taken between 7:30 and 8:30 AM for three days on all four approaches were (1) the number of vehicles stopped per cycle, (2) the number of vehicles through per cycle, and (3) the stop ratio [(1)/(2)]. The last term was analyzed in percentage form. A stop ratio greater than 100 indicates that more vehicles arrived during a particular red signal phase than were cleared on the subsequent green signal phase. The analysis of these variables was performed using the ANOVA technique.

It was determined that no changes were made in either the number of vehicles through per cycle or the number of vehicles stopped per cycle on the outbound approaches to the intersection (i.e., Central Avenue westbound and West Market Street northbound. Significant differences, however, were realized on the two inbound approaches to the intersection. The results of the analysis on the inbound approaches are summarized in Table G-89.

TABLE G-89

SUMMARY OF ANALYSIS

VARIABLE	MEAN VALUES "BEFORE"	MEAN VALUES "AFTER"	DIFFERENCE (%)	SIG. @ $\alpha=0.05$
(*a*) Central Ave. EB				
Vehicles stopped per cycle	33.3	16.2	−51.4	Yes
Vehicles through per cycle	44.3	48.9	+10.4	Yes
Stop ratio per cycle	75.3	33.1	−56.0	Yes
(*b*) West Market St. SB				
Vehicles stopped per cycle	13.1	34.1	+160.3	Yes
Vehicles through per cycle	22.3	18.2	−18.4	Yes
Stop ratio per cycle [a]	—	—	—	—

[a] Because the number of stopped vehicles could not be accurately recorded during the "after" stage, accurate stop ratios could not be calculated.

The number of vehicles stopped per cycle, indicated on the southbound West Market Street approach, is merely an approximate representation of conditions at the intersection, because the number of vehicles stopped per cycle was recorded only back to the next signalized intersection, which was as far as the observer could see. Comments on the field sheets indicated that queues extended considerably farther than was recorded. The average number of vehicles stopped per cycle on West Market Street, therefore, gives only an indication of the increased delays caused by the revision of the signal timing.

As indicated by the percentage changes in Table G-89, the traffic flow on Central Avenue has been improved at the expense of the southbound West Market Street traffic. The decrease of 51.4 percent in the number of vehicles stopped per cycle on Central Avenue has resulted in an increase of 160.3 percent for the same variable on the West Market Street southbound approach. Although this trend indicates a worsened condition for the intersection, the purpose of the experiment—to improve the Central Avenue approach in preparation for implementation of a progression—was accomplished. A significant trend of diversion was detected from the analysis. This shift in traffic from West Market Street to Central Avenue during the AM time period is indicated by the increase in the number of vehicles through per cycle on the Central Avenue eastbound approach and the corresponding decrease on the West Market Street southbound approach. In the peak hour, approximately 160 drivers apparently elected to use the Central Avenue route during the "after" period who had used the West Market Street route during the "before" period. Of these 160 vehicles, about 42 were eliminated from the left turn from southbound West Market Street to Central Avenue.

Actuated to Fixed-Time Control, Main Street and Second Street—Experiment D20

Progressive timing of traffic signals has been a useful tool to permit traffic to move more freely. Experiment D20 deals with the replacement of a fully actuated controller with a fixed-time controller so that the traffic signal timing at the subject intersection could be incorporated into the progressive systems on both intersecting streets.

Experimental Area

Experiment D20 was performed at the intersection of Main and Second Streets in Louisville (Fig. G-117). Main Street is a 61-ft-wide arterial street permitting one-way travel in a westbound direction. Parking is prohibited on both sides of Main Street east of the intersection between the hours of 7 AM and 6 PM and prohibited at all times on the north side, west of the intersection (Fig. G-118). West of the intersection, stopping is not permitted at any time on the south side of Main Street; nor is it permitted from 7 to 9 AM and 4 to 6 PM on the north side.

Second Street is a 42-ft-wide roadway permitting one-way travel in a northbound direction and intersecting Main Street at the approach to the Clark Memorial Bridge (Fig. G-118). Parking is not permitted at any time, and stopping is restricted between the hours of 7 to 9 AM and 4 to 6 PM on the west side of the street. Stopping is not permitted at any time on the east side of Second Street.

During the "before" period, traffic signals were controlled by a fully actuated controller using radar to detect vehicles approaching the intersection from Main Street, Second Street, and the Clark Bridge. This apparatus was replaced by a single-dial, fixed-time controller prior to the "after" period. The timing was adjusted to permit progressive movement on both Main Street and Second Street.

Design of Experiment

It was not possible to gather data for Experiment D20 until July 1968, because the Clark Bridge was closed for repairs until then.

Review of speed and delay runs made between July and September 1968 indicated that northbound Second Street traffic was being delayed, per run, for about 19 sec during the AM and 25 sec in the PM owing to the traffic signal at Main Street. During the same period, westbound Main Street traffic experienced delays, per run, of 1 sec and 11 sec during the morning and afternoon peak periods, respectively.

A field inspection of the site revealed that all detectors were functioning properly except that two of the three treadle detectors on Main Street had been removed. After the matter was discussed with representatives of the Louisville and Jefferson County Department of Traffic Engineering, the third treadle was disconnected, and two radar detectors were installed over the Main Street approach. The installation was put into full operation on November 4, 1968.

Time-space diagrams were drawn for Main Street and Second Street (Figs. G-119 and G-120). These illustrations were used to detail the allowable offsets as shown for the signals at Main Street and Second Street.

The signal timing given in Table G-90 was determined by a capacity analysis of the intersection and the peak-hour traffic volumes shown in Figure G-121.

Surveillance System Design

Measurements that were recorded included the following:

1. Number of vehicles through per cycle was recorded by three observers, one located at each of the three approaches. This measurement was classified by vehicle type and turning movement.
2. Number of vehicles stopped on red per cycle was recorded by the same three observers recording the number of vehicles through per cycle.
3. Travel time on Main Street between First Street and Second Street and travel time on Second Street between Market Street and Main Street was recorded by one observer on each street. He recorded the time in seconds for a vehicle to travel between the locations shown in Figure G-121. The vehicles were classified according to cars, light trucks, and heavy trucks.
4. Speed and delay runs were taken on Main Street westbound, Market Street eastbound, Jefferson Street westbound, and Liberty Street eastbound. All these runs were made between Baxter Avenue and Ninth Street.

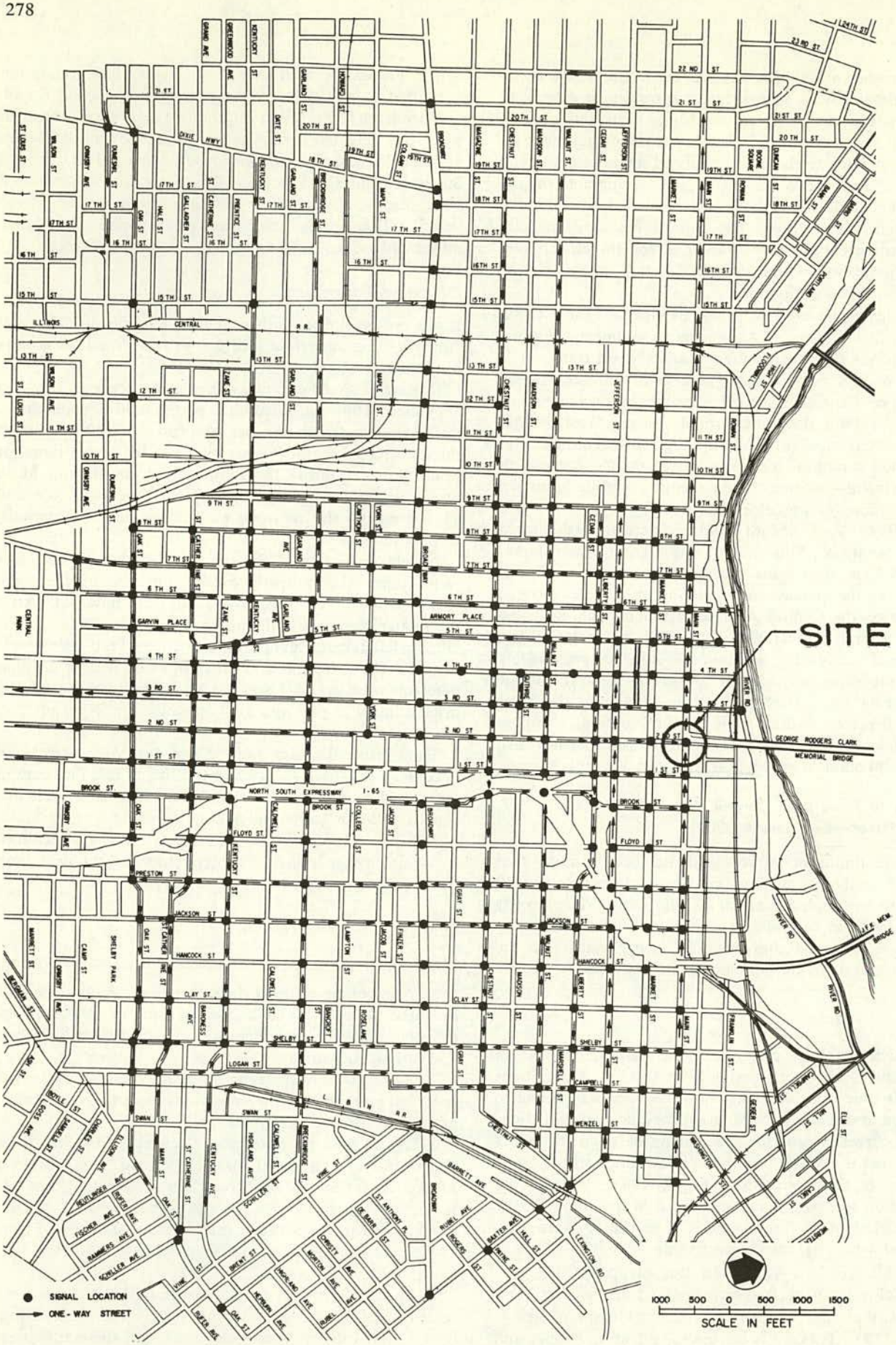


Figure G-117. Location map, Experiment D20.

All vehicle measurements were made between the hours of 7:30 and 8:30 AM; 1:30 and 2:30 PM; and 4:30 and 5:30 PM. Travel time measurements were made between 7:30 and 9 AM; 1:30 and 3 PM; and 4 and 5:30 PM.

Dates of implementation and measurement are as follows:

PERIOD	DATES
"Before"	2/17/69, 2/18/69, 2/19/69
Implementation	3/2/69
"After"	4/28/69, 4/29/69, 4/30/69

Analysis

Comparison of the number of *vehicles through per cycle* indicated no significant difference between the "before" and "after" mean values or variances for the Main Street and Second Street approaches during all time periods, and for the Clark Bridge approach during the midday and PM time periods. Significant reduction in mean values and variances between the "before" and "after" conditions were determined for the Clark Bridge approach during the AM period. A reduction in variability was expected inasmuch as there were no traffic control devices that could meter vehicles over the bridge from Indiana. This free-flow condition would extend the green phase for many cycles from minimum settings to the maximum allowed by the fully actuated controller. The fixed-time controller allocated the same amount of green time each cycle, thereby limiting the maximum number of vehicles through per cycle to a lesser value than was possible with the fully actuated controller.

The number of vehicles through was reduced from a mean "before" value of 22.1 vehicles to a mean "after" value of 20.5 vehicles, a reduction of 1.6 vehicles per cycle. This reduction was due to a shift of former Clark Bridge traffic during the AM time period to the I-65 off-ramp at Brook Street and Jefferson Street. Improvements were made to the I-65 off-ramp on March 18 and 19, 1969.

Analysis of the number of *vehicles stopped on red per cycle* on the Main Street approach indicated a significant

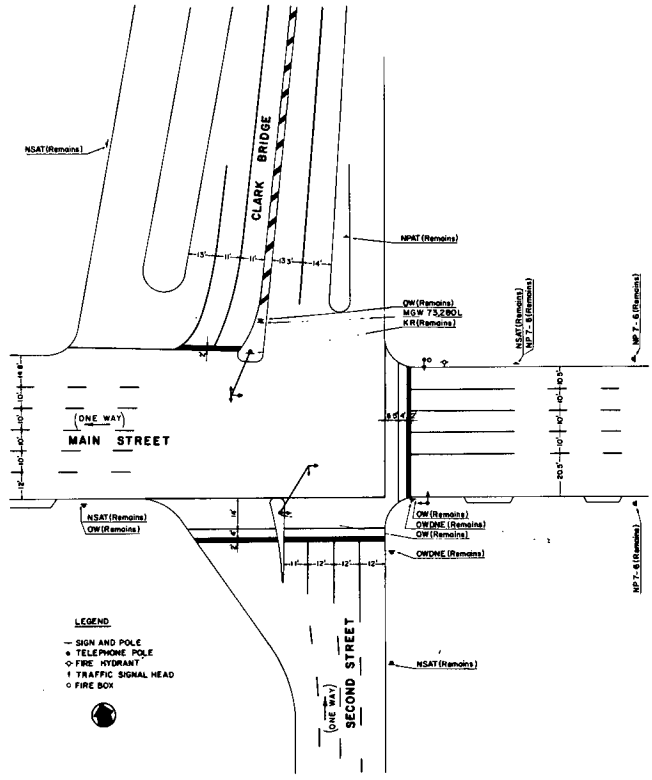


Figure G-118. Vicinity map.

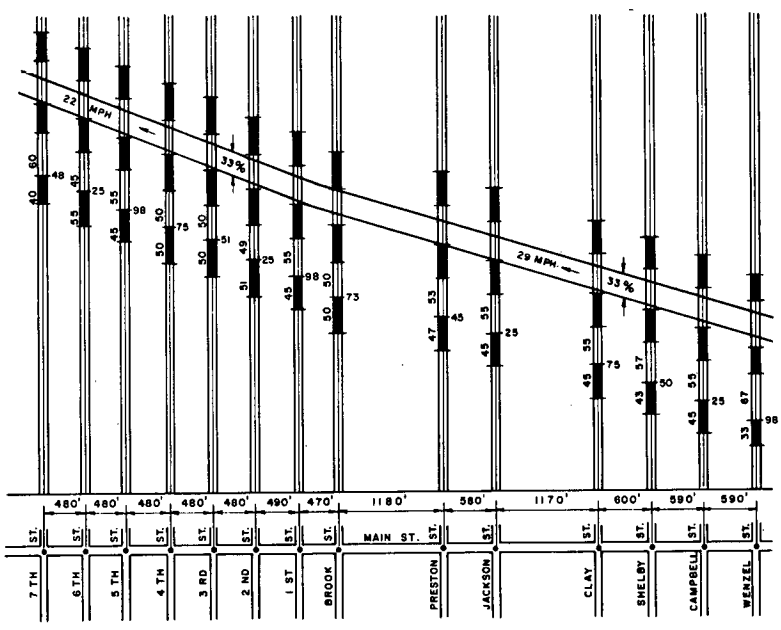


Figure G-119. Time-space diagram, Main Street.

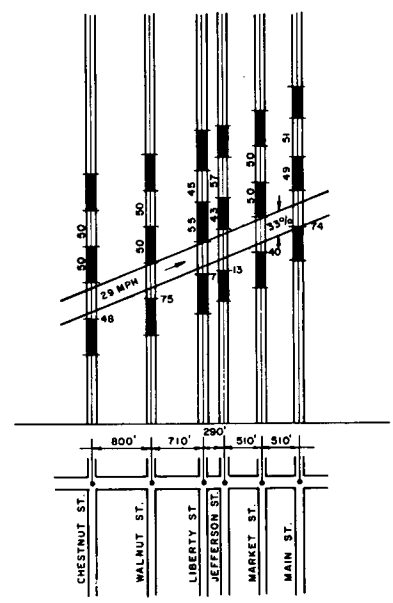


Figure G-120. Time-space diagram, Second Street.

TABLE G-90

MAIN STREET AND SECOND STREET SIGNAL TIMING

APPROACH	SIGNAL TIMING (SEC) GREEN	AMBER	RED
Main St.	26	3	31
Second St. and	28	3	29
Clark Bridge	28	3	29

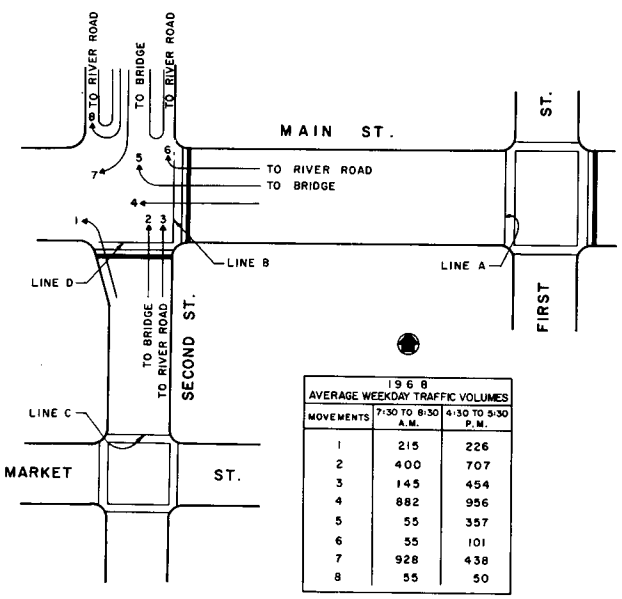


Figure G-121. Traffic measurements.

reduction in the mean values for the AM and PM periods, as given in Table G-91. A significant reduction in variability was also observed during all time periods (Table G-92). The only other approach that showed a significant reduction in the means and variabilities of the number of vehicles stopped per cycle was the Clark Bridge approach during the AM period, also given in Tables G-91 and G-92. This latter reduction was expected, because the number of vehicles through was also reduced.

Travel time comparisons of movements A to B and C to D (Fig. G-121) were made only for autos. Table G-93 gives the "before" and "after" mean values. As indicated, many of the "before" and "after' means are statistically different; however, only the following have time differences that are meaningful:

MOVEMENT	CONDITION	TIME PERIOD	DIFFERENCE (SEC)
A to B	With signal delay	AM	−15.7
		PM	−11.8
C to D	With signal delay	Midday	−7.4

Many of the other differences were statistically significant because of the large number of observations that were taken.

Table G-94 gives the occurrence of signal delay during the "before" and "after" conditions recorded with the stopwatch data given in Table G-93. As indicated, the occurrence of signal delay has been significantly reduced for movement A to B during the AM and PM periods and for movement C to D during the midday and PM periods.

Speed and delay run data taken on Main Street during the "before" and "after" periods were analyzed to determine the effect of the change on total travel time, number of stops per run, and delay time per stop.

A significant ($\alpha = 0.05$) reduction of 10.4 percent in travel time and 56.2 percent in number of stops was realized during the AM period (Table G-95). Changes in the mean values for the other time periods of travel time and number of stops and all time periods of delay per stop, when considering the total run, were not significantly different. Table G-95 also gives the delay per stop at Second Street, where substantial reductions were determined between the "before" and "after" mean values for AM and PM. In fact, no stops were recorded at Second Street in the "after" period except for one 35-sec stop during the midday period. Analysis of the occurrence of a stop at Second Street (Table G-96) indicated a significant reduction in the number of stops between the "before" and "after" conditions for the AM and PM periods.

Conclusions

Based on the foregoing analysis, it is concluded that:

1. The mean number of vehicles stopped on red during the AM and PM periods on the Main Street approach showed significant decreases between the "before" and "after" measurements.
2. The variability of the number of vehicles stopped on red on the Main Street approach during all time periods was significantly reduced from the "before" condition to the "after" condition.
3. Analysis of travel time of vehicles on the immediate approach blocks to the intersection of Main Street and Second Street on both those streets indicated a substantial reduction in travel time for those vehicles not encountering signal delay on Main Street during the AM and PM periods and on Second Street during the midday period.
4. The occurrence of a vehicle encountering signal delay in the direction of and during peak traffic flows was reduced on Main Street and Second Street.
5. Speed and delay run data on Main Street indicated a significant ($\alpha = 0.05$) reduction in travel time and number of stops during the AM period. They also indicated a large reduction in the occurrence of stops at Second Street during the AM and PM periods.

Pedestrian Control at Halsey Street, Academy Street, and Bank Street—Experiment A31

Experiment A31 examines the effects produced on pedestrian and vehicular traffic when some signals in an interconnected traffic signal system operate on a cycle length of one-half the system cycle length.

The experiment was conducted at the intersections of

TABLE G-91

VEHICLES STOPPED ON RED PER CYCLE, MEAN VALUES

APPROACH	TIME PERIOD	"BEFORE" MEAN (VEH)	"BEFORE" NO. OF OBS.	"AFTER" MEAN (VEH)	"AFTER" NO. OF OBS.	DIFFERENCE
Main St.	AM	9.5	164	1.2	180	−8.3(87.4%)
	PM	10.6	168	3.7	177	−6.9(65.1%)
Clark Bridge	AM	9.7	164	7.8	180	−1.9(20.0%)

Bank and Academy Streets with Halsey Street. These intersections, in the shopping center of Newark, are located between two major east-west arterial streets—Raymond Boulevard on the north and Market Street on the south (Figs. G-122 and G-123).

Experimental Area

Halsey Street is a one-way southbound collector street that is 33 ft wide, marked for three lanes of traffic, and has local transit service. Bank Street is a one-way eastbound local street that is 22 ft wide, and Academy Street is a one-way westbound local street that is 22 ft wide. Parking is generally prohibited on both sides of these streets at all times; stopping or standing is prohibited on the right side of each street in the direction of traffic flow during the 7 to 9 AM and 4 to 6 PM periods (Figs. G-124 and G-125).

Pedestrian and vehicular traffic volumes for a typical evening peak hour (5 to 6 PM) in 1969 at the Academy Street and Halsey Street intersection are shown in Figure G-126. During the peak evening period, when the pedestrians occupy the south crosswalk, a right-turning vehicle on Bank Street delays all following vehicles. Pedestrians cross both streets without regard to the traffic signals whenever the moving vehicles have passed. The traffic signals are part of the PR system in the city of Newark and operate on a basic 90-sec cycle.

TABLE G-92

VEHICLES STOPPED ON RED PER CYCLE, POOLED VARIANCES

APPROACH	TIME PERIOD	NO. OF VEHICLES "BEFORE"	NO. OF VEHICLES "AFTER"	*F* RATIO
Main St.	AM	38.03	1.68	22.64
	Midday	4.42	2.08	2.12
	PM	49.45	8.21	6.02
Clark Bridge	AM	41.65	21.76	1.91

TABLE G-93

TRAVEL TIME, STOPWATCH DATA

MOVEMENT	CONDITION	TIME PERIOD	MEAN VALUE (SEC) "BEFORE"	MEAN VALUE (SEC) "AFTER"	MEAN VALUE (SEC) DIFFERENCE	SIG. @ $\alpha=0.05$ LEVEL
A to B	Without signal delay	AM	14.9	13.5	−1.4 (9.4%)	Yes
		Midday	12.2	14.4	+2.2(18.0%)	Yes
		PM	12.3	14.1	+1.8(14.6%)	Yes
	With signal delay	AM	32.2	16.5	−15.7(48.8%)	Yes
		Midday	30.0	27.6	−2.4 (8.0%)	No
		PM	32.4	20.6	−11.8(36.4%)	Yes
C to D	Without signal delay	AM	13.4	14.6	+1.2 (9.0%)	Yes
		Midday	14.5	15.0	+0.5 (3.4%)	No
		PM	15.1	17.9	+2.8(18.5%)	Yes
	With signal delay	AM	31.6	32.4	+0.8 (2.5%)	No
		Midday	31.6	24.2	−7.4(23.4%)	Yes
		PM	31.7	32.6	+0.9 (2.8%)	No

Design of Experiment

Because PR local controllers can maintain progression only if they operate on the system cycle length, those at the two study intersections were removed and replaced with pretimed local controllers with 90-sec gears.

During the "before" period the dial keys were set to provide the same timing that was provided by the PR controllers. In the "after" period the 45-sec cycle was imple-

TABLE G-94

OCCURRENCE OF SIGNAL DELAY, STOPWATCH DATA

MOVEMENT	TIME PERIOD	CONDITION	SIGNAL DELAY	NO SIGNAL DELAY	ALL	CHI SQUARE TEST SIG. @ $\alpha=0.05$ LEVEL
A to B	AM	"Before"	151	72	223	
		"After"	21	210	231	
		All	172	282	454	Yes
	Midday	"Before"	47	222	269	
		"After"	28	198	226	
		All	75	420	495	No
	PM	"Before"	168	138	306	
		"After"	55	200	255	
		All	223	338	561	Yes
C to D	AM	"Before"	127	87	214	
		"After"	133	111	244	
		All	260	198	458	No
	Midday	"Before"	95	87	182	
		"After"	24	146	170	
		All	119	233	352	Yes
	PM	"Before"	130	83	213	
		"After"	44	167	211	
		All	174	250	424	Yes

TABLE G-95

TRAVEL CHARACTERISTICS, SPEED AND DELAY DATA

VARIABLE	TIME PERIOD	"BEFORE" MEAN	"BEFORE" NO. OF OBS.	"AFTER" MEAN	"AFTER" NO. OF OBS.
Travel time (sec)	AM	300.2	30	268.8	9
	Midday	287.3	23	298.5	8
	PM	297.5	24	316.8	6
No. of stops	AM	1.6	31	0.7	9
	Midday	1.2	23	0.9	8
	PM	1.6	24	1.8	6
Delay per stop (total run) (sec)	AM	21.1	26	16.0	5
	Midday	20.4	17	21.8	6
	PM	20.8	22	17.3	5
Delay per stop (at Main St.) (sec)	AM	14.8	9	0	0
	Midday	22.4	5	35	1
	PM	19.4	8	0	0

TABLE G-96

NUMBER OF STOPS AT SECOND STREET, SPEED AND DELAY DATA

TIME PERIOD	CONDITION	STOPPED AT SECOND ST. YES	NO	ALL	SIG. @ $\alpha=0.05$ LEVEL
AM	"Before"	9	21	30	
	"After"	0	9	9	
	All	9	30	39	Yes
Midday	"Before"	5	19	24	
	"After"	1	7	8	
	All	6	26	32	Yes
PM	"Before"	8	16	24	
	"After"	0	6	6	
	All	8	22	30	Yes

mented through the addition of dial keys, so that one revolution of the dial in 90 sec produced two revolutions of the camshaft (i.e., two 45-sec signal cycles).

The system reference for the interconnection was maintained by the use of the amber phase for the Market Street approach to Halsey Street. A fail-safe device was installed in each local controller to permit the controller to operate without interconnection in case of a failure in the interconnect cable or an excessive resynchronization period resulting from a change in cycle length in the traffic-responsive controller at the Market Street and Halsey Street intersection.

"Before" and "after" signal timings are shown in

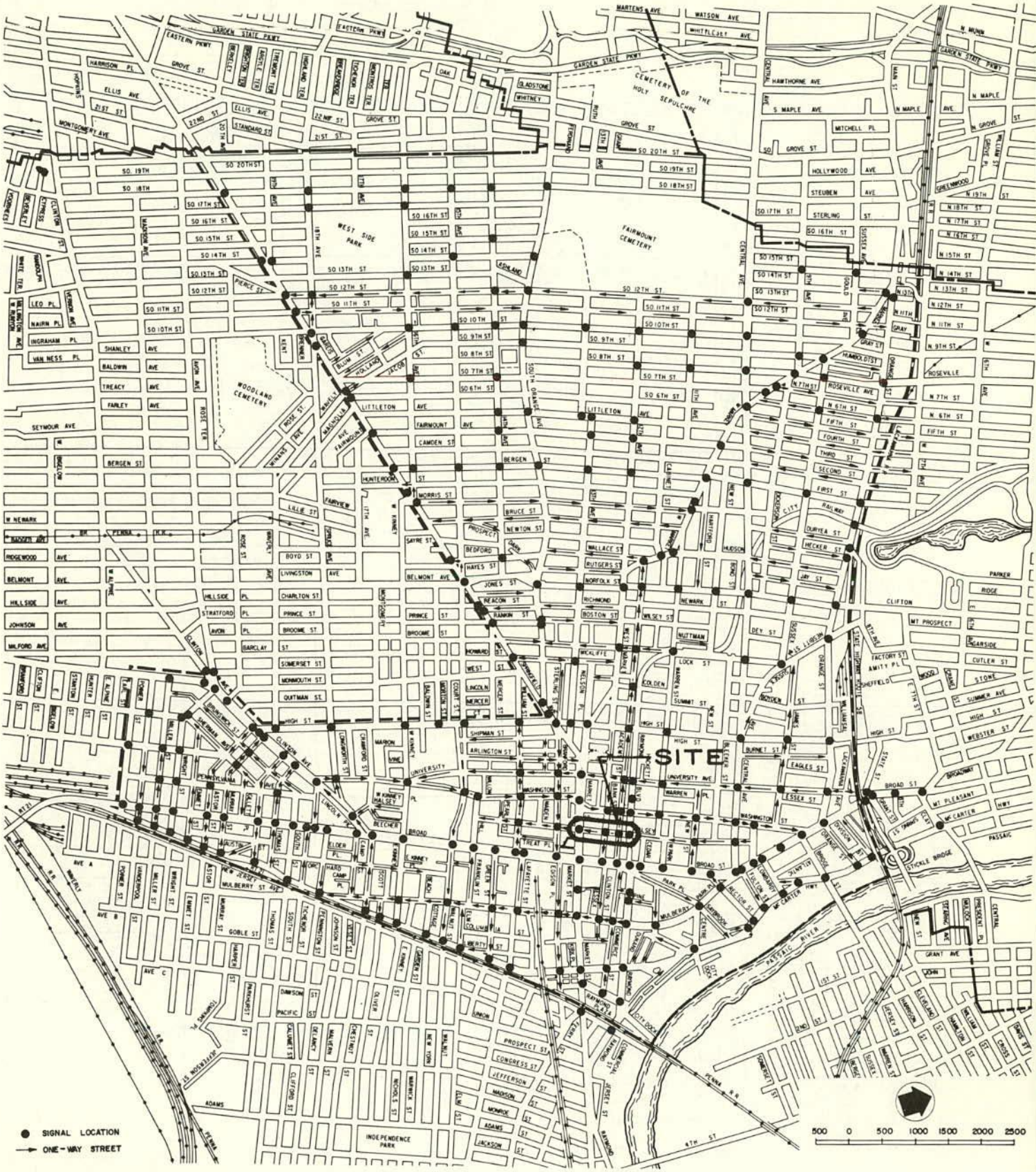


Figure G-122. Location map, Experiment A31.

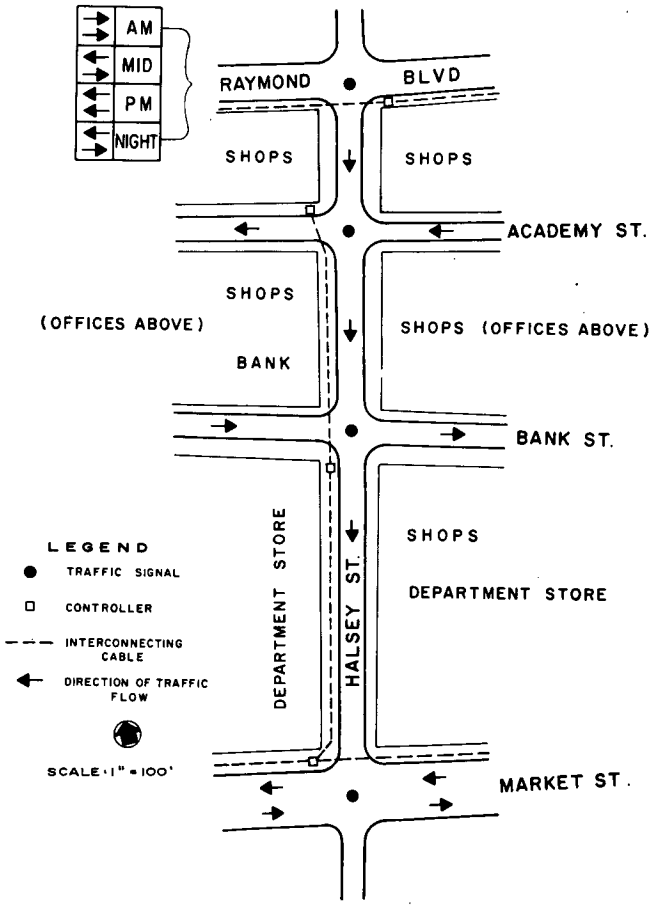


Figure G-123. Area plan.

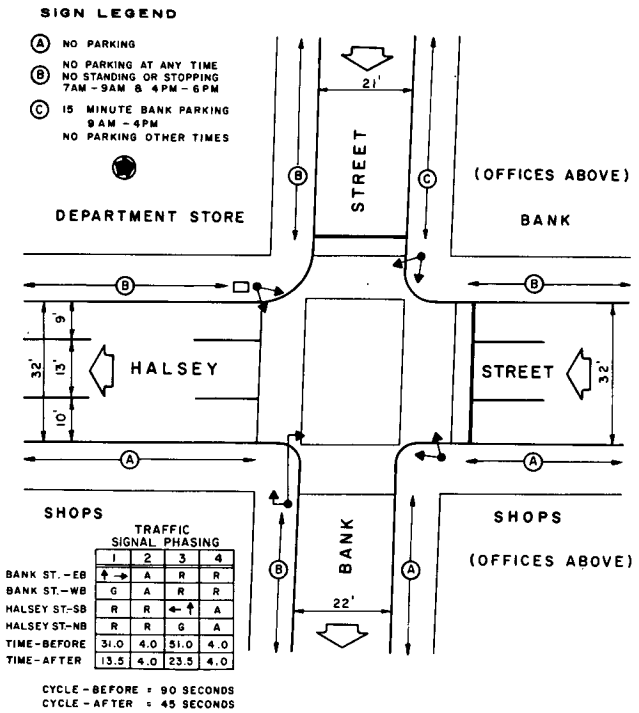


Figure G-124. Vicinity map, Halsey Street and Bank Street.

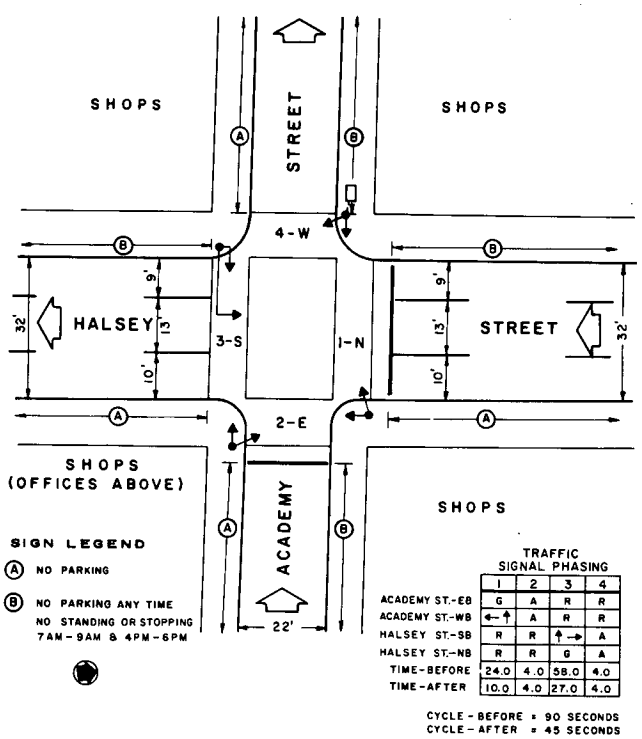


Figure G-125. Vicinity map, Halsey Street and Academy Street.

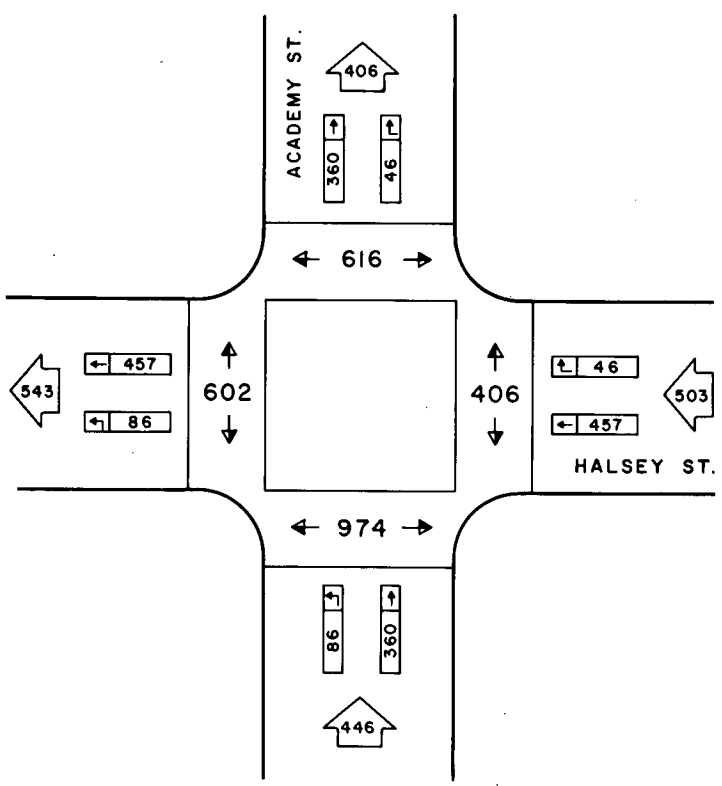


Figure G-126. 1969 traffic volumes, typical evening peak hour (4:30 to 5:30 PM).

Figures G-124 and G-125. Time-space diagrams of both "before" and "after" stages are shown in Figure G-127.

The surveillance system of traffic measurements included:

1. Manually counting the number of vehicles through per cycle and the number of vehicles stopped on red per cycle at each approach to the intersections of Halsey Street with Bank Street and Halsey Street with Academy Street.

2. Use of the Marbelite Data Compiler for speed and delay runs on Halsey Street between Raymond Boulevard and Market Street, on Bank Street between Washington Street and Broad Street, and on Academy Street between Broad Street and Washington Street.

3. Use of time-lapse photography to record pedestrian activity. Restrictions in camera location limited the photography to the intersection of Halsey Street with Academy Street.

Measurements were taken between 4:30 and 5:30 PM on the following dates (1969):

	DATES, BY MEASUREMENT		
CONDITION	MANUAL COUNTS	SPEED AND DELAY RUNS	TIME-LAPSE PHOTOGRAPHY
"Before"	1/27 (Day 1)	3/11	1/27 (Day 1)
	2/20 (Day 3)	3/12	2/20 (Day 3)
	3/11 (Day 2)		3/11 (Day 2)
"After"	3/20 (Day 3)	3/25	3/20 (Day 3)
	3/25 (Day 2)	3/26	3/25 (Day 2)
	4/7 (Day 1)		4/7 (Day 1)

Extremely cold weather and snow disrupted the schedule during the "before" condition, resulting in the measurements being taken between January and March.

Analysis

Comparison of the "before" and "after" measurements of the number of pedestrians crossing during the proper and improper signal intervals at the intersection of Halsey Street and Academy Street indicated a significant reduction in illegal movement across certain crosswalks (Table G-97). Pedestrian movement at crosswalk 2-E, S-3, and 4-W showed a reduction in illegal movement of 42 percent, 25 percent, and 21 percent, respectively.

The number of vehicles through each intersection by approach is given in Table G-98. A chi square analysis of each approach indicated that only the approaches to the Academy Street and Halsey Street intersection were significantly different between "before" and "after" by day. Based on further analysis, it was concluded that the volume measurement on Day 3 was different from Days 1 and 2.

The number of vehicles stopped on red is given in Table G-99. In all but one day the number of vehicles stopped in the "before" period was less than the number of vehicles stopped in the "after" period. Day 2 measurements on the Halsey Street approach to Bank Street were different, because the signal offset during the "before" condition between Academy Street and Bank Street was 5 sec instead of 0 sec. These measurements were excluded from further analysis. To eliminate any effect the variation in the

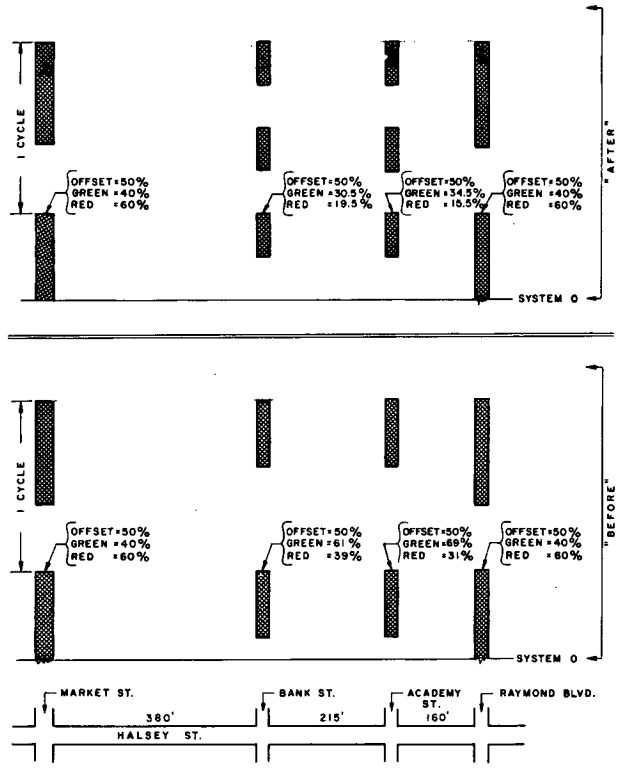


Figure G-127. Time-space diagram, Halsey Street.

TABLE G-97

NUMBER OF PEDESTRIANS CROSSING, CHI SQUARE ANALYSIS

CROSS-WALK [a]	CONDITION	NO. OF PEDESTRIANS PROPER	IMPROPER	ALL
1-N	"Before"	1058	173	1231
	"After"	976	160	1136
	All	2034	333	2367
2-E [b]	"Before"	1838	774	2612
	"After"	2793	449	3242
	All	4631	1223	5854
3-S [b]	"Before"	1469	369	1838
	"After"	1537	275	1812
	All	3006	644	3650
4-W [b]	"Before"	1273	691	1964
	"After"	1483	548	2031
	All	2756	1239	3995

[a] Crosswalk locations are defined in Fig. G-125.
[b] Significant change at $\alpha = 0.05$ level.

TABLE G-98

VEHICLES THROUGH EACH INTERSECTION

APPROACH	CONDITION	VEHICLES THROUGH, BY DAY 1	2	3	ALL
Academy St. at Halsey St.	"Before"	421	415	333	1169
	"After"	391	382	405	1178
	All	812	797	738	2347
Halsey St. at Academy St.	"Before"	563	509	461	1533
	"After"	509	467	511	1487
	All	1072	976	972	3020
Bank St. at Halsey St.	"Before"	280	288	282	850
	"After"	293	277	282	852
	All	573	565	564	1702
Halsey St. at Bank St.	"Before"	583	515	477	1575
	"After"	522	475	518	1515
	All	1105	990	995	3090

TABLE G-99

VEHICLES STOPPED ON RED

APPROACH	CONDITION	VEHICLES STOPPED, BY DAY 1	2	3	ALL
Academy St. at Halsey St.	"Before"	340	290	180	810
	"After"	481	353	732	1566
	All	821	643	912	2376
Halsey St. at Academy St.	"Before"	77	42	43	162
	"After"	276	306	443	1025
	All	353	348	486	1187
Bank St. at Halsey St.	"Before"	243	193	138	574
	"After"	307	207	238	752
	All	550	400	376	1326
Halsey St. at Bank St.	"Before"	180	216	183	579
	"After"	373	187	387	947
	All	553	403	570	1526

TABLE G-100

PERCENTAGE OF VEHICLES STOPPED

APPROACH	VEHICLES STOPPED (%) "BEFORE"	"AFTER"	DIFFERENCE
Academy St. at Halsey St.	75.4	107.9	+32.5 [a]
Halsey St. at Academy St.	11.1	59.6	+48.5 [a]
Bank St. at Halsey St.	67.5	88.3	+20.8
Halsey St. at Bank St.	34.2	73.1	+38.9 [b]

[a] Data for Days 1 and 2 only.
[b] Data for Days 1 and 3 only.

number of vehicles through might have on the analysis of the number of vehicles stopped on red, ratios of the percentage of vehicles stopped were calculated for each approach (Table G-100). In all cases the percentage of vehicles stopped increased significantly ($\alpha = 0.05$).

Analysis of "before" and "after" travel times indicated a significant reduction in trip time on Bank Street and Academy Street (Table G-101). Even though the Halsey Street travel times were not significantly ($\alpha = 0.05$) different, the mean "after" value was less than the mean "before" value.

Table G-102 gives the number of runs with a certain number of stops determined from the speed and delay data. Individual chi square analyses indicated no significant differences ($\alpha = 0.05$) between "before" and "after."

An ANOVA of the delay time per stop indicated significant differences between the "before" and "after" means (Table G-103).

Conclusions

Based on the foregoing analysis, it is concluded that:

1. Illegal pedestrian movement during the red signal phase was reduced by 21 to 42 percent at three out of the four crosswalks studied.
2. The number of vehicles through did not change be-

TABLE G-101

TRAVEL TIME, SPEED AND DELAY DATA

STREET	TRAVEL TIME (SEC) "BEFORE" MEAN	OBS.	"AFTER" MEAN	OBS.	DIFFERENCE	SIG. @ $\alpha=0.05$ LEVEL
Halsey St.	84.1	13	65.4	16	−18.7 (22.2%)	No
Bank St.	124.5	15	88.1	17	−36.4 (29.2%)	Yes
Academy St.	117.7	15	81.4	17	−36.3 (30.8%)	Yes

tween the "before" and "after" conditions at any approach.

3. The percentage of vehicles stopped on red increased significantly at each of the four approaches. The largest increase occurred at the Halsey Street approach to Academy Street.

4. Travel times on Academy Street and Bank Street were reduced by 30.8 and 29.2 percent, respectively. Halsey Street also showed a slight, but not significant, reduction.

5. The speed and delay runs on Academy Street, Bank Street, and Halsey Street indicated no difference in the number of stops per run.

6. Delay time per stop based on speed and delay run data indicated significant ($\alpha = 0.05$) reductions of 40.7, 37.5, and 51.2 percent on Academy Street, Bank Street, and Halsey Street, respectively.

TABLE G-102

NUMBER OF STOPS, SPEED AND DELAY DATA

STREET	CONDITION	FREQUENCY, BY NO. OF STOPS 0	1	2	3	4	ALL
Halsey St.	"Before"	2	8	3	0	0	13
	"After"	7	2	5	2	0	16
	All	9	10	8	2	0	29
Bank St.	"Before"	0	5	7	3	0	15
	"After"	0	5	10	2	0	17
	All	0	10	17	5	0	32
Academy St.	"Before"	0	5	8	1	1	15
	"After"	2	4	9	2	0	17
	All	2	9	17	3	1	32

Signal Rephasing at Central Avenue and High Street—Experiment A40

The optimum allocation of available green time to each conflicting movement at a signalized intersection is always an interesting problem, considering normal variations and fluctuations of traffic patterns. When a large number of left-turning vehicles conflict with opposing through movements, a separate turn phase can often result in improved use of green time. When the turns occur from a one-lane approach, an advance or delayed green interval is often used in place of a separate turn phase.

Experiment A40, located at the intersection of Central Avenue at High Street and Sussex Avenue, is designed to measure the improvement derived from an advance green interval. The advance green interval is to be given to the High Street northbound approach, which has one moving lane with a large number of left turns. During the evening peak traffic periods the volume on this approach is higher than the opposing flow.

Experimental Area

Experiment A40 was conducted at the intersection of Central Avenue, High Street and Sussex Avenue (Fig. G-128).

Central Avenue is an east-west arterial street that provides access to downtown Newark from the west. High Street is a south-north arterial street and is the western limit of the downtown area. Sussex Avenue is a collector road that serves to distribute traffic to both north-south and east-west streets. Both Central Avenue and High Street are two-way; Sussex Avenue, which orginates at High Street, is one-way northwestward for one block. A plan of the intersection is shown in Figure G-129.

The traffic signal at this intersection is under the jurisdiction of the city of Newark (Fig. G-130). The controller is a single-dial, pretimed controller operating on a 90-sec cycle, and maintains a simultaneous offset relationship with the signalized intersections to the north, south, and west. The other signals on Central Avenue, east of High Street, are part of the city of Newark PR system, to which there is no constant offset relationship.

Prior to experimentation, the signal operated with 49 percent of the cycle devoted to Central Avenue and 51 percent devoted to High Street. The volume of traffic during peak hours on each approach is given in Table G-104.

Purpose and Scope

The purpose of Experiment A40 is to determine the effect that an advance green interval will have on delays encountered at the intersection of High Street and Central Avenue.

The experiment is to be confined to the evening peak period when both the total volume and number of left-turning vehicles are higher at the northbound approach.

TABLE G-103

DELAY TIME PER STOP, SPEED AND DELAY DATA

STREET	DELAY TIME PER STOP (SEC) "BEFORE" MEAN	OBS.	"AFTER" MEAN	OBS.	DIFFERENCE	SIG. @ $\alpha=0.05$ LEVEL
Halsey St.	50.6	11	24.7	9	−25.9 (51.2%)	Yes
Bank St.	42.7	15	26.7	17	−16.0 (37.5%)	Yes
Academy St.	41.0	15	24.3	15	−16.7 (40.7%)	Yes

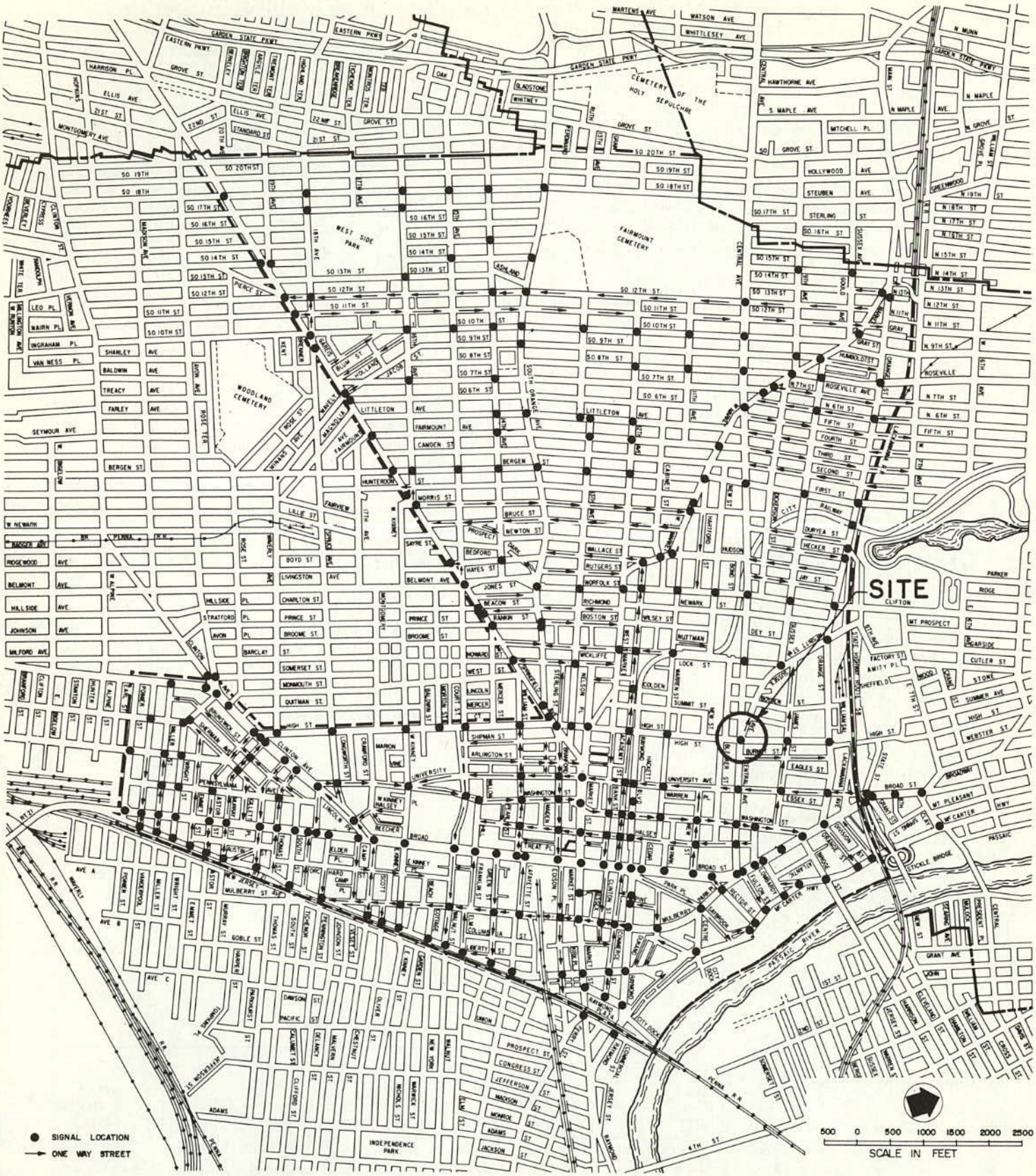


Figure G-128. Location map, Experiment A40.

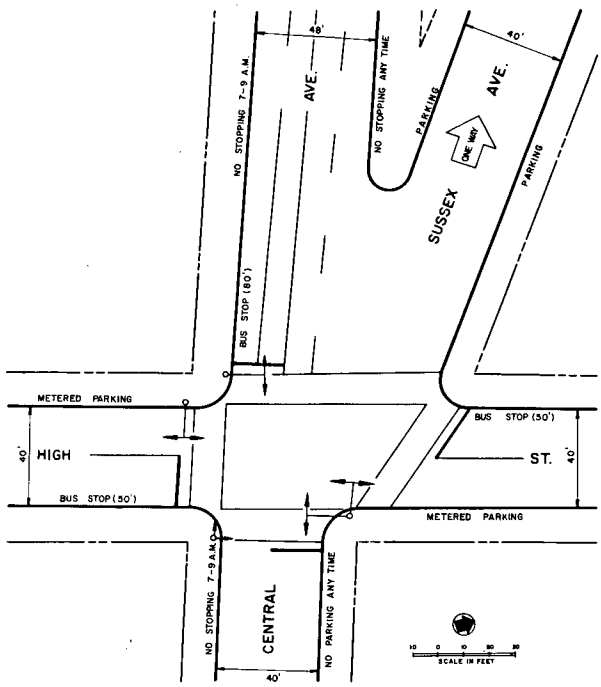
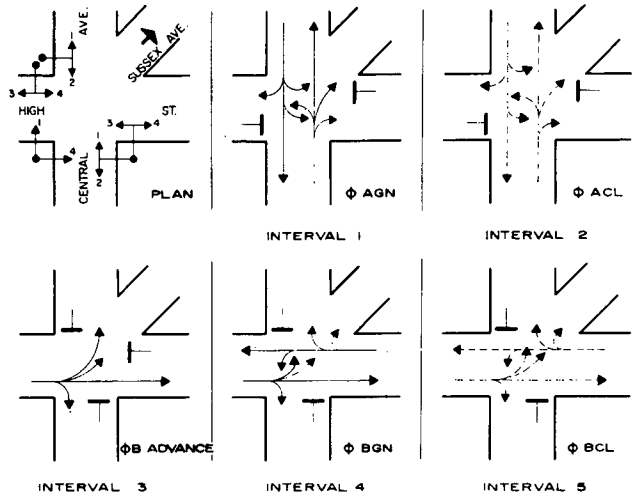


Figure G-129. Vicinity map, Central Avenue and High Street.



STREET	SIGNAL NO.	INTERVAL 1	2	3	4	5
CENTRAL AVE EB	1	G	A	R	R	R
CENTRAL AVE WB	2	G	A	R	R	R
HIGH ST. NB	3	R	R	G	G	A
HIGH ST. SB	4	R	R	R	G	A
"BEFORE"- TIME IN %		44	5	2	44	5
"AFTER"- TIME IN %		44	5	12	34	5

CYCLE = 90 SECONDS

Figure G-130. Traffic signal sequence and timing, Central Avenue and High Street.

Design of Experiment

Preliminary observations of the intersection during evening peak hours disclosed long queues on the northbound High Street approach. At the beginning of the High Street green interval, the presence of a northbound left-turning vehicle often caused considerable delay while the driver awaited a gap in southbound traffic.

The time required for each approach at 95-percent acceptance (i.e., 95 percent of the cycles serve the demand) was computed. Computations were based on the concepts of the Poisson distribution of vehicle arrival during the hour and average minimum headway.

As the northbound advance green interval is not required in the AM peak, and the AM time requirements are very close to the existing split timing, it was proposed to change to a two-dial operation, Dial 1 retaining the original timing with a 2-percent advance green northbound interval. Dial 2, adjusted for the evening peak hour, retained 49 percent on Phase A, and the northbound Phase B was given a 12-percent (or 10.8 sec) advance; southbound Phase B was reduced to 39 percent (or 35 sec).

The two-dial operation, with revised timing, was installed by the city of Newark and implemented on August 12, 1968, as follows:

TABLE G-104

EQUIVALENT PASSENGER-VEHICLE APPROACH VOLUME

APPROACH	PASSENGER VEHICLES: MORNING PEAK 7:30-8:30 AM	PASSENGER VEHICLES: EVENING PEAK 4:30-5:30 PM
Central Ave. EB	1133	691
Central Ave. WB	376	1012
High St. NB	294	635
High St. SB	334	357

CENTRAL AVE. AND HIGH ST. SIGNAL TIMING (SEC)

PHASE	DIAL 1 LEADING GREEN	DIAL 1 GREEN	DIAL 1 AMBER	DIAL 1 RED	DIAL 2 LEADING GREEN	DIAL 2 GREEN	DIAL 2 AMBER	DIAL 2 RED
Central Ave.	0.0	39.6	4.5	45.9	0.0	39.6	4.5	45.9
High St. NB	1.8	39.6	4.5	44.1	10.8	30.6	4.5	44.1
High St. SB	0.0	39.6	4.5	45.9	0.0	30.6	4.5	54.9

Surveillance System Design.—The variables to be measured for each approach are vehicles through per green interval, vehicles in queue at the end of each red interval, and arrival rates during the red interval to estimate vehicle delay time.

The "before" measurements were scheduled for 3:30 to 5:30 PM on July 31, and 7:30 to 8:30 AM and 3:30 to 5:30 PM on August 1. The "after" measurements were scheduled for 3:30 to 5:30 PM on August 14, 7:30 to 8:30 AM and 3:30 to 5:30 PM on August 15, and 3:30 to 5:30 PM on August 16. ATR counters were placed to measure approach volumes for 48-hr periods between July 31 and August 2, and again between August 14 and August 16, 1968.

Implementation of Improvement.—The physical changes made to the traffic signal include the separation of northbound and southbound green, amber, and red signal circuits, installation of a second dial, a dial transfer relay, and a timeclock.

The two-dial operation began on August 12, 1968, as scheduled.

Surveys.—On August 14, an excavation in the pavement was made for a sewer connection on the westbound Central Avenue approach; this created turbulence in the flow of traffic. The surveys were postponed until Friday, August 16, when the excavation was filled in and the surface was restored.

The remaining "after" surveys were made on August 16, August 21, and August 22, so as to maintain the same days of the week in both the "before" and "after." A preliminary analysis of the data revealed a large difference in the number of vehicles stopped during each of the days of the "before" study. Because of this variation, a second study of the "before" conditions was scheduled for Tuesday, Wednesday, and Thursday, September 17, 18, and 19, 1968. The signal control was returned to the original "before" settings for this special study.

First Level Analysis

The purpose of the First Level Analysis is to evaluate the significance of changes in local traffic flow caused by the improvement. This objective was accomplished through the statistical comparison of "before" and "after" measurements of certain quantities that were judged to be indicators of traffic flow improvement. The quantities of total vehicles through the intersection per cycle, total vehicles stopped on red per cycle, and the number of saturated cycles, were studied for each approach. The number of left turns per cycle from the northbound High Street approach was also investigated.

The analysis was performed on data for the evening peak hour of 4:30 to 5:30 PM recorded on September 18 and 19, 1968, for the "before" conditions, and August 21 and 22, 1968, for the "after" conditions.

The field sheets were summarized by cycle in preparation for the First Level Analysis. The number of vehicles stopped on red, the number of vehicles through, and the number of left turns per cycle, were recorded directly from survey forms. The number of saturated cycles was determined by counting the cycles when the number of vehicles queued at the beginning of the green interval was greater than the number of vehicles through during the green interval.

Statistical Analysis of Data.—The individual observations of vehicles stopped on red and vehicles through for each approach, and the number of left turns on the northbound High Street approach, were formated into a series of two factor tables of days, and "before" and "after" for each approach. Bartlett's test, performed on the data, indicated that the null hypothesis of equal variance was accepted for the number of vehicles through on all approaches, rejected for the number of vehicles stopped on red for all approaches, and accepted for the number of left turns on the northbound High Street approach. Transformation of the data for the number of vehicles stopped ($10\sqrt{x+1}$) and a subsequent Bartlett's test indicated that null hypothesis of equal variance was accepted for the Central Avenue approaches and rejected for the High Street approaches.

Analysis of Variance.—ANOVA was performed on all data that indicated equal variance. The results given in Table G-105 indicate a significant ($\alpha = 0.05$) reduction from the "before" to the "after" conditions for the number of vehicles through on the southbound High Street approach, the number of vehicles through on the eastbound Central Avenue approach, and the number of vehicles stopped on the eastbound Central Avenue approach. Table G-105 also gives the results of the analysis for the number of left turns on the northbound High Street approach.

Alternate Analysis.—Because the data tables for vehicles stopped on red on the northbound and southbound High Street approaches rejected the null hypothesis of equal variance, an alternate method of analysis was used. The "before" and "after" conditions were compared for individual days. As a first step, the variances for each day were compared by an F ratio; then the t test or median test was performed. The results of this analysis, given in Table G-106, indicate a significant reduction in the number of vehicles stopped on the northbound High Street approach.

Chi Square Test.—The frequency of the number of saturated cycles comparing the "before" and "after" conditions was analyzed by a chi square test, combining both days for each approach (Table G-107).

The test indicated a significant reduction in the number of saturated cycles from the "before" condition to the "after" condition for the northbound High Street approach and for the eastbound Central Avenue approach.

Figures G-131 through G-134 compare the "before" and "after" conditions at each approach during an evening peak hour. The figures are plots of the vehicles through each cycle and the vehicles queued each cycle, which show the number of vehicles not clearing in one cycle. There is very little difference between "before" and "after" conditions at any approach except High Street northbound, where the improvement due to the advance green interval is evidenced by the decrease in vehicles not clearing a cycle.

Summary and Conclusions

The introduction of an advance, or leading green interval, did produce significant improvement at the $\alpha = 0.05$ level. There was a substantial decrease in the mean number of

TABLE G-105

ANALYSIS OF VARIANCE, CENTRAL AVENUE AND HIGH STREET

APPROACH	VARIABLE	DATA	NO. OF OBS.	COEF. OF VARIATION	STANDARD DEVIATION	MEAN "BEFORE"	MEAN "AFTER"	MEAN DIFFERENCE	CHANGE (%)	SIG. @ $\alpha=0.05$
High St. NB	Vehicles through	Original	157	19.48	2.68	13.98	13.51	−0.47	—	No
	Number of left turns	Original	158	53.50	1.85	3.63	3.30	−0.33	—	No
High St. SB	Vehicles through	Original	159	33.52	3.31	10.69	9.05	−1.64	−15.3	Yes
Central Ave. EB	Vehicles through	Original	159	27.30	4.11	16.05	14.02	−2.03	−12.6	Yes
	Vehicles stopped	Transformed	159	21.20	7.49	13.30	9.78	−3.50	−26.5	Yes
Central Ave. WB	Vehicles through	Original	160	20.24	4.99	25.05	24.21	−0.84	—	No
	Vehicles stopped	Transformed	160	23.49	11.12	22.47	20.38	−2.09	—	No

vehicles queued per cycle and the number of saturated cycles on the approach (High Street northbound) receiving the advance green, and there was no adverse effect measured at any of the other approaches.

Vehicles Through Each Cycle.—There was a significant decrease in the mean number of vehicles through each cycle for the opposing approach (High Street southbound) and for the Central Avenue eastbound approach. This experiment imposed no physical changes that would affect the traffic on the Central Avenue approaches, so the volume change on Central Avenue must be attributed to influence beyond the scope of this experiment.

Vehicles Stopped per Cycle.—Meaningful significant differences in the mean number of vehicles stopped per cycle were measured as a result of the advance green interval. High Street northbound, the approach receiving the advance green, showed a 62-percent reduction in the number of vehicles stopped after implementing the advance green. High Street southbound, the opposing approach, displayed no significant change as a result of the loss in green time.

This experiment did not revise the Central Avenue approaches, and the measurements indicate no substantial change in vehicles stopped on either of the Central Avenue approaches. Central Avenue eastbound did show a decrease in the number of vehicles stopped, but this was evidently due to a decrease in volume.

Number of Saturated Cycles.—A significant difference in the number of saturated cycles (cycles where the number of vehicles queued at the beginning of the green interval exceeds the number through on the green interval) occurred as a result of the advance green interval. High Street northbound showed a 70-percent decrease in the number of saturated cycles during the evening peak hour; the opposing approach, High Street southbound, displayed no significant change. Again, Central Avenue eastbound showed a significant decrease of the number of saturated cycles after the

TABLE G-106

ALTERNATE ANALYSIS, CENTRAL AVENUE AND HIGH STREETS, VEHICLES STOPPED ON RED INTERVAL

APPROACH	DAY	CONDITION	VARIANCE	ARE VARIANCES EQUAL	TEST	AVERAGE[a]	DIFFERENCE	SIG. @ $\alpha=0.05$
High St. NB	1	"Before"	118.96	No	Median	23.5	−14.5	Yes
		"After"	47.11			9.0		
	2	"Before"	35.12	Yes	t	28.60	−17.55	Yes
		"After"	40.97			11.05		
High St. SB	1	"Before"	9.32	No	Median	4.0	0.0	No
		"After"	27.13			4.0		
	2	"Before"	16.82	No	Median	4.0	0.0	No
		"After"	7.56			4.0		

[a] Median values listed where median test was used; mean values listed where t test was used.

TABLE G-107

CHI SQUARE ANALYSIS, CENTRAL AVENUE AND HIGH STREET, SATURATED CYCLES

APPROACH	CONDITION	NO. OF CYCLES SATURATED	NOT SATURATED	ALL	SIG. DIFFERENCE @ $\alpha=0.05$
High St. NB	"Before"	66	14	80	Yes
	"After"	20	58	78	
	All	86	72	158	
High St. SB	"Before"	2	77	79	No
	"After"	7	73	80	
	All	9	150	159	
Central Ave. WB	"Before"	26	54	80	No
	"After"	19	61	80	
	All	45	115	160	
Central Ave. EB	"Before"	21	59	80	Yes
	"After"	5	74	79	
	All	26	133	159	

green interval was implemented; however, this difference cannot be directly attributed to this experiment.

General Comments.—Although this experiment does result in a more efficient use of the available green time at this intersection, the conclusions (Table G-108) are somewhat clouded due to the traffic variations on the minor approaches. Figures G-131 through G-134 show the marked improvement on the High Street northbound approach with only small changes on the other approaches.

The data summarized for this experiment pointed out the lack of sufficient green time for both critical approaches (High Street northbound and Central Avenue westbound). A further improvement will be implemented to measure the effect of parking restrictions and signal retiming at this intersection (Experiment A68).

Convenience and Safety.—Results of the accident studies showed that the intersection of Central Avenue and High Street had ten reported accidents in 1966 and two in 1967. It is probable that congestion of the northbound High Street approach and driver irritation because of extensive delays were contributing factors to this situation. If this is so, the reduction of delay should have a beneficial effect on the safety of operation at this intersection.

The decrease in total vehicles queued each cycle and in

TABLE G-108

SUMMARY OF ANALYSIS

VARIABLE	APPROACH	MEAN VALUE "BEFORE"	"AFTER"	DIFFERENCE (%)	METHOD OF ANALYSIS	SIG. DIFFERENCE @ $\alpha=0.05$
Vehicles through each cycle	Central Ave. WB	25.05	24.21	—	ANOVA	No
	Central Ave. EB	16.05	14.02	−12.6	ANOVA	Yes
	High St. SB	10.69	9.05	−15.3	ANOVA	Yes
	High St. NB	13.98	13.51	—	ANOVA	No
	High St. N-left	3.63	3.30	—	ANOVA	No
Vehicles queued per cycle	Central Ave. WB	22.5	20.4	—	ANOVA [a]	No
	Central Ave. EB	13.3	9.8	−26.5	ANOVA [a]	Yes
	High St. SB	4.0	4.0	—	Median	No
	High St. NB	26.5	10.0	−62.3	Median	Yes
Number of saturated cycles	Central Ave. WB	26	19	—	Chi square	No
	Central Ave. EB	21	5	−76	Chi square	Yes
	High St. SB	2	7	—	Chi square	No
	High St. NB	66	20	−70	Chi square	Yes

[a] Data were transformed ($10\sqrt{x+1}$).

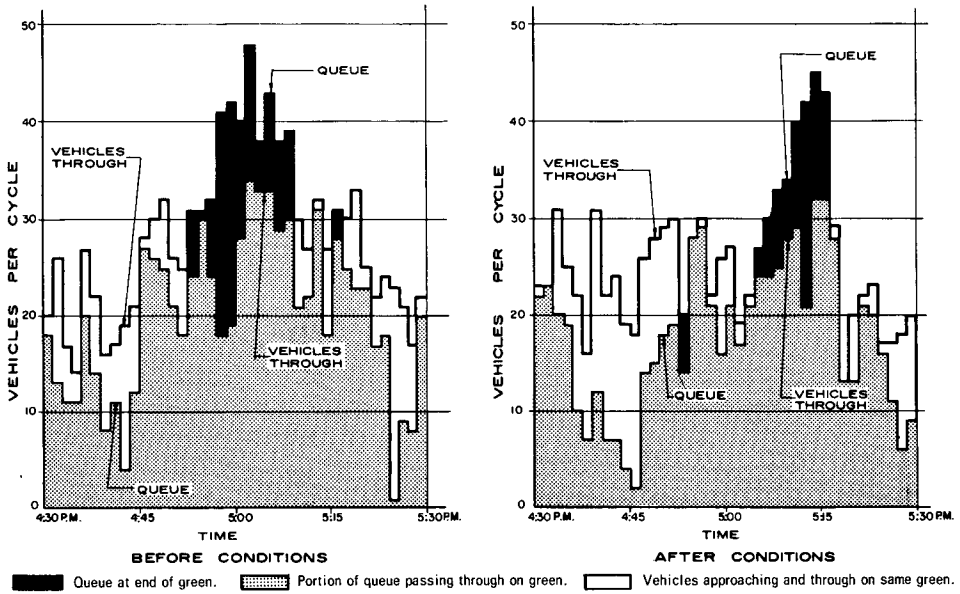


Figure G-131. Vehicles through and queue length per cycle, Central Avenue westbound.

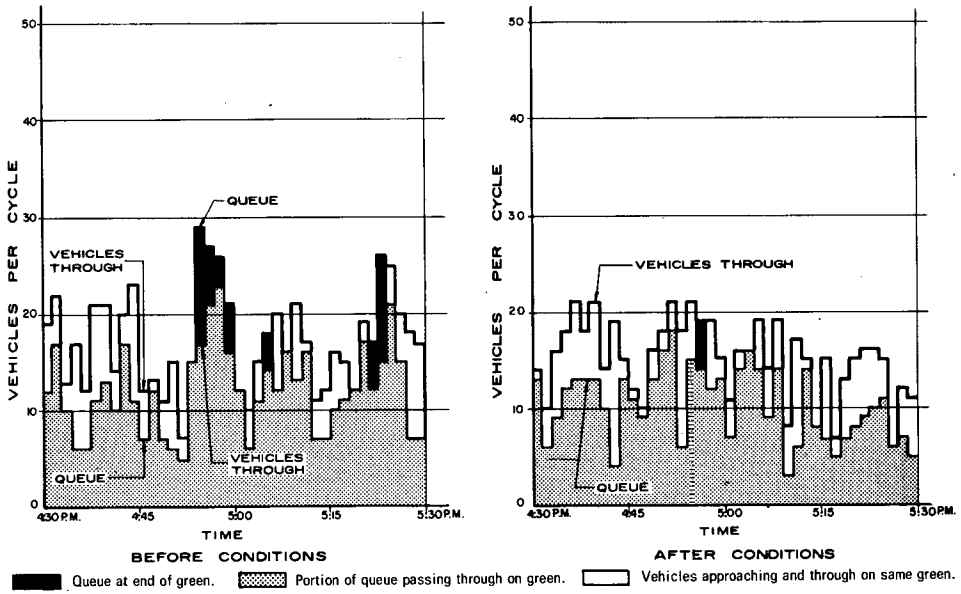


Figure G-132. Vehicles through and queue length per cycle, Central Avenue eastbound.

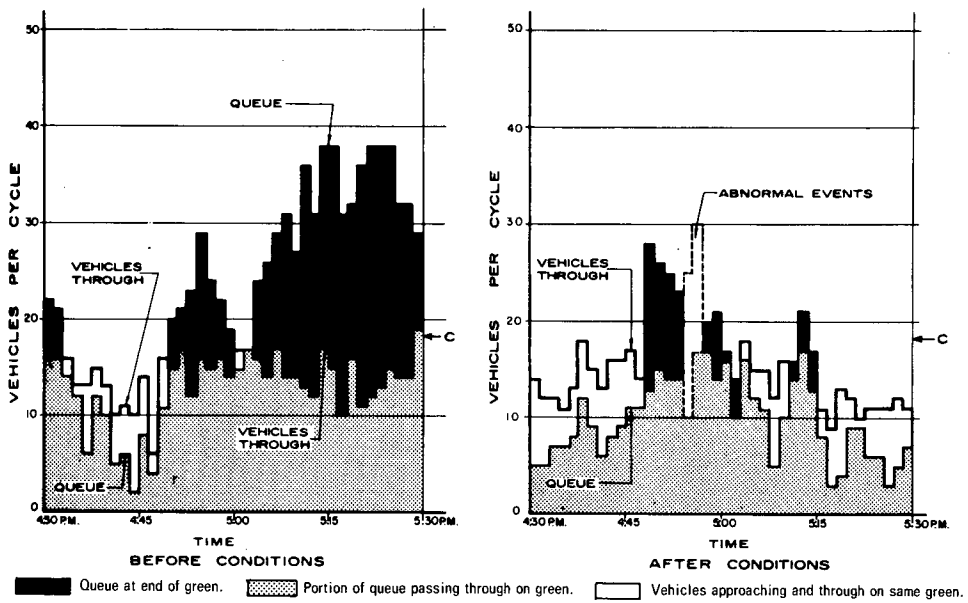


Figure G-133. Vehicles through and queue length per cycle, High Street northbound.

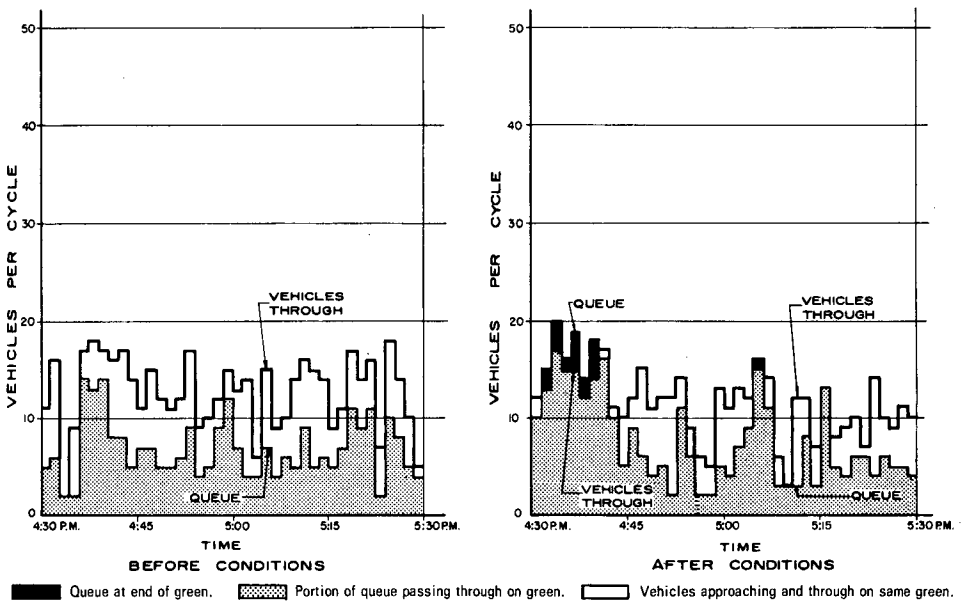


Figure G-134. Vehicles through and queue length per cycle, High Street southbound.

the number of saturated cycles resulting from the advance green is important when considering the drivers' convenience. The number of vehicles queued per cycle on High Street northbound decreased by 62 percent and the number of saturated cycles decreased by 70 percent.

Springfield Avenue Signal Progression—Experiment B88

Experiment B88 was designed to evaluate the effect of revising the offset relationships between adjacent traffic-signal-controlled intersections along an urban arterial. Concurrently, this implementation was intended to test the capabilities of the existing interconnected electronic traffic signal system, because the system had been operated on a 90-sec cycle and a simultaneous offset plan since its installation in 1959.

The revision of offset relationships was accomplished by first repositioning the dial controls on the local controllers and then visually calibrating the resulting offset operation, using a stopwatch. This calibration was necessary owing to the age and condition of the equipment and the inherent limited accuracy of the electronic timing circuitry.

The supervisory capabilities of the system's master controller were modified to ensure that the offset in operation was in accordance with the experimental design plan. This change was accomplished through the use of automatic time switches electrically connected to the master controller.

Experimental Area

Experiment B88 was conducted on Springfield Avenue from Hayes Street on the west to University Avenue on the east, and along High Street from William Street on the south to West Market Street on the north. The location of the experimental area is shown in Figure G-135. Springfield Avenue performs functionally as one of the primary arterial routes into Newark from the west, as well as being an extension of N.J. 24, and is used by commuters from many adjacent suburbs. Between 16,000 and 20,000 vehicles use this facility daily in the experimental area, with peak-hour, peak-direction volumes being about 1,100 vehicles.

Although the experiment was involved primarily with Springfield Avenue, the area along High Street was included in the experiment because the signal-controlled intersections within the limits described are a part of the interconnected signal system. Revision to the offset relationship of the common Springfield Avenue-High Street signals affects the offset relationships of the adjacent signals.

The Springfield Avenue area was selected for this experiment because its location external to the interconnected grid signal system permitted revisions to offset relationships without affecting other arteries, except for the previously mentioned High Street signals.

Within the experimental area, Springfield Avenue land use consists primarily of small retail stores, portions of which were destroyed in the 1967 riots. A deteriorating street surface of Belgian block, partially overlaid with asphalt, and lax enforcement of parking prohibitions limit the use of this facility to its maximum potential.

From Hayes Street to South Orange Avenue, the width of Springfield Avenue is 40 ft between curbs. No-stopping-or-standing regulations are posted for the 7:00 to 9:00 AM period on the south side and the 4:00 to 6:00 PM period on the north side. In addition, no-parking regulations are posted for the 4:00 to 6:00 PM period on the south side and the 7:00 to 9:00 AM period on the north side. These regulations are intended to provide for two moving lanes of traffic in each direction during periods of peak traffic flow. As a practical matter, frequent violations of the no-stopping-or-standing regulations condition the average repeating motorist to avoid use of the curb lane. The fact that there was a strip of exposed block and several unrepaired holes in the curb lanes, together with the use of this lane for bus passenger service, also contributed to its limited use by through traffic. Bus stops are not signed. Passenger service operations are commonly performed in the driving lane, particularly during the midday period, when parking is legal at both the north and south curbs. Crosswalk lines, stop bars, and the white center line have not been renewed for more than a year, resulting in faded or nonexistent markings for most of the route.

From South Orange Avenue to High Street, the clear width between curbs is 46.6 ft. Reversible lane signals and lane markings are applied to operate this section as five lanes during peak traffic periods, with three lanes in the direction of dominant flow. From High Street to Market Street, Springfield Avenue widens from 49 to 60 ft, providing for two lanes of moving traffic in each direction. On Market Street the clear width varies from 105 ft at Springfield Avenue to 96 ft at University Avenue, providing for three lanes of moving traffic in each direction during periods of peak traffic flow.

Restrictions to smooth flow are experienced at the William Street, 13th Avenue, and Market Street intersections with Springfield Avenue, as right-of-way is unassigned at these points of conflicting traffic movement. Left turns are prohibited from Springfield Avenue only at the High Street intersection. Left-turn movements occurring at the intersection of Springfield Avenue with Prince Street and at the intersection of Springfield Avenue with Jones Street and Belmont Avenue severely impede through traffic movement.

Experimental Design

Each local controller in the system is capable of providing at least four different offset plans under master control supervision. Similarly, the local controller provides for three separate split arrangements, each associated with a specific offset plan or plans. Prior to the experiment, unplanned changes in system operations due to equipment malfunctions had been minimized by setting the dials on each local controller identically for all offsets and splits. The supervisory functions of the master controller also were limited by the use of manual switches that overrode its automatic capabilities.

These conditions made possible the functional testing of the entire system as a part of this experiment, without affecting the established operational timing outside the experimental area. The existing 90-sec cycle length was not changed for this experiment, as only the effects of revising offset relationships were to be evaluated.

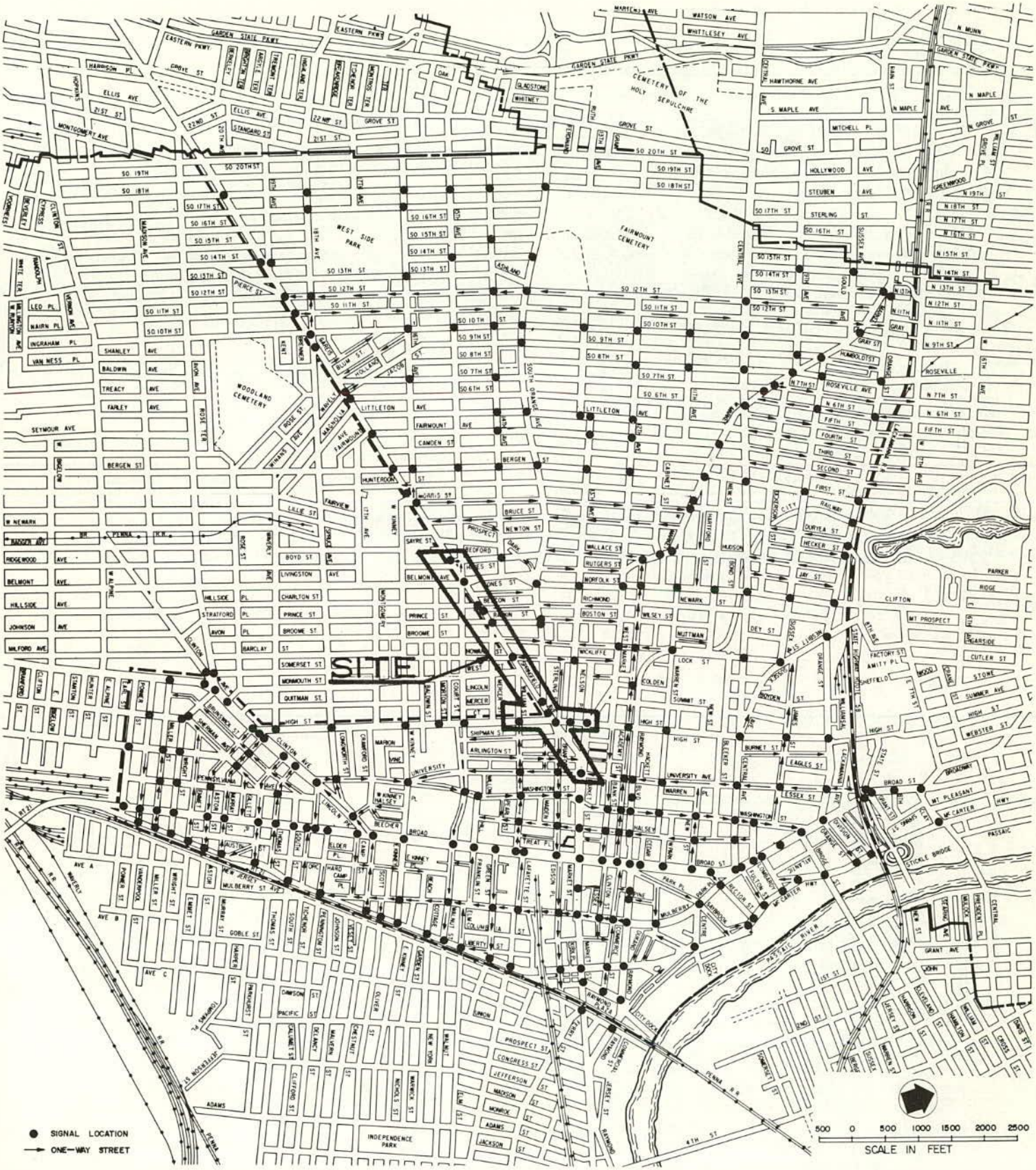


Figure G-135. Location map, Experiment B88.

The "before" offset and split relationships for Springfield Avenue are shown in Figure G-136.

The "before" offset and split relationships on High Street are shown in Figure G-137. In order that the High Street offset relationships would not change and introduce additional variables into the effects of the designed changes, they were adjusted to remain identical for all stages of the improvement.

Stage 1 of the experiment provided three offset plans. A 25-mph speed of progression was used for both east and westbound traffic flows in off-peak periods (average offset), as shown in Figure G-138. A 25-mph preferential progression also was used for eastbound traffic flow from 7:00 AM until 9:00 AM (inbound offset), as shown in Figure G-139, and for westbound traffic flow from 4:00 PM until 6:00 PM (outbound offset), as shown in Figure G-140. Split relationships were revised at the Charlton Street-Rankin Street intersection for all three offset plans and at the South Orange Avenue intersection for the inbound and outbound offset plans, to ensure adequate "green" time for through traffic.

Stage 2 of the experiment provided for average inbound and outbound progressive traffic movements, with the speed of progression changed to 30 mph. The offset relationships and splits used for this stage of the experiment are shown in Figures G-141, G-142, and G-143. Split relationships were revised from the Stage 1 conditions at the Hayes Street and Belmont Avenue-Jones Street intersections for all three offset plans and also at the Prince Street intersection for the outbound offset plan, to expedite through traffic flows.

In both stages the offset and split arrangements of the Market Street-University Avenue signal were maintained as in the "before" condition to serve as a base reference for all timing revisions.

Stage 1 was implemented on March 11, 14, and 17, 1969. The outbound progression for Stage 2 was installed on April 23, 1969; the average and inbound progressions were implemented on May 8, 1969.

Evaluation of the effects of the revised offset plans was based on four areas of surveillance:

1. Automatic Traffic Recorder (ATR) counts.
2. Speed and delay runs.
3. Manual intersection counts.
4. Bus trip time between cordons.

The ATR counts were used to determine daily variations in approach volumes and to determine if traffic volumes were "normal" between March 11 and April 15, 1969. Two locations were selected for monitoring each direction—one just east of 13th Avenue and the other just east of Morris Avenue. Considerable difficulty was experienced in this operation owing to vehicles parking on the pneumatic hose, vandalism, and mechanical problems with the recorders. The mean values of all recorded data are given in Table G-109.

Speed and delay runs were made during all time periods, using a Marbelite Traffic Data Compiler. Only this method of surveillance was used to provide data for evaluation of Stage 2. The total number of runs analyzed for each condition is given in Table G-110.

Manual intersection counts were made of vehicles through and stopped per cycle for three days "before" and three days "after" the implementation of Stage 1 at the intersections of South Orange Avenue and Prince Street with Springfield Avenue. Both directions of flow were surveyed for the midday time period, with only the predominant flows surveyed in the AM and PM peak hours (eastbound in the AM,

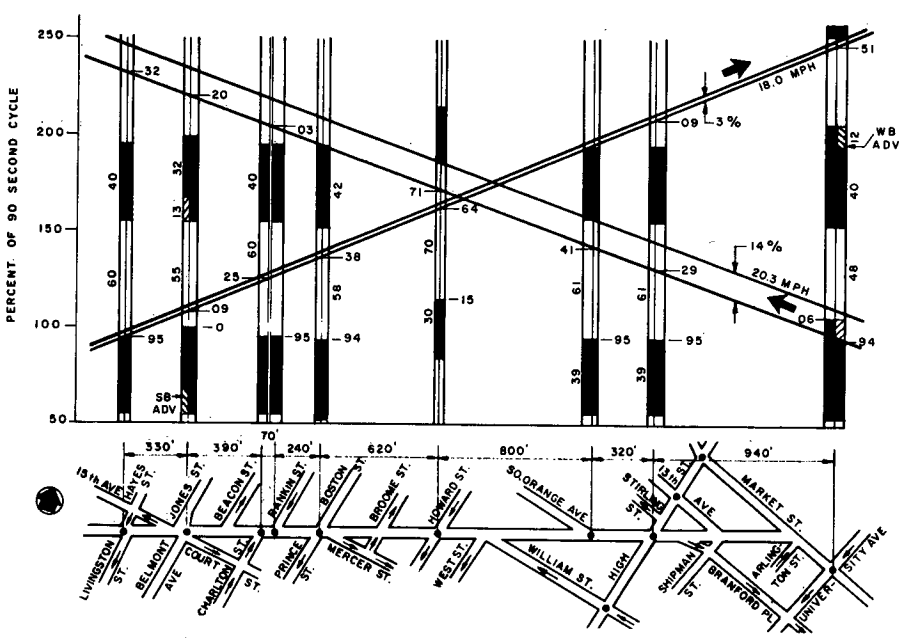


Figure G-136. Offset plan, "before."

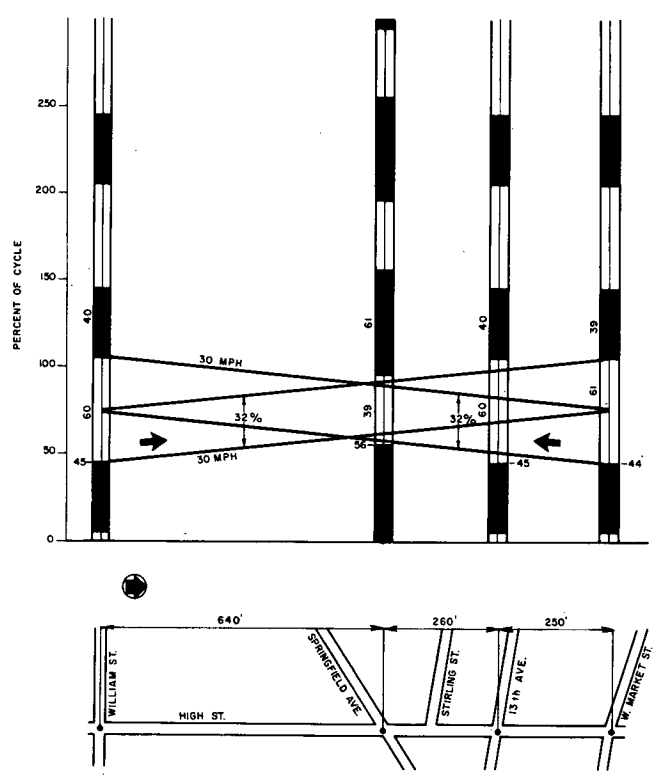


Figure G-137. High Street offset plan, "before."

westbound in the PM). These surveys were made on February 26, 27, and 28, 1969, for the "before" condition, and on March 26, 27, and 28, and April 9 and 11, 1969, for the "after" Stage 1 condition.

A special study of bus trip time was performed by Public Service Coordinated Transport Company to provide a means of evaluating the effects of the offset plan revisions on local transit service.

Trip times were measured during peak periods for buses traversing Springfield Avenue between the Pennsylvania Railroad Station and Bergen Street on September 19, 1968, for the "before" condition and on March 27, 1969, for the Stage 1 "after" condition. The bus company's "checkpoints" were located 1,700 ft west of and 2,000 ft east of the experimental area.

One-day bus trip time studies were conducted "before" and "after" Stage 1 during the AM time period to provide data for comparison with the bus company's survey findings. The University Avenue intersection with Market Street and the Morris Avenue intersection with Springfield Avenue were used as cordon lines in this latter study. These limits exclude most of the roadway outside the experimental area included in the bus company's survey. The Morris Avenue intersection lies 1,500 ft west of the Hayes Street intersection on Springfield Avenue. These data were collected on March 11 and April 8, 1969.

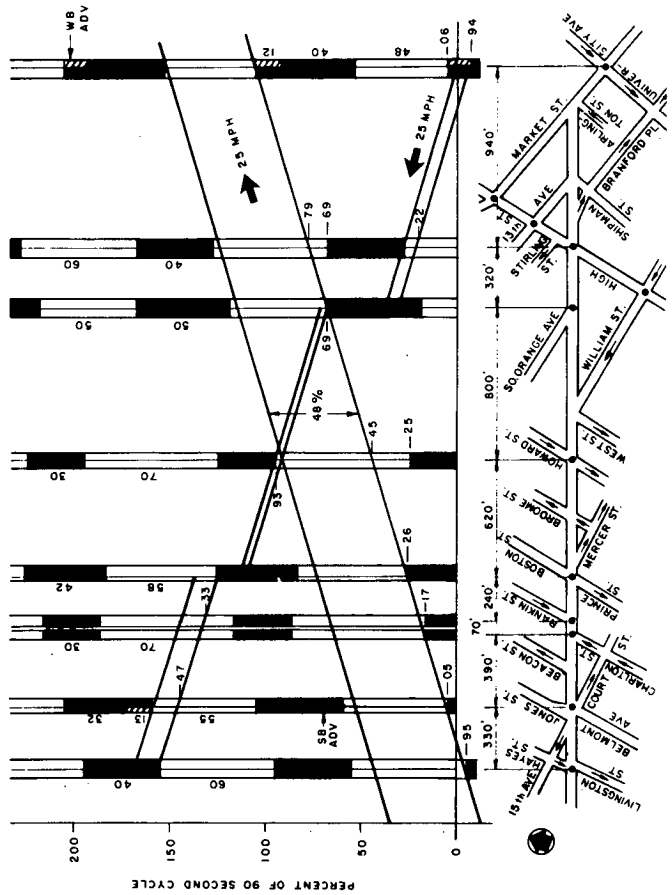


Figure G-139. Inbound offset plan, Stage 1.

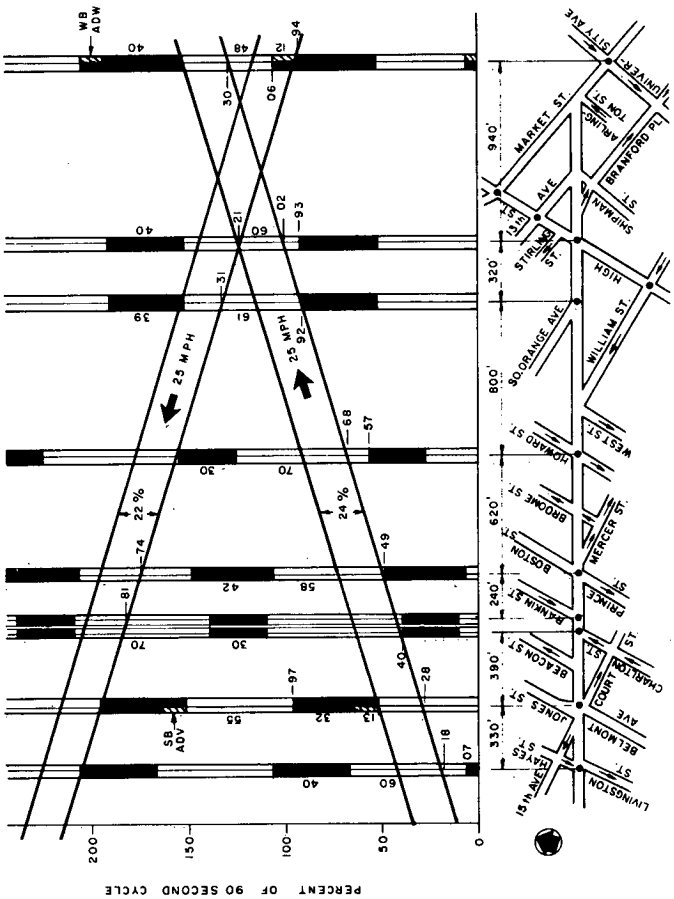


Figure G-138. Average offset plan, Stage 1.

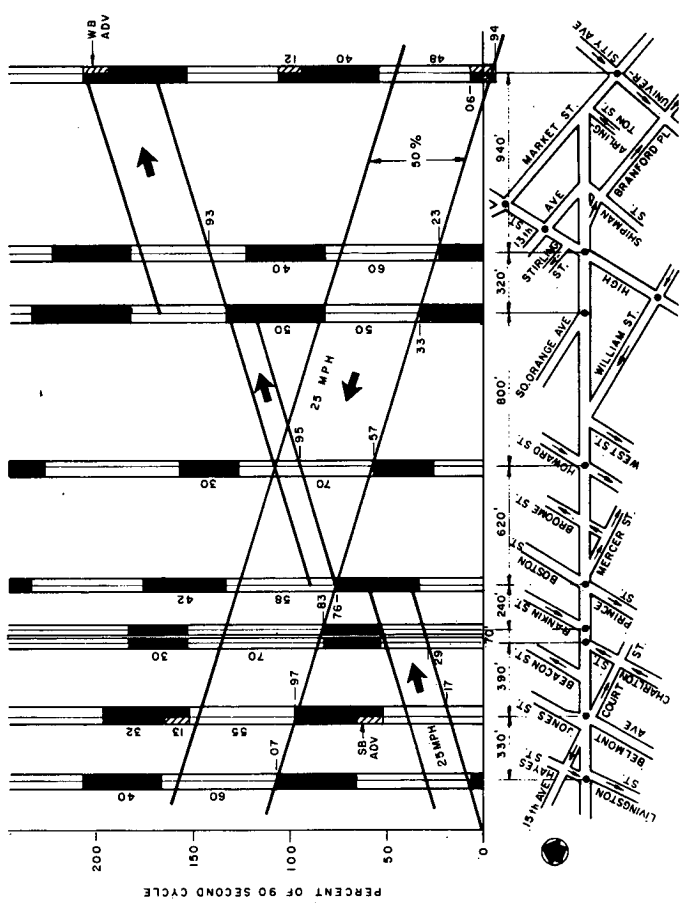


Figure G-140. Outbound offset plan, Stage 1.

Analysis

Analysis of ATR data taken "before" and "after" Stage 1 showed that no significant day-of-week variations existed. It also indicated that the speed and delay runs and the manual surveys were made on "normal" days.

Speed and delay runs were made between Hayes Street and University Avenue during the AM and PM time periods in both directions. The variables analyzed from speed and delay data included:

1. Travel time.
2. Speed.
3. Delay time.
4. Number of stops.
5. Delay time per stop.

Tables G-111 and G-112 summarize the analysis and give the averages for each of the foregoing variables for all speed and delay runs. As indicated, implementation of Stage 1 significantly improved the eastbound (peak direction) traffic flow in the AM peak hour for all variables analyzed and adversely affected the minor westbound traffic flow. During the PM peak hour all variables except delay time per stop and speed indicated a significant improvement in the predominant westbound movement. The minor eastbound flow

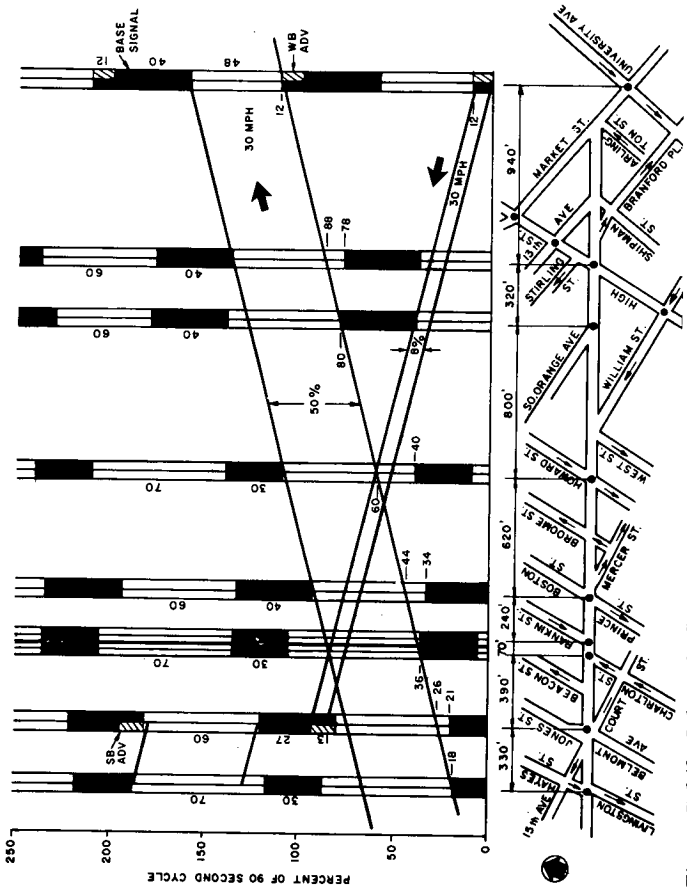


Figure G-142. Inbound offset plan, Stage 2.

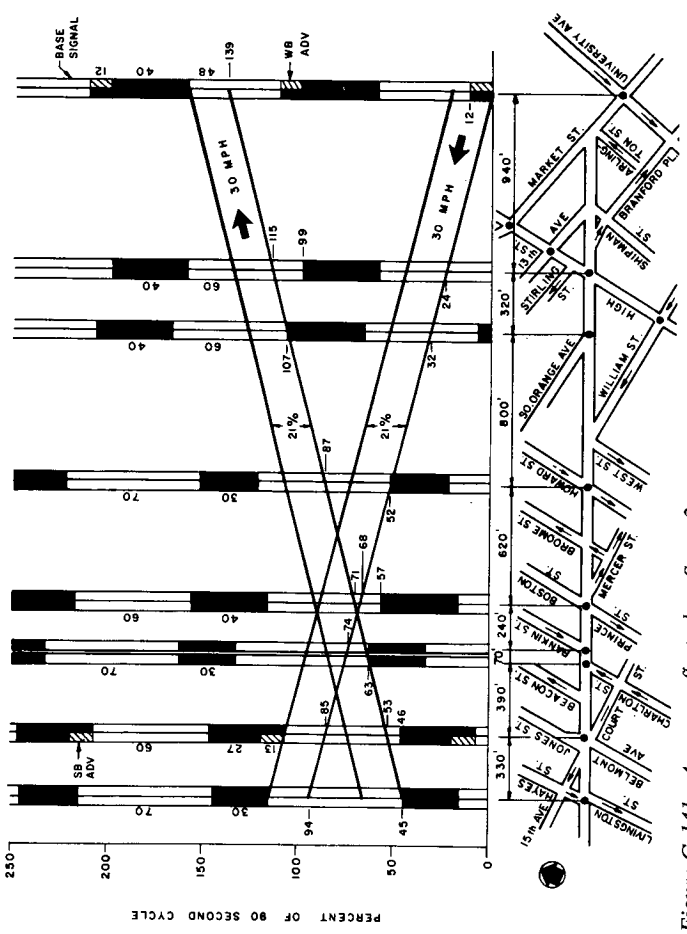


Figure G-141. Average offset plan, Stage 2.

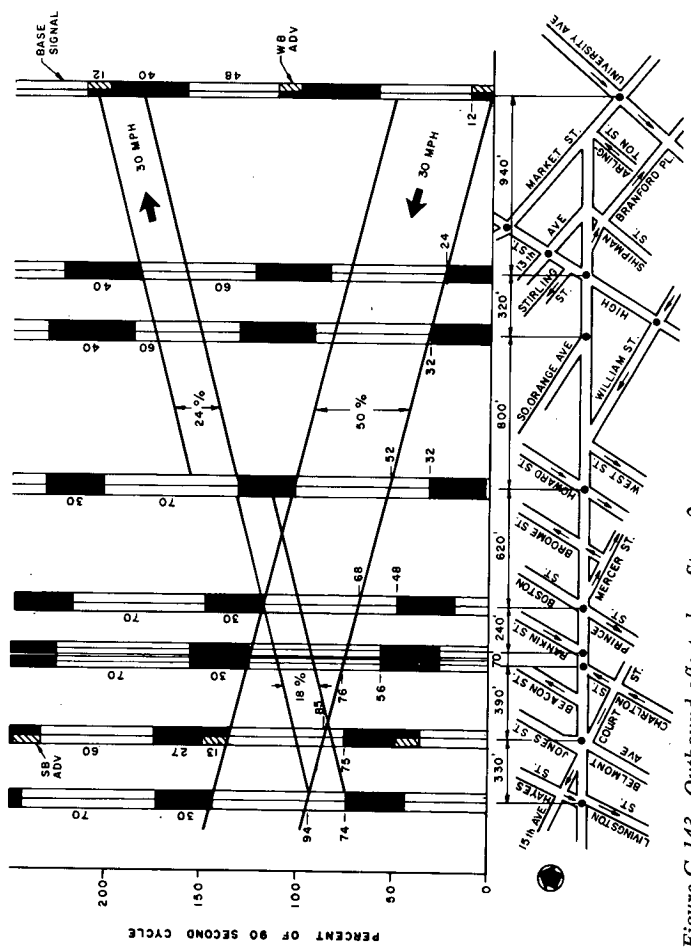


Figure G-143. Outbound offset plan, Stage 2.

was not adversely affected, and delay time per stop was significantly reduced for this direction of traffic.

Implementation of Stage 2 significantly improved speed during the AM period for the eastbound traffic flow. As given in Table G-112, additional analysis indicated that the adverse effect of the Stage 1 improvement on the minor westbound traffic flow was eliminated. Implementation of Stage 2 also made no significant changes in the improvements realized with Stage 1 for the predominant westbound traffic flow in the PM peak hour. Again, the minor eastbound traffic flow was not adversely affected.

Data gathered during the manual intersection counts

TABLE G-109

ATR MEASURED TRAFFIC VOLUMES, SPRINGFIELD AVENUE

TIME PERIOD	AT MORRIS ST. EB DAYS	AT MORRIS ST. EB VOLUME	AT MORRIS ST. WB DAYS	AT MORRIS ST. WB VOLUME	AT 13TH AVE. EB DAYS	AT 13TH AVE. EB VOLUME	AT 13TH AVE. WB DAYS	AT 13TH AVE. WB VOLUME
24-hr	20	8,302	15	8,186	16	10,562	17	10,305
AM	24	947	22	299	25	1,166	23	511
PM	25	355	24	770	28	447	23	1,082

TABLE G-110

NUMBER OF SPEED AND DELAY RUNS, SPRINGFIELD AVENUE

CONDITION	DATES (1969)	SPEED AND DELAY RUNS EB AM	EB MIDDAY	EB PM	WB AM	WB MIDDAY	WB PM
"Before"	2/25–3/11	27	20	20	28	18	24
"After" (Stage 1)	3/26–4/11	16	13	13	19	11	13
"After" (Stage 2)	4/23–5/13	32	—	15	34	—	16

TABLE G-111

SPEED AND DELAY ANALYSIS, EASTBOUND SPRINGFIELD AVENUE

CONDITION	TRAVEL TIME (SEC)	SPEED (MPH)	DELAY TIME (SEC)	NO. OF STOPS	DELAY TIME/STOP (SEC)
(*a*) AM time period					
"Before"	231.4	11.13	86.9	2.9	31.0
"After" (Stage 1)	138.6	19.13	18.8	0.7	13.2
Percent change	−40.1	+71.9	−78.4	−75.9	−57.4
Sig. @ $\alpha=0.05$	Yes	Yes	Yes	Yes	Yes
"After" (Stage 2)	159.4	16.58	37.8	1.6	22.8
Percent change (Stage 1 to Stage 2)	+15.0	−13.3	+101.1	+128.6	+72.7
Sig. @ $\alpha=0.05$	No	No	No	No	Yes
(*b*) PM time period					
"Before"	233.0	11.17	82.9	2.7	31.8
"After" (Stage 1)	214.9	12.29	71.4	3.5	19.2
Percent change	−7.8	+10.0	−13.9	+29.6	−39.6
Sig. @ $\alpha=0.05$	No	No	No	No	Yes
"After" (Stage 2)	248.1	10.43	99.1	3.9	25.5
Percent change (Stage 1 to Stage 2)	+15.4	−15.1	+38.8	+11.4	+32.8
Sig. @ $\alpha=0.05$	No	No	No	No	No

TABLE G-112

SPEED AND DELAY ANALYSIS, WESTBOUND SPRINGFIELD AVENUE

CONDITION	TRAVEL TIME (SEC)	SPEED (MPH)	DELAY TIME (SEC)	NO.OF STOPS	DELAY TIME/STOP (SEC)
(*a*) AM time period					
"Before"	160.6	16.43	39.3	1.6	22.5
"After" (Stage 1)	211.3	12.11	81.1	3.3	25.0
Percent change	+31.6	−26.3	+106.4	+106.3	+11.1
Sig. @ $\alpha=0.05$	Yes	Yes	Yes	Yes	No
"After" (Stage 2)	171.5	15.29	50.1	2.4	21.0
Percent change (Stage 1 to Stage 2)	−18.8	+26.3	−38.2	−27.3	−16.0
Sig. @ $\alpha=0.05$	No	Yes	No	No	No
Percent change ("before" to Stage 2)	+6.8	−6.9	+27.5	+50.0	+6.7
Sig. @ $\alpha=0.05$	No	No	No	No	No
(*b*) PM time period					
"Before"	281.3	9.77	110.5	4.1	27.2
"After" (Stage 1)	206.5	12.72	47.0	1.9	26.3
Percent change	−26.6	+30.2	−57.5	−53.7	−3.3
Sig. @ $\alpha=0.05$	Yes	No	Yes	Yes	No
"After" (Stage 2)	210.3	12.95	60.0	2.3	25.4
Percent change (Stage 1 to Stage 2)	+1.8	+1.8	+27.7	+21.1	−3.4
Sig. @ $\alpha=0.05$	No	No	No	No	No

TABLE G-113

SUMMARY OF INTERSECTION ANALYSIS FOR THE STAGE 1 IMPROVEMENT

CONDITION	PRINCE ST.			SOUTH ORANGE AVE.		
	VEHICLES STOPPED/ CYCLE	VEHICLES THROUGH/ CYCLE	STOP RATIO	VEHICLES STOPPED/ CYCLE	VEHICLES THROUGH/ CYCLE	STOP RATIO
(*a*) EB Springfield Ave.—AM						
"Before"	4.2	22.7	18.5	20.1	20.6	97.2
"After"	2.0	22.8	8.6	5.2	20.2	25.0
Percent change	−52.4	+0.4	−53.5	−74.1	−1.9	−74.3
Sig. @ $\alpha=0.05$	Yes	No	Yes	Yes	No	Yes
(*b*) EB Springfield Ave.—Midday						
"Before"	3.2	10.6	29.3	7.6	8.5	85.9
"After"	2.3	10.9	20.2	1.7	8.9	18.8
Percent change	−28.1	+2.8	−31.1	−77.6	+4.7	−78.1
Sig. @ $\alpha=0.05$	Yes	No	Yes	Yes	No	Yes
(*c*) WB Springfield Ave.—Midday						
"Before"	8.8	10.9	78.8	2.7	11.3	23.1
"After"	5.3	10.4	47.5	1.4	11.5	13.3
Percent change	−39.8	−4.6	−39.7	−48.1	+1.8	−42.4
Sig. @ $\alpha=0.05$	Yes	No	Yes	Yes	No	Yes
(*d*) WB Springfield Ave.—PM						
"Before"	16.4	20.1	82.1	4.0	21.5	18.6
"After"	5.0	19.0	24.6	3.2	21.9	14.4
Percent change	−69.5	−5.5	−70.0	−20.0	+1.9	−22.6
Sig. @ $\alpha=0.05$	Yes	No	Yes	Yes	No	Yes

TABLE G-114

BUS TRIP TIME ON SPRINGFIELD AVENUE (PENNSYLVANIA RR STATION TO BERGEN STREET)

CONDITION	LOCAL BUSES NO. OF OBS.	LOCAL BUSES AVERAGE TRIP TIME (MIN)	LOCAL BUSES RANGE (MIN)	EXPRESS BUSES NO. OF OBS.	EXPRESS BUSES AVERAGE TRIP TIME (MIN)	EXPRESS BUSES RANGE (MIN)
(*a*) EB—AM						
"Before"	20	14.3	11–22	9	12.8	11–14
"After" (Stage 1)	20	13.6	10–20	9	12.0	9–15
(*b*) WB—PM						
"Before"	18	15.2	10–19	10	14.2	12–16
"After" (Stage 1)	19	14.2	11–19	9	12.8	11–16

TABLE G-115

BUS TRIP TIME ON SPRINGFIELD AVENUE BETWEEN MORRIS AVENUE AND UNIVERSITY AVENUE (EASTBOUND—A.M.)

CONDITION	LOCAL BUSES NO. OF OBS.	LOCAL BUSES AVERAGE TRIP TIME (MIN)	LOCAL BUSES VARIANCE	EXPRESS BUSES NO. OF OBS.	EXPRESS BUSES AVERAGE TRIP TIME (MIN)	EXPRESS BUSES VARIANCE	ALL BUSES NO. OF OBS.	ALL BUSES AVERAGE TRIP TIME (MIN)	ALL BUSES VARIANCE
"Before"	14	6.82	4.11	6	6.62	2.96	20	6.76	3.60
"After" (Stage 1)	17	4.88	1.00	4	4.54	0.63	21	4.82	0.91
Percent change	—	−28.4	—	—	−31.4	—	—	−28.7	—
Significant difference	—	Yes	Yes	—	Yes	Yes	—	Yes	Yes
α level of sig.	—	0.005	0.005	—	0.025	0.10	—	0.001	0.005

were summarized to represent the number of vehicles through per cycle, the number of vehicles stopped per cycle, and the ratio of the number of vehicles stopped per cycle to vehicles through per cycle (stop ratio), and were analyzed by use of the ANOVA technique. Table G-113 summarizes the results of the ANOVA.

In all cases, reductions in the number of vehicles stopped per cycle and the stop ratio were significant, comparing the "before" conditions and "after" conditions of Stage 1, whereas the number of vehicles through per cycle did not change. Because intersection counts were not made after the implementation of the Stage 2 improvements, a further comparison of intersection data could not be made.

The bus company's trip time data were analyzed to determine the effect of the Stage 1 improvements on local transit service. A summary of these data is given in Table G-114. Because 50 percent of the distance in this study was outside the experimental area and trip times were recorded only to the nearest minute, only very large differences could be detected as being significant. However, it is noted that all trip times were reduced in the "after" surveillance. This trend is verified by the statistical analysis of the one-day "before" and "after" (Stage 1) study performed by project personnel, as summarized in Table

TABLE G-116

STOP RATIO COMPARISON FOR STAGE 1

CONDITION	PRINCE ST. SPEED AND DELAY METHOD	PRINCE ST. MANUAL SURVEY METHOD	SOUTH ORANGE AVE. SPEED AND DELAY METHOD	SOUTH ORANGE AVE. MANUAL SURVEY METHOD
(*a*) EB Springfield Ave.—AM				
"Before"	6.9	18.5	106.9	97.2
"After"	0.0	8.6	25.0	25.0
(*b*) EB Springfield Ave.—Midday [a]				
"Before"	23.8	29.3	104.8	85.9
"After"	23.1	20.2	7.7	18.8
(*c*) WB Springfield Ave.—Midday [a]				
"Before"	89.5	78.8	26.3	23.1
"After"	30.0	47.5	0.0	13.3
(*d*) WB Springfield Ave.—PM				
"Before"	88.5	82.1	38.5	18.6
"After"	33.3	24.6	11.1	14.4

[a] A number of speed and delay runs made during the midday time period and not used in the analysis of the experiment were used for this stop ratio comparison.

TABLE G-117

COMPARISON OF VEHICLE HOURS OF TRAVEL

TIME[a]	DIRECTION	PEAK-HOUR VOLUME	VEHICLE-HOURS "BEFORE"	"AFTER" STAGE 1	"AFTER" STAGE 2
AM	EB	1057	67.97	40.69	46.83
	WB	405	18.06	23.77	19.28
	Both	1462	86.03	64.46	66.11
PM	EB	401	25.94	23.94	27.63
	WB	926	72.32	53.15	54.08
	Both	1327	98.26	77.09	81.71

[a] AM peak hour (7:30-8:30 AM); PM peak hour (4:30-5:30 PM).

TABLE G-118

ORIGINAL SIGNAL TIMING

LOCATION	HOURS IN OPERATION	OFFSET (%)	PHASE A, CENTRAL AVE. (%)	PHASE B, CROSS-STREET (%)	CYCLE (SEC)
South 12th St.	24	0	70	30	90
South 10th St.	24	0	70	30	90
South Eighth St.	24	0	71	29	90
West Market St.—Dial 1	9–7 AM	0	52	48	90
West Market St.—Dial 2	7–9 AM	0	70	30	90
First St.	24	50	61	39	90
Norfolk St.	24	50	60	40	90
Lock St.	24	50	69	31	90
High St.—Dial 1	6–4 PM	25	61	39 NB 37 SB	90
High St.—Dial 2	4–6 PM	25	49	51 NB 39 SB	90

G-115. Field measurements in this study were made to the nearest second. It can be seen that total trip times for all buses were significantly reduced by almost 2 min (28.7 perment) in the AM time period. In addition, the variability in trip times was significantly reduced in the "after" condition.

Comparison of Survey Methods

To verify whether the speed and delay runs were truly representative of "average" traffic, a comparison was made of the stop ratio obtained by manual survey methods and the stop ratio experienced during speed and delay runs at the locations of the manual surveys. Although statistical tests were not performed to establish the level of significance of the difference, it is apparent in Table G-116 that the mean values obtained by the two methods are not greatly different and, also, that the direction of change is identical for both methods. It is not necessarily expected that the two sets of data would agree in magnitude, but they should agree in direction of change, because the speed-and-delay-run vehicle proceeded straight through the intersection in every case, whereas the intersection counts reflect all traffic, including turning vehicles.

Conclusions

Implementing a 25-mph speed of progression (Stage 1) resulted in greatly improved quality of flow without significantly changing traffic volumes. However, changing the speed of progression from 25 to 30 mph (Stage 2) made no statistically significant changes in quality of flow except for increasing the delay time per stop for the eastbound AM movement. Obviously, the increased speed of progression could not be matched by the motorists, probably owing to the previously mentioned physical limitations of the street. Comments from several motorists who use this facility daily indicate that the improvements were recognized and appreciated.

TABLE G-119

ADJUSTED SIGNAL TIMING

LOCATION	HOURS IN OPERATION	OFFSET BEGINNING OF GREEN (%) IN-BOUND	OUT-BOUND	PHASE A, CENTRAL AVE. (%)	PHASE B, CROSS-STREET (%)	CYCLE (SEC)
South 12th St.	24	35	38	70	30	90
South 10th St.	24	48	27	70	30	90
South Eighth St.	24	60	17	71	29	90
West Market St.—Dial 1	9–7 AM	—	25	52	48	90
West Market St.—Dial 2	7–9 AM	75	—	70	30	90
First St.	24	2	98	61	39	90
Norfolk St.	24	43	57	60	40	90
Lock St.	24	70	30	69	31	90
High St.—Dial 1	6–4 PM	0	—	61	39 NB 37 SB	90
High St.—Dial 2	4–6 PM	—	0	49	51 NB 39 SB	90

The extent of the improvement may be depicted in terms of "vehicle-hours" for the various conditions by multiplying the average total trip time produced by analysis of the speed and delay runs (Tables G-111 and G-112) by the average peak-hour volume produced from the ATR counts recorded at each end of the experimental area. This is indicated in Table G-117.

Central Avenue Signal Progression—Experiment B90

Experiment B90 examines the effects produced by the implementation of separate progression plans to favor the predominant flow of traffic during the morning and evening peak traffic periods in place of the single average progression plan in use at all times of the day.

The area of the experiment is on Central Avenue between High Street and the East Orange-Newark city line, a distance of approximately 7,900 ft (Fig. G-144). Central Avenue west of High Street is an Essex County highway and is a two-way major arterial street that extends westward from the center of Newark to the adjoining community of East Orange. West of West Market Street, Central Avenue has an ADWT of 31,000 vehicles; east of West Market Street the ADWT is 24,500 vehicles.

Traffic volumes for a typical day during the AM peak hour, a midday hour, and the PM peak hour are shown in Figure G-145.

Experimental Area

Central Avenue is four lanes wide (48 ft), marked with a center line and two lane lines from High Street to West Market Street. From West Market Street to the city line, Central Avenue is six lanes wide (60 ft), marked with a center line and four lane lines. These sections and the transition are shown in Figure G-146. Parking is prohibited on the south, or inbound, side from 7:00 to 9:00 AM and is permitted at all other times. On the north, or outbound, side parking is prohibited from 4:00 to 6:00 PM and is permitted at all other times.

Within the limits of the experiment are eight pretimed signals. The High Street signal is under the jurisdiction of the city of Newark, and the other seven are under the jurisdiction of the County of Essex. To the east of High Street is the city of Newark's traffic responsive system, to which no definite offset relationship exists. To the west, beyond the project boundary, is the county's interconnected system, to which there is no constant offset relationship.

Table G-118 gives the signal locations and major operational data. The signals are operated by pretimed controllers on a 90-sec cycle. The original progression is shown in Figure G-147.

Design of Experiment

Analysis of the existing "average" progression and signal timing disclosed that, whereas improvement could be made in individual signal timing, no major improvement could be made in the "average" progression.

Figure G-144. Location map, Experiment B90.

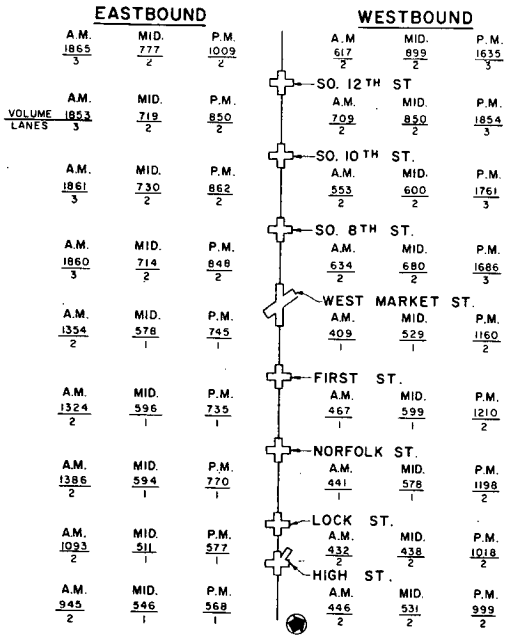


Figure G-145. Typical volumes.

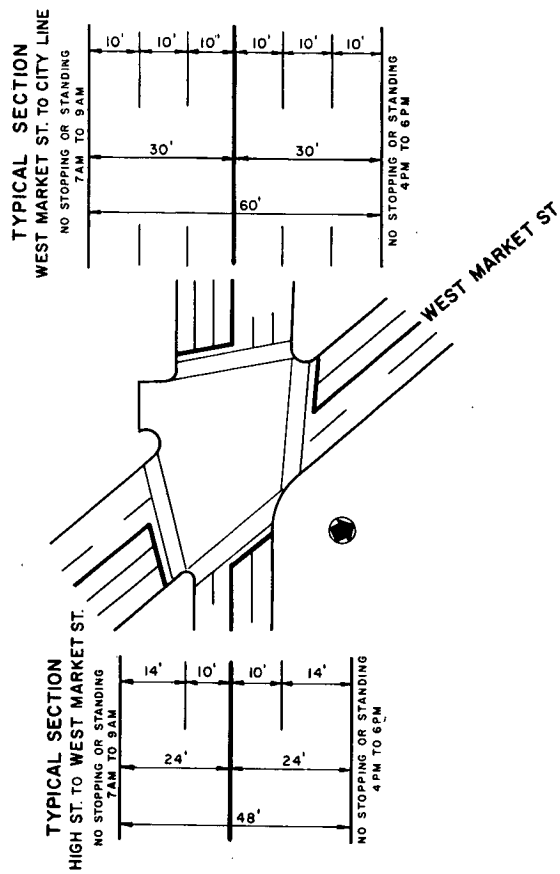


Figure G-146. Typical sections.

Two new progression plans, one for inbound and one for outbound traffic, were prepared to conform to the existing posted speed limits of 35 mph west of West Market Street and 30 mph east of West Market Street (Table G-119). No adjustments were made to the existing average progression plan. The two new progression plans are shown in Figures G-148 and G-149. The inbound plan was in effect from 7:00 to 9:00 AM, the outbound from 4:00 to 6:00 PM, and the average plan at all other times.

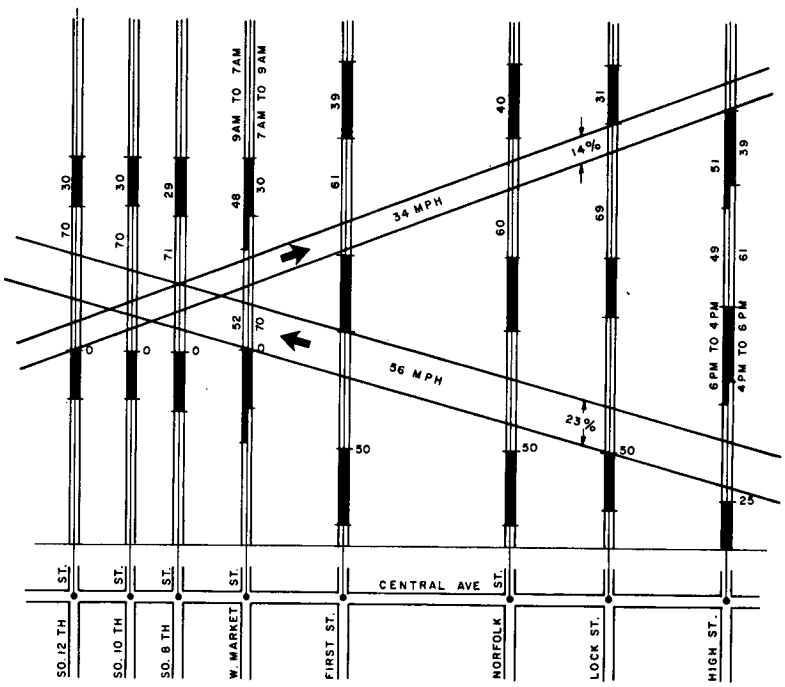


Figure G-147. "Before" progression.

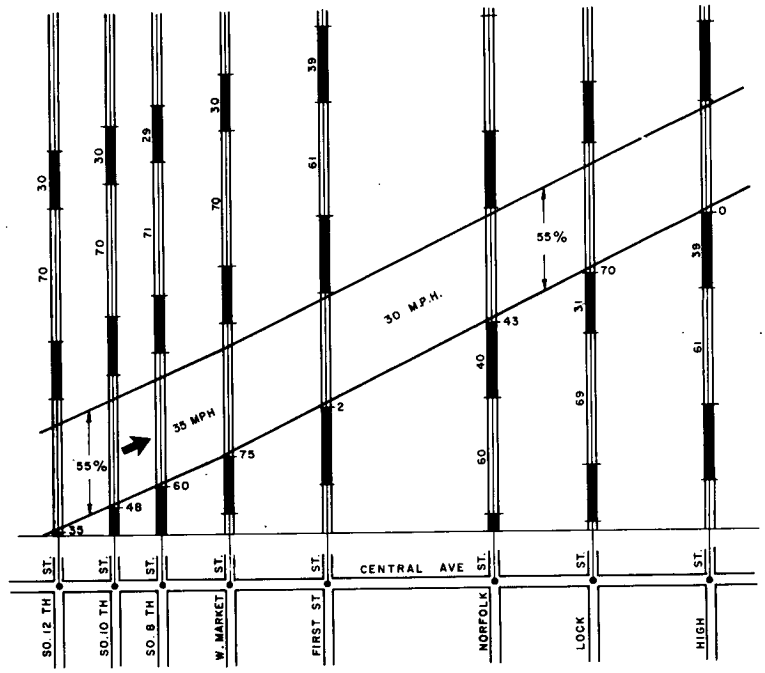


Figure G-148. Inbound progression.

As the traffic signal controllers are noninterconnected (six with one dial and two with two dials) it was necessary to design a three-offset master dial to be installed in each local controller. The "master" is a dial provided with three offsets that are wired through a three-position rotary selector switch to the coil of a normally closed single-pole relay. The output side of the relay is connected to the single-offset circuit of the local controller.

When an offset is selected, the local dial will dwell at the beginning of Central Avenue "green" until released by the proper offset key in the master dial (Figs. G-150 and G-151).

Use of the master dial in each controller required only the ability to turn the rotary selector switch to the correct position at the proper time of day. It took approximately 15 min, including travel time, to change seven controllers.

The surveillance system consisted of vehicle counts and speed and delay runs. These were made at the following locations during the AM and PM time periods:

TYPES OF MEASUREMENT	LOCATION	INFORMATION RECORDED
Speed and delay runs	Central Ave. (East Orange city line to High St.)	Travel time Delay time Number of stops
Vehicle counts	Central Ave. approaches to West Market St., First St., and High St.	Vehicles through per cycle Vehicles stopped per cycle

The "before" measurements were taken between March 19, 1969, and April 7, 1969; and the "after" measurements were conducted between April 15, 1969, and April 30, 1969.

Analysis

The results of the analysis performed on the number of vehicles stopped per cycle and the number of vehicles

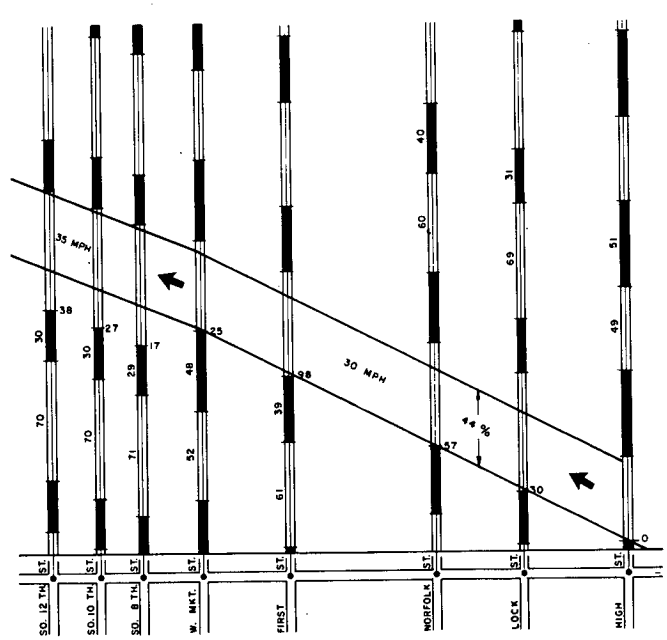


Figure G-149. Outbound progression.

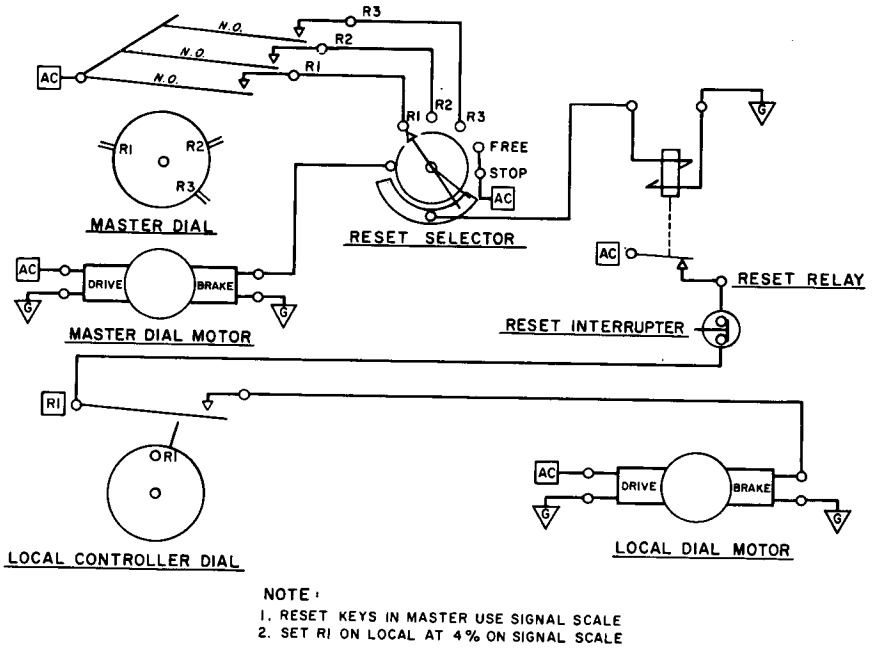


Figure G-150. Master dial.

through per cycle are summarized in Tables G-120 and G-121 for the intersections of Central Avenue with High Street and West Market Street, respectively. Inspection of the data collected during the AM period at the intersection of Central Avenue with First Street indicated that no appreciable changes were evident for either the number of vehicles stopped per cycle or the number of vehicles through per cycle. During the PM peak hour the data recorded at Central Avenue and First Street were very inconsistent, preventing the formulation of conclusions.

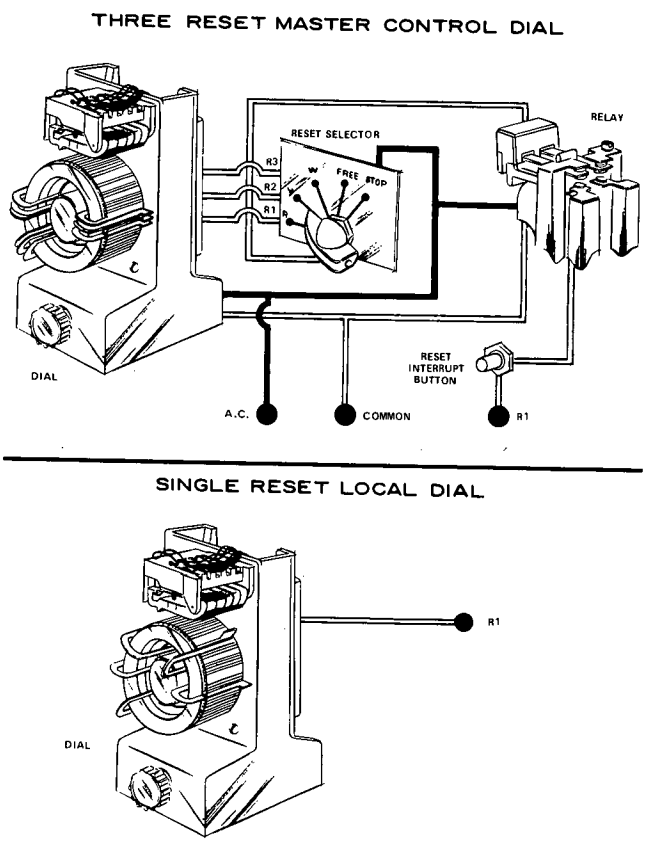


Figure G-151. Master dial.

From Tables G-120 and G-121 it is evident that fewer vehicles are being stopped on Central Avenue in the peak directions at West Market Street during both peak-hour time periods and at Central Avenue eastbound at High Street during the AM peak hour. Inasmuch as the westbound progression started at High Street, it was not expected that the number of vehicles stopped per cycle would be significantly influenced or changed for the Central Avenue westbound approach. The slight decrease in the number of vehicles through per cycle at High Street in the PM (−8.3 percent) was probably due to a change in traffic demand characteristics and is not considered to be significant, as the condition did not result in any additional traffic delays.

Speed and delay data were analyzed to detect significant differences in the time required to travel between the East Orange city line and High Street—a distance of approximately 7,600 ft. Although the results of the intersection analysis seemingly indicate that a significant improvement was made to traffic on Central Avenue, the analysis of the speed and delay runs does not substantiate these findings. An ANOVA performed on the travel time data indicated that no significant differences exist between the "before" and "after" means for the two time periods and two directions considered. Table G-122 summarizes the results of the analysis of the travel time data.

TABLE G-120

SUMMARY OF INTERSECTION ANALYSIS, CENTRAL AVENUE AND HIGH STREET

DIRECTION OF TRAVEL	TIME PERIOD	MEAN NO. OF VEHICLES STOPPED PER CYCLE			MEAN NO. OF VEHICLES THROUGH PER CYCLE		
		"BEFORE"	"AFTER"	DIFFERENCE (%)	"BEFORE"	"AFTER"	DIFFERENCE (%)
EB	AM	14.7	7.7	−47.6 [a]	25.5	24.9	−2.4
WB	PM	21.0	21.2	0.9	25.4	23.3	−8.3

[a] Significant at $\alpha = 0.05$.

TABLE G-121

SUMMARY OF INTERSECTION ANALYSIS, CENTRAL AVENUE AND WEST MARKET STREET

DIRECTION OF TRAVEL	TIME PERIOD	MEAN NO. OF VEHICLES STOPPED PER CYCLE			MEAN NO. OF VEHICLES THROUGH PER CYCLE		
		"BEFORE"	"AFTER"	DIFFERENCE (%)	"BEFORE"	"AFTER"	DIFFERENCE (%)
EB	AM	16.2	6.2	−61.7 [a]	48.9	49.4	+1.0
WB	PM	32.6	22.8	−30.1 [a]	23.9	23.0	−3.8

[a] Significant at $\alpha = 0.05$.

Investigations into the travel time and delay time incurred between various checkpoints along the route were made to enable the designer to have a better understanding of the traffic flow characteristics. The mean times given in Tables G-123 through G-126 represent the time elapsed between checkpoints; for example, the value of 11.0 sec given under the subheading of South 12th Street for the "before" condition in Table G-123 represents the mean delay time between the start of the run at the East Orange city line and the checkpoint at South 12th Street. It is evident from these tables that the improvements made between certain checkpoints were partially offset by increased delays and travel time between other checkpoints along the same run. Table G-125 illustrates this trend by showing that, although 16.6 sec of delay time were saved between the start of the run at High Street and the checkpoint at Norfolk Street, 8.7 sec of delay time were added as increased delay time at First Street. This method of investigation is particularly useful for determining critical points along the route that may require additional green time or a revised offset.

A comparison was made to determine how well the stop ratio calculated from the speed and delay data compared with the stop ratio calculated from data collected as part of the intersection counts. Although no statistical tests were performed to detect differences between the two methods, it can be seen that some of the means do seem to be considerably different. It also can be seen that the directions of change between the "before" and "after" conditions are not the same for each method. Although it is not necessarily expected that the two sets of data will agree in magnitude, they should agree in direction. The mean values investigated are given in Table G-127.

TABLE G-122

SUMMARY OF TRAVEL TIME DATA

DIRECTION OF TRAVEL	TIME PERIOD	MEAN VALUE (SEC)		DIFFERENCE (%)	SIG. @ $\alpha=0.05$
		"BEFORE"	"AFTER"		
EB	AM	312.6	327.4	+4.7	No
	PM	383.7	386.9	+0.8	No
WB	AM	270.0	301.7	+11.7	No
	PM	451.2	416.9	−7.6	No

It is possible that the sample size for the speed and delay data, although adequate to estimate the travel time, number of stops, and delay time over the entire length of run, may not be sufficient to reliably predict the stops and delays at any specific point. Over the entire length of roadway the speed and delay data do indicate fewer stops and fewer stops at signals (Table G-128).

TABLE G-123

MEAN DELAY TIME BETWEEN CHECKPOINTS, CENTRAL AVENUE EASTBOUND, AM

CONDITION	MEAN DELAY TIME (SEC), BY CHECKPOINT					
	SOUTH 12TH ST.	WEST MARKET ST.	FIRST ST.	NORFOLK ST.	LOCK ST.	HIGH ST.
"Before"	11.0	14.2	2.8	18.4	6.7	16.7
"After"	11.0	15.3	9.6	6.1	2.9	28.0
Difference	0.0	1.1	6.8	−12.3	−3.8	11.3
Percent change	0.0	+7.7	+242.9	−66.8	−56.7	+67.7

TABLE G-124

MEAN TRAVEL TIME BETWEEN CHECKPOINTS, CENTRAL AVENUE EASTBOUND, AM

CONDITION	MEAN TRAVEL TIME (SEC), BY CHECKPOINT					
	SOUTH 12TH ST.	WEST MARKET ST.	FIRST ST.	NORFOLK ST.	LOCK ST.	HIGH ST.
"Before"	43.6	65.1	38.7	71.0	37.2	55.6
"After"	49.7	71.2	45.6	54.6	31.6	74.5
Difference	6.1	6.1	6.9	−16.4	−5.6	18.9
Percent change	+14.0	+9.4	+17.8	−23.1	−15.1	+34.0

TABLE G-125

MEAN DELAY TIME BETWEEN CHECKPOINTS, CENTRAL AVENUE WESTBOUND, PM

CONDITION	MEAN DELAY TIME (SEC), BY CHECKPOINT					
	LOCK ST.	NORFOLK ST.	FIRST ST.	WEST MARKET ST.	SOUTH 12TH ST.	EAST ORANGE CITY LINE
"Before"	7.0	16.5	16.4	64.7	40.8	1.0
"After"	1.8	5.1	25.1	57.3	29.6	3.3
Difference	−5.2	−11.4	8.7	−7.4	−11.2	2.3
Percent change	−74.3	−69.1	+53.0	−11.4	−27.5	+230.0

TABLE G-126

MEAN TRAVEL TIME BETWEEN CHECKPOINTS, CENTRAL AVENUE WESTBOUND, PM

CONDITION	MEAN TRAVEL TIME (SEC), BY CHECKPOINT					
	LOCK ST.	NORFOLK ST.	FIRST ST.	WEST MARKET ST.	SOUTH 12TH ST.	EAST ORANGE CITY LINE
"Before"	37.6	48.0	76.6	139.9	109.5	38.8
"After"	32.3	32.7	83.7	120.0	109.4	38.5
Difference	−5.3	−15.3	7.1	−19.9	−0.1	−0.3
Percent change	−14.1	−31.9	+9.3	−14.2	−0.1	−0.8

TABLE G-127

COMPARISON OF STOP RATIOS, CENTRAL AVENUE

		MEAN VALUE			
		"BEFORE"		"AFTER"	
CROSS STREET	DIRECTION AND TIME	SPEED AND DELAY DATA	INTERSECTION DATA	SPEED AND DELAY DATA	INTERSECTION DATA
West Market St.	EB, AM	10.5	33.1	17.5	12.2
	WB, PM	94.1	141.4	92.3	100.8
High St.	EB, AM	47.4	57.5	60.9	30.7
	WB, PM		Not in study area		

TABLE G-128

COMPARISON OF STOPS

	EB—AM		WB—PM	
MEASUREMENT	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
Total stops per run	3.32	3.26	6.47	5.08
Percent change		−1.8		−21.5
Traffic signal stops per run	2.63	1.78	3.35	3.00
Percent change		−32.3		−10.4
Non-traffic signal stops per run	0.68	1.48	3.12	2.08
Percent change		+117.6		−33.3

Conclusions

Based on the foregoing analysis, it can be concluded that the implementation of the inbound and outbound progression plans did not change the travel time or delay time for the average vehicle traveling on Central Avenue; but the number of stops was significantly reduced. The analysis also indicated that the number of stops at signalized intersections was reduced, and the resulting delay per stop was increased.

The number of stops at nonsignalized intersections was increased from 0.68 stop per eastbound run in the "before" period to 1.48 stops per run in the "after" period. This increase may be attributed to the following observed conditions:

1. Repairs to underground utilities during the "after" period resulted in temporary uneven patches in the pavement or steel plates over pavement openings. These patches or plates caused traffic to slow or stop.
2. Vehicles entering two automobile repair shops often formed queues that extended into the street.
3. Cross-street traffic at nonsignalized intersections frequently blocked Central Avenue traffic during the "after" condition.
4. Virtual elimination of queues made more time available to move traffic rather than to start queues. Where three lanes had been required by volume of traffic with the average progression plan, fewer than three lanes were needed with the inbound and outbound plan. This condition resulted in more drivers parking illegally in restricted areas. Drivers, therefore, tended to avoid use of the curb lane for travel.
5. At the beginning of the school year the majority of police officers assigned to traffic duty are assigned to school-crossing duty, and the arterial street parking prohibitions are left unenforced. Inasmuch as parking places are in short supply throughout the area, drivers quickly took advantage of the lack of enforcement and parked illegally during the peak hours.
6. Crossing guards located at nonsignalized intersections, who took advantage of gaps in the traffic stream during the "before" study when the signals turned red, created their own gaps during the "after" condition rather than stop traffic after the platoon had passed in the preferential direction.

Proper timing of signals to provide a band width adequate to accommodate platoons of traffic, enforcement of parking controls to maintain the number of lanes required for moving traffic, and control of nonsignalized intersections to prevent interference with arterial traffic are all required for a successful signal progression system. Implementation of an offset plan without these necessary items will make it impossible to derive the full benefits that are obtainable.

McCarter Highway Signal Progression—Experiment B93

Experiment B93 was designed to evaluate the relative merits of three types of signal offset plans to accommodate peak-period traffic movement on an urban arterial street. These were a simultaneous offset plan, an average offset plan providing for progression in both directions of movement, and an outbound offset plan permitting progression in the dominant direction of movement. Signal phasing also was revised as necessary.

The revision of offset relationships was accomplished by changing the offset timings of the local controllers. Owing to the age and condition of the equipment and the inherent limited accuracy of the electronic timing circuitry, it was then necessary to verify the resulting offset, using a stopwatch. Phasing changes were accomplished in a similar manner.

Experimental Area

Experiment B93 was conducted on McCarter Highway (N.J. 21) in Newark, between Clay Street on the north and Poinier Street on the south, a distance of 13,620 ft (2.58 miles). As a result of the analog signal model analysis, Broad Street from Miller Street to Poinier Street, Mulberry Street from Kinney Street to McCarter Highway, and Raymond Boulevard from Raymond Plaza East to McCarter Highway were included in the experimental area, to eliminate restrictions to favorable traffic movement on McCarter Highway.

The experimental area (Fig. G-152) includes 35 signalized intersections. Within this area, McCarter Highway is intersected by 38 different streets, 29 of which are provided with traffic-signal control. These signals are part of a PR system with 117 local units that operate on a 90-sec cycle length. The simultaneous offset plan and the split relationships existing in the "before' conditions are shown in Figure G-153 for the area between Poinier Street to Market Street (Segment A), and in Figure G-154 for the area between Market Street and Clay Street (Segment B).

McCarter Highway is one of the major north-south arterial streets in the city of Newark, with an average daily traffic volume (ADT) of from 33,000 to 38,000 vehicles. At Poinier Street it terminates into ramps leading to and from U.S. 1, 9, and 22. North of Clay Street it becomes a controlled-access facility. From Poinier Street to Lafayette Street, McCarter Highway parallels the right-of-way of the Pennsylvania Railroad, located to the east. Where access to McCarter Highway is practical, mixed types of commercial land use are present. Together with Broad Street, it provides a travel route for the majority of north-south traffic in Newark. A high percentage of through vehicles and trucks are present during all time periods, as indicated in Experiment A7, Raymond Boulevard and McCarter Highway, Left-Turn and Pedestrian Control.

The curb-to-curb width of McCarter Highway in the experimental area varies from 50 to 87 ft. Except as described as follows, pavement markings consisted of a double yellow center line. Traffic generally operated in two lines for each direction, because parking was prohibited during AM and PM peak traffic periods. Exceptions to these conditions occurred between Poinier and Wright Streets and between Lafayette and Centre Streets. Between Poinicr and Wright Streets the maximum street width of 87 ft was used by four lanes of northbound traffic.

The two left lanes were used by vehicles turning left into Wright Street to go north on Broad Street. The two right lanes were used by northbound traffic. The center-line marking was offset to encourage such operations. Between Wright Street and Lafayette Street the curb-to-curb width of 58 ft remained constant. From Lafayette Street to Commerce Street the curb-to-curb width of 66 ft permitted traffic to operate in three lanes in each direction. From Commerce Street to Park Street a curb-to-curb width of 76 ft was used to provide auxiliary left-turn lanes and three other lanes for each direction of McCarter Highway traffic at Raymond Boulevard by means of the lane markings applied in Experiment A7. From Park Street to Centre Street a curb-to-curb width in excess of 76 ft, with only the center line marked, resulted in traffic operating in three lines for each direction. From Centre Street to Clay Street a basic curb-to-curb width of 58 ft along a curving alignment reduced flow to two lines of traffic in each direction. The minimum roadway width in the experimental area is 50 ft and is located between the bridge abutments of the Lackawanna Railroad overpass.

Experimental Design

The first stage of Experiment B93 involved changing the "existing" simultaneous offset plan to an "average" offset plan providing progressive movement for both northbound and southbound traffic flows for all time periods.

For Segment A, a progression speed of 26.8 mph was provided for northbound traffic from Vanderpool Street to Market Street, and 26.4 mph for southbound traffic from Lafayette Street to Poinier Street (Fig. G-155). The proximity of the signalized intersections between Chestnut and Elm Streets, requiring almost simultaneous offset operation, resulted in a relatively narrow green band for southbound traffic. In addition, intersection spacing required southbound traffic to stop at Lafayette Street.

For Segment B, the proximity of the signalized intersections between Saybrook Place and Orange Street again required an almost simultaneous operation and resulted in a designed stop for northbound traffic at Fulton or Lombardy Streets. Conversely, this offset operation resulted in a relatively wide green band for southbound traffic from Clay Street to Market Street, at a speed of 26.4 mph from Clay Street to Bridge Street and 31.0 mph from Bridge Street to Market Street (Fig. G-156).

The second stage involved revising the average offset plan to provide for preferential outbound flow during the PM time period. This change was possible because the system master controller was continuing to specify average, inbound, and outbound offset operations established as a result of Experiment B88, Springfield Avenue Signal Progression. This outbound offset plan was designed to provide preferential flow southbound from Market Street (Segment A) and northbound from Market Street (Segment B), as shown in Figures G-157 and G-158, respectively. These plans were designed to provide for maximum band width

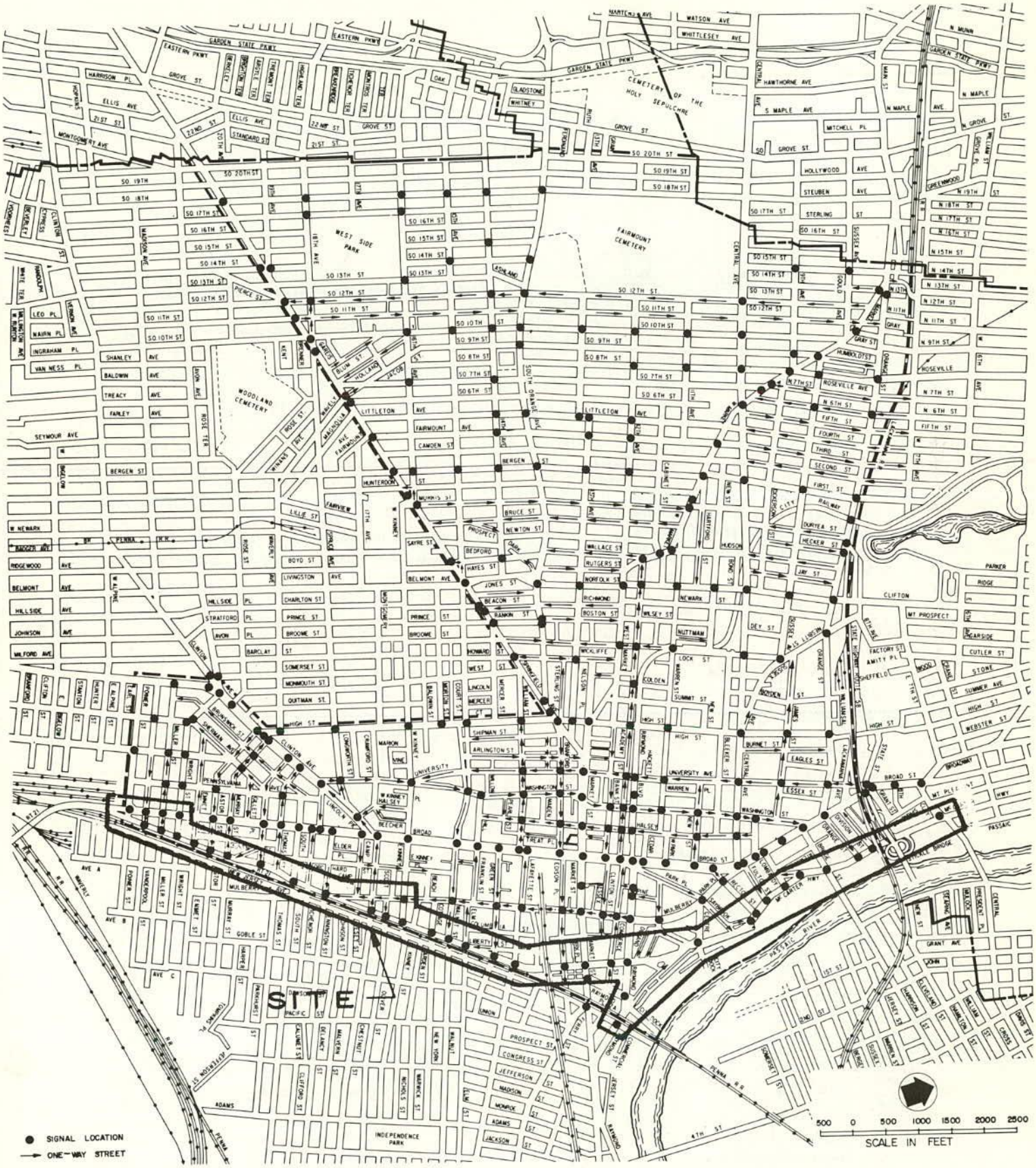


Figure G-152. Location map, Experiment B93.

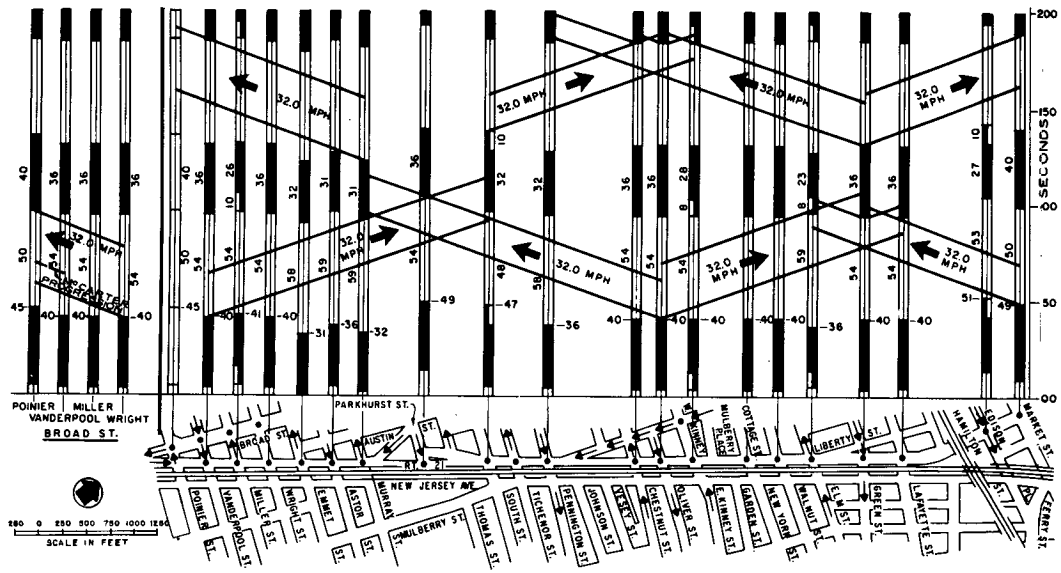


Figure G-153. "Before" simultaneous offset plan, Segment A.

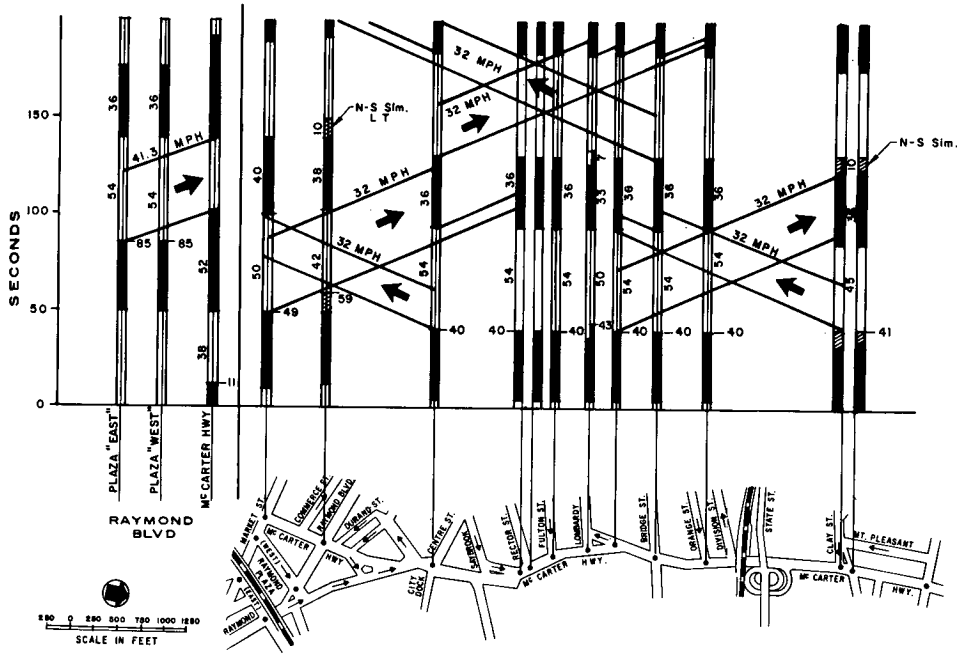


Figure G-154. "Before" simultaneous offset plan, Segment B.

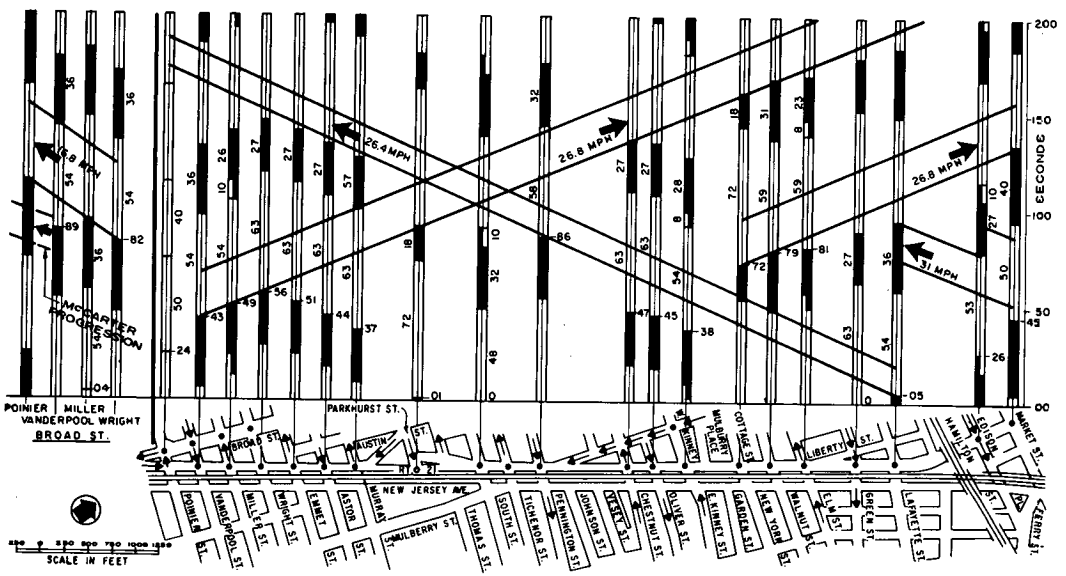


Figure G-155. Average offset plan, Segment A.

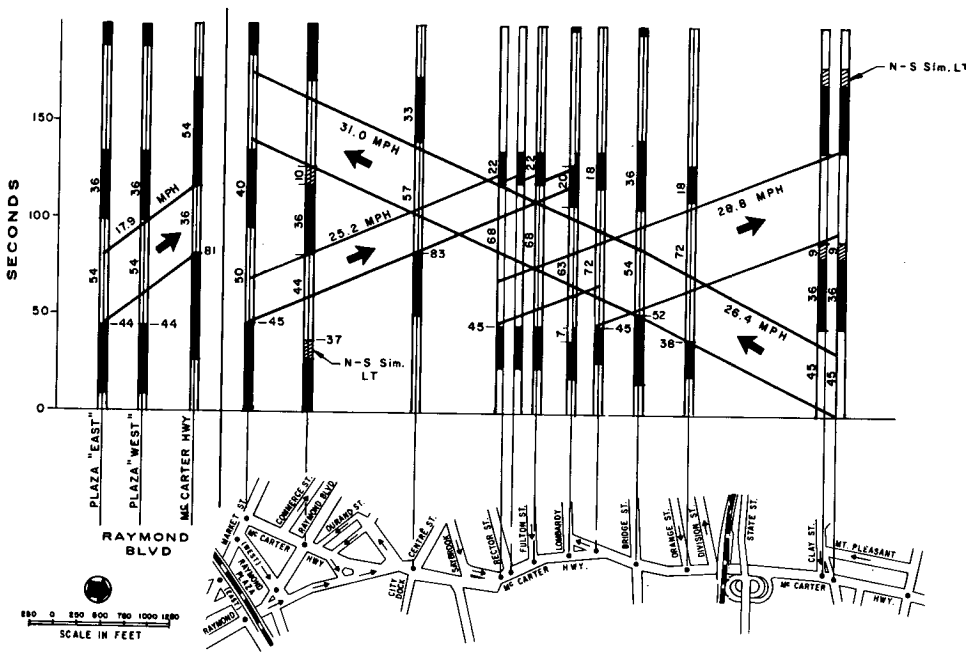


Figure G-156. Average offset plan, Segment B.

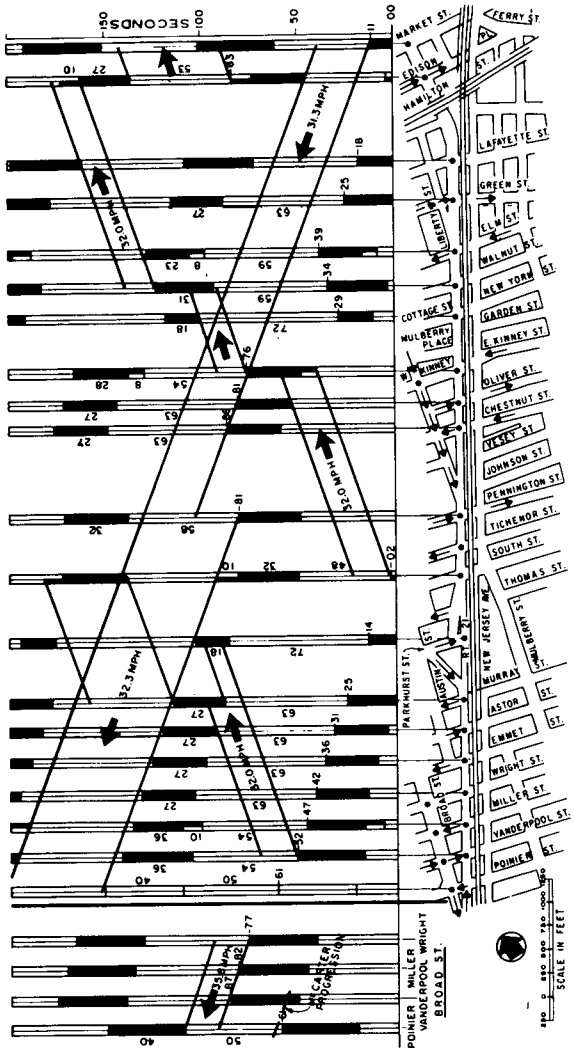


Figure G-157. Outbound offset plan, Segment A.

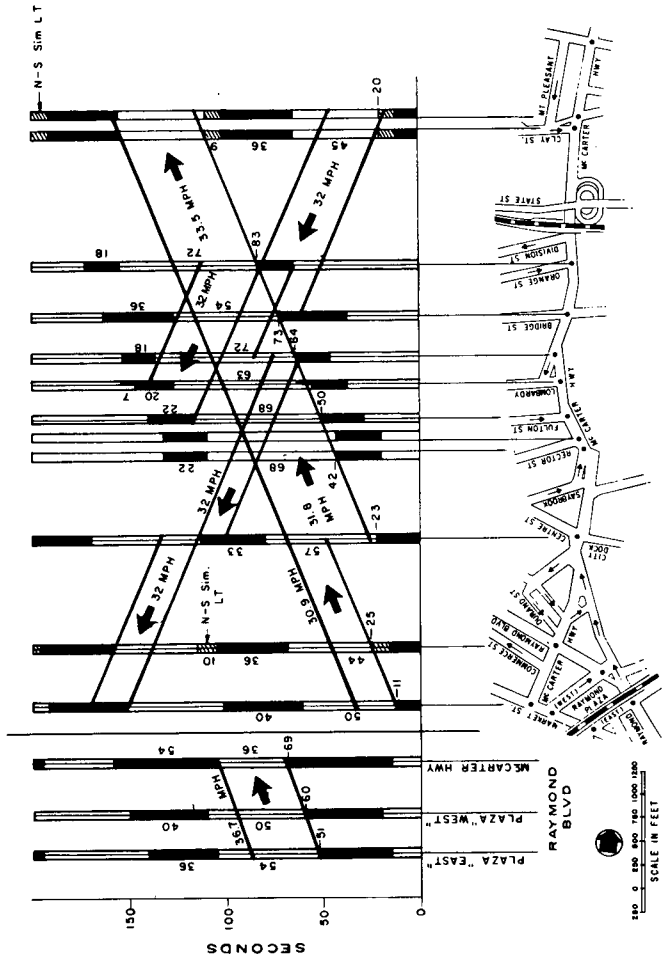


Figure G-158. Outbound offset plan, Segment B.

in the direction of preferential flow while considering the effect on the opposite direction.

Dates of implementation and surveillance measurements are as follows:

CONDITION	DATES
"Before"	5/27/69 to 6/13/69
Stage 1	6/26/69
"After" Stage 1	6/27/69 to 7/3/69
Stage 2	7/8/69
"After" Stage 2	7/8/69 to 7/11/69

All surveillance measurements were taken during the AM and PM time periods except after Stage 2, when data were collected only during the PM period. The measurements included:

1. Automatic Traffic Recorder (ATR) equipment was installed in Segments A and B.
2. Speed and delay runs were made from 7:00 to 9:00 AM and 4:00 to 6:00 PM. Southbound runs were made from the Erie-Lackawanna Railroad overpass to the intersection of Poinier Street with Broad Street, a distance of 12,400 ft (2.348 miles). Northbound runs were made from Miller Street to the same overpass, a distance of 12,135 ft (2.298 miles). These different lengths resulted from the turn-around maneuvers dictated by the one-way-street operations in the Poinier Street area. All northbound runs were begun after the survey vehicle entered McCarter Highway from Vanderpool Street, resulting in an automatic stop at Miller Street due to signal-offset operations. This condition should be taken into account when examining the data for northbound speed and delay runs.

Analysis

Data summarized from ATR counts are given in Table G-129. As indicated, many of the differences are statistically significant.

The effects of the progression plan on traffic flow on McCarter Highway were determined by "before" and "after" comparisons of the following speed and delay run characteristics:

1. The time required to travel the facility (trip time).
2. The time stopped during trips along this facility (delay time).
3. The number of stops occurring during each trip on the facility.

TABLE G-129

ATR SUMMARY, MEAN VOLUMES

LOCATION	CONDITION	NB 7–9 AM	NB 4–6 PM	NB 24 HR	SB 7–9 AM	SB 4–6 PM	SB 24 HR	BOTH DIRECTIONS, 24 HR
McCarter Hwy. between Murray St. and Parkhurst St. (Segment A)	"Before"	2,196	1,941	15,646	1,750	2,546	16,820	33,538
	"After" average	2,247	2,150	—	1,793	2,600	17,066	—
	Net change	+51	+209	—	+43	+54	+246	—
	Percent change	+2.3	+10.8	—	+2.4	+2.1	+1.5	—
	Sig. @ $\alpha=0.05$	No	No	—	No	No	No	—[a]
	"After" outbound		2,041	16,537		2,476	17,703	34,240
	Net change (from "before")		+100	+891		−70	−883	+702
	Percent change		+4.9	+5.7		−2.7	+5.2	+2.1
	Sig. @ $\alpha=0.05$		No	No		No	Yes	—[a]
McCarter Hwy. between Centre St. and Saybrook Place (Segment B)	"Before"	2,005	1,788	16,224	2,744	2,527	20,458	36,622
	"After" average	2,805	2,496	19,672	1,981	2,121	18,368	38,040
	Net change	+800	+708	+3,448	−763	−406	−1,090	+1,418
	Percent change	+39.9	+39.6	+21.3	−27.8	−16.1	−10.2	−3.9
	Sig. @ $\alpha=0.05$	Yes	Yes	Yes	Yes	Yes	Yes	—[a]
	"After" outbound		2,265	18,601		2,286	18,162	36,751
	Net change (from "before")		+477	+2,377		−241	−2,296	+129
	Percent change		+26.7	+14.7		−9.5	−11.2	+0.4
	Sig. @ $\alpha=0.05$		Yes	Yes		Yes	Yes	—[a]

[a] Data not analyzed.

Tables G-130 and G-131 summarize the statistical analysis of "before" and "after" speed and delay measurements for Segments A and B combined and give the mean values observed for each factor.

In the AM time period, only the change from the simultaneous offset plan to the average offset plan was implemented, and the comparisons indicated the following to be statistically significant:

TABLE G-130

SUMMARY OF ANALYSIS, NORTHBOUND TRAFFIC, SEGMENTS A AND B

CONDITION	NO. OF RUNS	TRIP TIME (SEC)	DELAY (SEC)	NO. OF STOPS
(*a*) AM time period				
"Before"	23	650.8	254.4	8.1
"After" average	24	570.0	164.2	6.5
Net change	—	−80.8	−90.2	−1.6
Percent change	—	−12.4	−35.5	−19.8
Sig. level	—	0.005	0.0005	0.005
(*b*) PM time period				
"Before"	22	763.8	350.3	10.7
"After" average	9	578.6[a]	179.4[a]	7.6[b]
Net change	—	−185.2	−170.9	−3.1
Percent change	—	−24.2	−48.8	−29.0
Sig. level	—	0.005	0.0005	0.01
"After" outbound	16	568.0	185.9	7.4
Net change (from "before")	—	−195.8[a]	−164.4[a]	−3.3[a]
Percent change	—	−25.6	−46.9	−30.8
Sig. level	—	0.005	0.0005	0.005

[a] Variability reduced at $\alpha = 0.005$.
[b] Variability reduced at $\alpha = 0.100$.

TABLE G-131

SUMMARY OF ANALYSIS, SOUTHBOUND TRAFFIC, SEGMENTS A AND B

CONDITION	NO. OF RUNS	TRIP TIME (SEC)	DELAY (SEC)	NO. OF STOPS
(*a*) AM time period				
"Before"	23	644.9	237.6	7.7
"After" average	23	554.0[a]	165.0	6.8
Net change	—	−90.9	−72.6	−0.9
Percent change	—	−14.1	−30.6	−11.7
Sig. level	—	0.0005	0.0005	0.05
(*b*) PM time period				
"Before"	22	672.0	251.9	8.3
"After" average	18	625.2	202.7	8.2
Net change	—	−46.8	−49.2	−0.1
Percent change	—	−7.0	−19.5	−1.2
Sig. level	—	0.10	0.005	NS
"After" outbound	17	542.4	157.2	5.8
Net change (from "before")	—	−129.6	−94.7	−2.5
Percent change	—	−19.3	−37.6	−30.1
Sig. level	—	0.0005	0.0005	0.0005

[a] Variability reduced at $\alpha = 0.025$.
NS = not significant at $\alpha \leqq 0.10$.

1. For northbound traffic, total trip time decreased by 80.8 sec (12.4 percent), delay time decreased by 90.2 sec (35.5 percent), and the number of stops decreased by 1.6 stops (19.8 percent).

2. For southbound traffic, total trip time decreased by 90.9 sec (14.1 percent), delay time decreased by 72.6 sec (30.6 percent), and the number of stops decreased by 0.9 stop (11.7 percent).

In terms of quality of flow, these changes become reductions of 0.6 min per mile in trip time, 0.7 min per mile in delay time, and 0.7 stop per mile for northbound traffic; and, similarly, reductions of 0.6 min per mile in trip time, 0.5 min per mile in delay time, and 0.4 stop per mile for southbound traffic. Average speed increased from 12.72 to 14.52 mph for northbound traffic and from 13.11 to 15.26 mph for southbound traffic. As indicated in Table G-129, these improvements in northbound quality of flow were accompanied by a 39.9-percent increase in northbound volume and a 27.8-percent decrease in southbound volume for Segment B, with only small increases in volume measured for Segment A.

These improvements indicated that the average offset plan was preferable to the simultaneous offset plan under the conditions observed. The relatively low average speeds observed after the improvement, compared to the 25.2 to 31.0-mph speeds of progression, reflect the frequent stops encountered, which may be attributable to the combination of a relatively narrow green band and congestion at intersections resulting from left-turning vehicles.

In the PM time period, the comparisons of data measured for the change from the simultaneous offset plan to the average offset plan for Segments A and B combined indicated the following to be statistically significant:

1. For northbound traffic, total trip time decreased by 185.2 sec (24.2 percent), delay time decreased by 170.9 sec (48.8 percent), and the number of stops decreased by 3.1 stops (29.0 percent).

2. For southbound traffic, total trip time decreased by 46.8 sec (7.0 percent) and delay time decreased by 49.2 sec (19.5 percent).

In terms of quality of flow, these improvements became reductions of 1.3 min per mile in trip time, 1.2 min per mile in delay time, and 1.3 stops per mile for northbound traffic; and, similarly, reductions of 0.3 min per mile in trip time and 0.3 min per mile in delay time for southbound traffic. Average speed increased from 10.83 to 14.30 mph for northbound traffic and from 12.58 to 13.52 mph for southbound traffic. These values when compared to the similar values for the AM time period indicated almost twice as much improvement for northbound traffic and slightly less improvement for southbound traffic. These improvements indicated that the average offset plan was preferable to the simultaneous offset plan under the conditions observed.

Comparisons of the measured data in the PM time period for Segments A and B combined for the change from the "before" conditions (simultaneous offset) to the "after" outbound offset indicated almost identical improvements for northbound traffic as were obtained with the average offset. These were reductions of 1.4 min per mile in travel time, 1.2 min per mile in delay time, and 1.4 stops per mile, with an increase in average speed from 10.83 to 14.56 mph. However, the improvements for southbound traffic realized with the outbound offset increased compared with those realized with the average offset. Reductions of 0.9 min per mile in trip time, 0.7 min per mile in delay time, and 1.1 stops per mile of travel were realized by southbound traffic. In addition, average speed increased from 12.58 to 15.59 mph.

Because the outbound offset was designed to provide for preferential progression away from Market Street, speed and delay data were also analyzed by sections (Segments A and B). Tables G-132 and G-133 summarize the comparisons of "before" and "after" measurements for all three offset plans.

For Segment A, a comparison of the conditions "before" and "after" the average offset plan indicated the following to be statistically significant:

1. A decrease in delay time of 36.3 sec (30.0 percent) for northbound traffic.

2. An increase in the number of stops of 1.3 (26.5 percent) for southbound traffic.

In terms of miles of travel for the 7,170-ft (1.358-mile) northbound travel route and the 7,435-ft (1.408-mile) southbound travel route of Segment A, these changes represent a decrease of 0.4 min in delay time per mile of travel for northbound traffic and an increase of 0.9 stop per mile for southbound traffic. As indicated in Table G-129, the decrease in delay time for northbound traffic was accompanied by a 10.8-percent increase in traffic volume; southbound traffic increased only 2.1 percent.

A comparison of the conditions "before" (simultaneous offset) and "after" the outbound offset plan indicated the following to be statistically significant:

1. Increases in travel time of 48.3 sec (15.6 percent) and in the number of stops of 1.9 stops (51.4 percent) for northbound traffic.

2. Decreases in travel time of 92.3 sec (23.3 percent), in delay time of 86.2 sec (56.3 percent), and in the number of stops of 2.4 stops (49.0 percent) for southbound traffic.

In terms of miles of travel, these changes in quality of flow were increases of 0.6 min per mile in trip time and 1.4 stops per mile for northbound traffic, with contrasting reductions for southbound traffic of 1.1 min per mile in trip time, 1.0 min per mile in delay time, and 1.7 stops per mile. Average speed decreased from 15.76 to 13.64 mph for northbound traffic and increased from 12.83 to 16.72 mph for southbound traffic. Table G-129 indicates that these changes were accompanied by an increase of 4.9 percent and a decrease of 2.7 percent in volumes for northbound and southbound traffic, respectively.

With a predominant southbound direction of flow for Segment A, the over-all effects of the outbound offset plan were a marked improvement in operations for this section of the experimental area.

For Segment B, a comparison of the conditions "before"

TABLE G-132

SUMMARY OF ANALYSIS, NORTHBOUND, P.M. TIME PERIOD

DATA GROUP	SEGMENT A NO. OF RUNS	SEGMENT A TRAVEL TIME (SEC)	SEGMENT A DELAY TIME (SEC)	SEGMENT A NO. OF STOPS	SEGMENT B NO. OF RUNS	SEGMENT B TRAVEL TIME (SEC)	SEGMENT B DELAY TIME (SEC)	SEGMENT B NO. OF STOPS
"Before"	22	310.0	120.5	3.7	22	456.9	237.5	7.1
"After" average	11	318.7	84.2	4.2	9	223.1 [a]	63.2 [a]	2.8 [a]
Net change		+8.7	−36.3	+0.5		−233.8	−174.3	−4.3
Percent change		+2.8	−30.1	+13.5		−51.2	−73.4	−60.6
Significant (α=0.05)		No	Yes	No		Yes	Yes	Yes
"After" outbound	16	358.3	131.3	5.6	19	206.8	48.9	1.9
Net change (from average)		+39.6	+47.1	+1.4		−16.3	−14.3	−0.9
Percent change		+12.4	+55.9	+33.3		−7.3	−22.6	−32.1
Significant (α=0.05)		Yes	Yes	Yes		No	No	No
Net change (from "before")		+48.3	+10.8	+1.9		−250.1	−188.6	−5.2
Percent change		+15.6	+9.0	+51.4		−54.7	−79.4	−73.2
Significant (α=0.05)		Yes	No	Yes		Yes	Yes	Yes

[a] Variability reduced.

and "after" the average offset plan indicated the following to be statistically significant:

1. Decreases in trip time of 233.8 sec (51.2 percent), in delay time of 174.3 sec (73.4 percent), and in the number of stops of 4.3 (60.6 percent) for northbound traffic.
2. Decreases in trip time of 84.0 sec (31.2 percent), in delay time of 58.4 sec (58.7 percent), and in the number of stops of 1.3 (39.4 percent) for southbound traffic.

In terms of miles of travel for the 4,965-ft (0.940-mile) Segment B, these improvements in quality of flow resulted in reductions of 3.7 min per mile in trip time, 2.7 min per mile in delay time, and 4.0 stops per mile for northbound traffic, with similar reductions for southbound traffic of 1.3 min per mile in trip time, 0.8 min per mile in delay time, and 1.2 stops per mile of travel. Average speeds increased from 7.41 to 15.71 mph for northbound traffic and from 12.57 to 18.28 mph for southbound traffic. Table G-129 indicates these changes were accompanied by a 39.6-percent increase in northbound volume and a 16.1-percent decrease in southbound volume.

A comparison of the conditions "before" (simultaneous offset) and "after" the outbound offset plan indicated the following to be statistically significant:

1. Decrease in trip time of 250.1 sec (54.7 percent), in delay time of 188.6 sec (79.4 percent), and in the number of stops of 5.2 (73.2 percent) for northbound traffic.
2. A decrease in trip time of 30.3 sec (11.3 percent) for southbound traffic.

TABLE G-133

SUMMARY OF ANALYSIS, SOUTHBOUND, PM TIME PERIOD

DATA GROUP	SEGMENT A NO. OF RUNS	SEGMENT A TRAVEL TIME (SEC)	SEGMENT A DELAY TIME (SEC)	SEGMENT A NO. OF STOPS	SEGMENT B NO. OF RUNS	SEGMENT B TRAVEL TIME (SEC)	SEGMENT B DELAY TIME (SEC)	SEGMENT B NO. OF STOPS
"Before"	22	395.5	153.0	4.9	24	269.2	99.5	3.3
"After" average	18	441.7	163.3	6.2	18	185.2	41.1	2.0
Net change		+46.2	+10.3	+1.3		−84.0	−58.4	−1.3
Percent change		+11.7	+6.7	+26.5		−31.2	−58.7	−39.4
Significant (α=0.05)		No	No	Yes		Yes	Yes	Yes
"After" outbound	17	303.2	66.8	2.5	22	238.9	91.0	3.4
Net change (from average)		−138.5	−96.5	−3.7		+53.7	+49.9	+1.4
Percent change		−31.4	−59.0	−59.7		+29.0	+121.4	+70.0
Significant (α=0.05)		Yes	Yes	Yes		Yes	Yes	Yes
Net change (from "before")		−92.3	−86.2	−2.4		−30.3	−8.5	+0.1
Percent change		−23.3	−56.3	−49.0		−11.3	−8.5	+3.0
Significant (α=0.05)		Yes	Yes	Yes		Yes	No	No

In terms of miles of travel, the reductions resulting from the outbound offset plan as compared to the "before" simultaneous offset plan were 4.4 min per mile in trip time, 3.3 min per mile in delay time, and 5.5 stops per each mile of travel for northbound traffic. In addition, average speed for northbound traffic increased from 7.41 to 16.37 mph. For southbound traffic, this comparison indicated a reduction in trip time of 0.5 min per mile of travel, resulting in an increase in average speed from 12.57 to 14.17 mph. Table G-129 indicates that these changes were accompanied by a 26.7-percent increase in northbound volume and a 9.5-percent decrease in southbound volume, reflecting both the emphasis and success achieved in preferential northbound traffic movement.

The improvements for southbound traffic resulting from the average offset plan compared to the simultaneous offset plan were mostly eliminated with the implementation of the outbound offset plan; however, over-all southbound flow was improved when compared to the "before" conditions, and a high degree of flow improvement was achieved for the predominant northbound flow.

Conclusions

For the conditions investigated, the average offset plan appeared to be preferable for safe and efficient operation, when compared to the simultaneous offset plan in both the AM and PM peak traffic periods.

In the AM, the average offset plan resulted in significant reductions of 0.6 and 1.3 min per mile in trip time, of 0.7 and 1.3 min per mile in delay time, and 0.7 and 0.3 stop in the number of stops per mile for northbound and southbound traffic, respectively.

In the PM peak traffic period, the outbound offset plan appeared to be preferable to either the simultaneous or average offset plans. The average offset plan compared to the simultaneous offset plan resulted in significant reductions of 1.3 and 0.3 min per mile in trip time, 1.2 and 0.3 min per mile in delay time, and 1.3 and 0.0 stops in the number of stops per mile for northbound and southbound traffic, respectively. The outbound offset plan compared to the simultaneous offset plan resulted in significant reductions of 1.4 and 0.9 min per mile in trip time, 1.2 and 0.7 min per mile in delay time, and 1.4 and 1.1 stops per mile of travel time for northbound and southbound traffic, respectively.

For Segment A during the PM peak periods, the outbound offset plan compared to the simultaneous offset plan resulted in significant reductions of 1.1 min per mile in trip time, 1.0 min per mile in delay time, and 1.7 stops per mile for the dominant southbound flow. For Segment B, the outbound offset plan compared to the simultaneous offset plan resulted in significant reductions of 4.4 min per mile in trip time, 3.3 min per mile in delay time, and 5.5 stops per mile of travel for the dominant northbound flow, with a 26.7-percent increase in volume.

Broad Street Signal Progression—Experiment B100

Experiment B100 evaluates the effects produced by revising offset relationships between adjacent traffic-signal-controlled intersections on an urban arterial street to achieve preferential traffic movement by using different speeds of progression and varying cycle lengths. The different traffic phases were also revised as necessary to appropriately accommodate the traffic demand for three general categories: off-peak traffic, AM peak traffic, and PM peak traffic.

The offset relationships were revised by first changing the offset timings of the local controllers and then visually calibrating the resulting offset operation, using a stopwatch. This calibration was necessary due to the age and condition of the equipment and the inherent limited accuracy of the electronic timing circuitry. Phasing changes were accomplished in a similar manner.

Experimental Area

Experiment B100 is a continuation of the staged improvements on Broad Street in Newark, begun with Experiment B78, Broad Street Reversible Lanes. The report on Experiment B78 contains information relative to most of the experimental area involved in Experiment B100. Broadway between Crane Street and Clay Street, Park Place between Broad Street and Centre Street, and Broad Street between Central Avenue and New Street were added to the experimental area (Fig. G-159) to eliminate both upstream and downstream restrictions to favorable preferential signal progression. The experimental area included intersections controlled by traffic signals at 14 locations.

Experimental Design

The signal offset and split relationships in effect during the "before" condition, with a 90-sec cycle in constant operation, are shown in Figure G-17. As the system master controller was specifying inbound, outbound, or average offset operations within the interconnected signal system as a result of Experiment B88, Springfield Avenue Signal Progression, separate and distinct offset operations were designed for each of these conditions for this experiment. The experiment was implemented in two stages.

Stage 1 involved lengthening the existing signal cycle during peak traffic periods and providing for a 25-mph preferential signal progression in the direction of dominant flow. During the off-peak traffic periods the existing signal cycle was shortened, and a 30-mph signal progression was provided for both directions of flow.

The inbound offset plan was designed for preferential southbound traffic movement and was used for the period from 7:00 to 9:00 AM. The constant 25-mph signal progression was begun at the intersection of Crane Street with Broadway and extended to the intersection of New Street with Broad Street (Fig. G-160). Additional progression for southbound vehicles turning left into Park Place was provided through the Park Place-Centre Street intersection. A cycle length of 100 sec was used with this offset plan to provide a continuous green band of 56 percent of the cycle.

Signal phasing was revised at the Broad Street-Central Avenue intersection. The high-volume southbound left turn from Broad Street into Park Place, as reported in Experiment B78, was given the maximum advance possible. This was accomplished by manually repositioning the timing dial

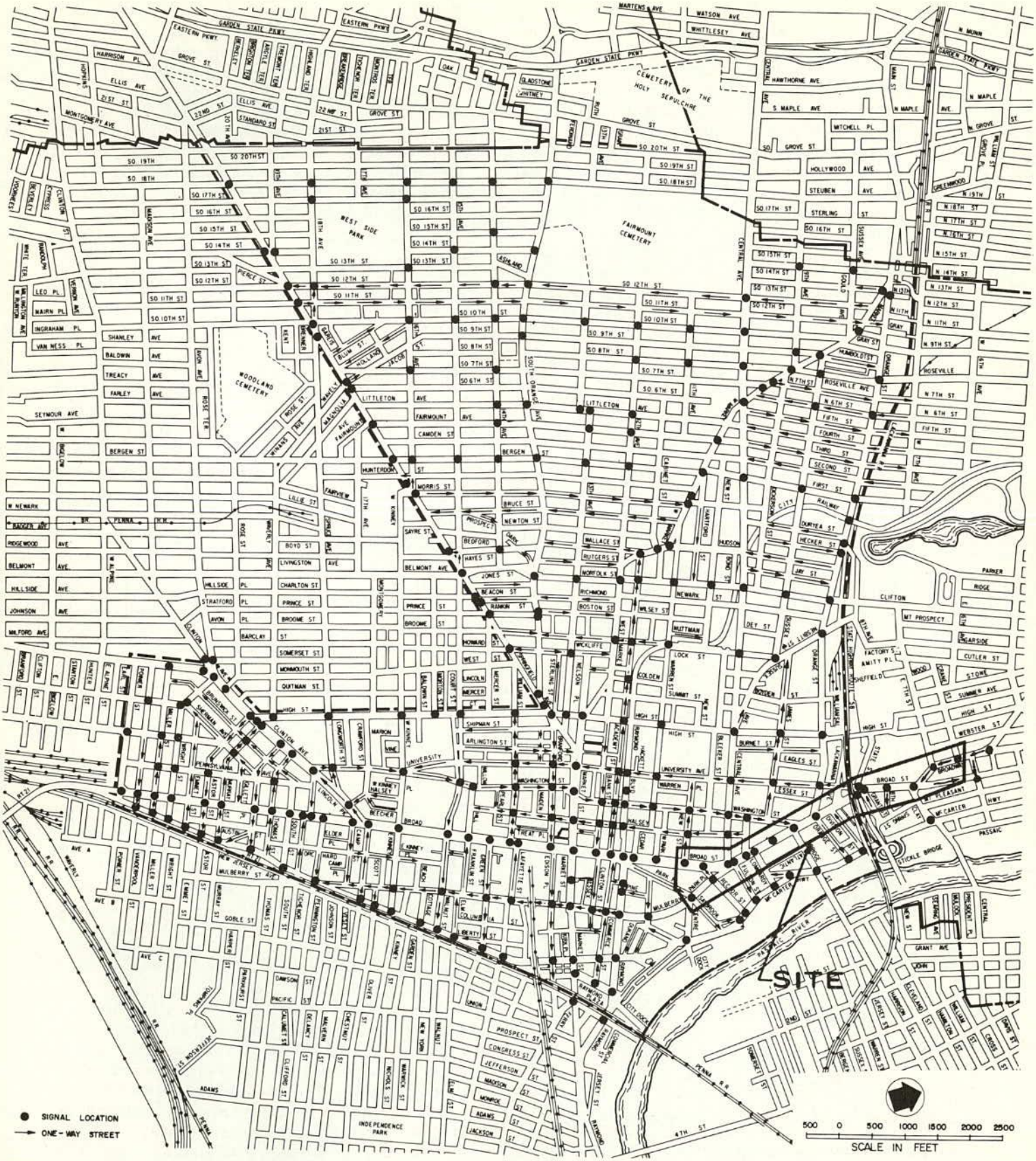


Figure G-159. Location map, Experiment B100.

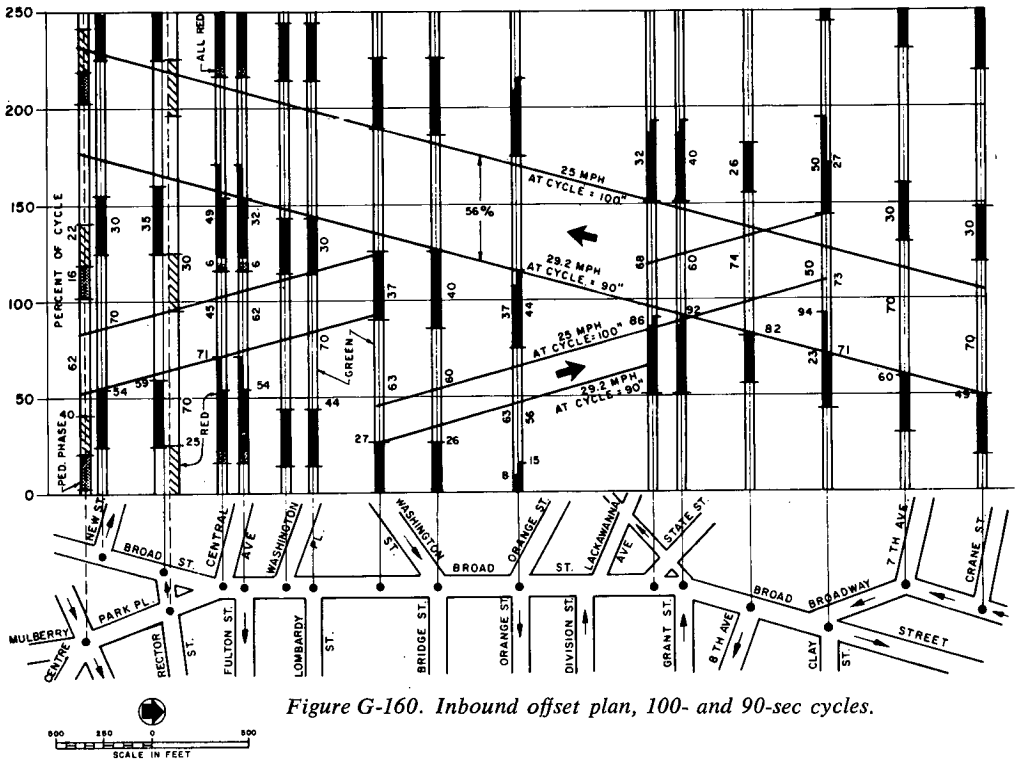


Figure G-160. Inbound offset plan, 100- and 90-sec cycles.

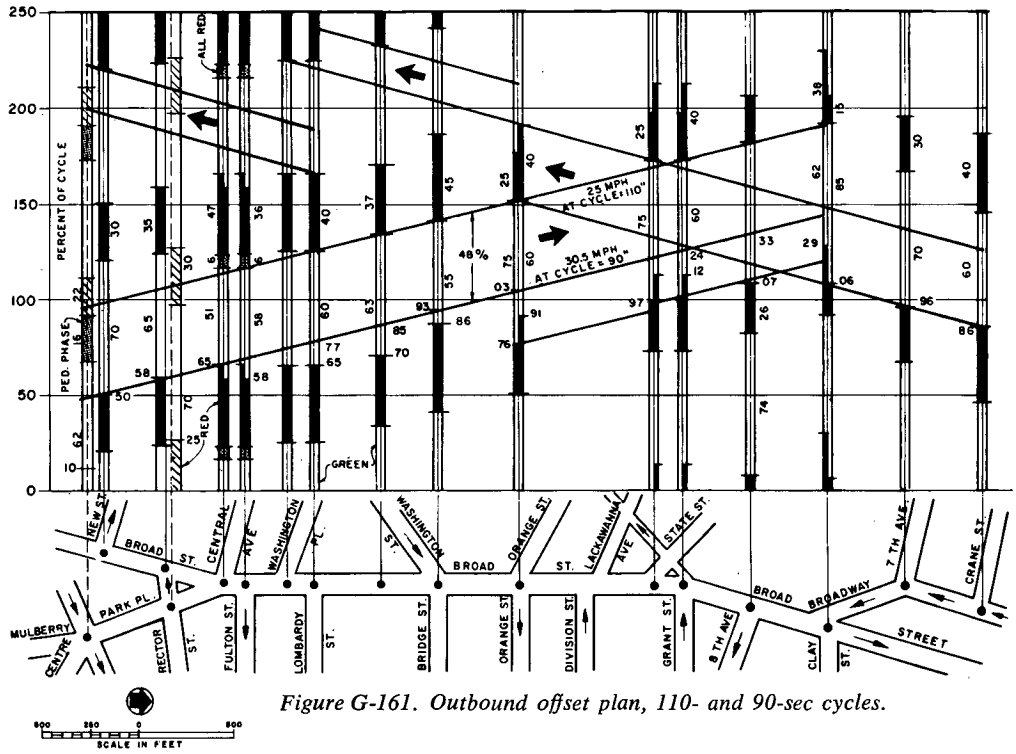


Figure G-161. Outbound offset plan, 110- and 90-sec cycles.

controlling this phase at 7:00 AM and again at 9:00 AM, resulting in a total of 17 sec of advance for this period.

The outbound offset plan was designed for preferential northbound traffic movement and was used for the period from 4:00 to 6:00 PM. The constant 25-mph signal progression was begun at the intersection of New Street with Broad Street and extended to the intersection of Clay Street with Broad Street (Fig. G-161). Additional progression was provided for northbound Park Place traffic from Centre Street through Broad Street onto Central Avenue. A cycle

length of 110 sec was used with this offset plan to provide for a continuous green band.

Signal phasing was revised at the Orange and State Street intersections to give the maximum northbound advance possible, in a manner identical with that described previously for the inbound offset plan at the Broad Street-Central Avenue intersection.

The average offset plan, designed for off-peak operation, was intended to provide for simultaneous northbound and southbound progression. A cycle length of 70 sec and a 30-mph speed of progression were used in recognition of lower traffic volumes during the off-peak period. Also, the 70-sec cycle length was needed for adequate pedestrian crossing time. The close proximity of adjacent signalized intersections, together with the shorter cycle length, resulted in a narrow green band width (Fig. G-162).

Stage 2 involved increasing the speeds of progression and shortening the cycle lengths. Owing to the design limitations of the interconnected signal system, only one common cycle length could be used. The change from a "before" 90-sec cycle length to the 70-, 100-, and 110-sec cycle lengths used for Stage 1 resulted in operating conditions at some locations that city officials could not tolerate on a continuing basis. Therefore, it was decided to return to the previous 90-sec cycle for all time periods.

Because the signal system establishes the offset relationships of the local controller by a timing dial set at a percentage of the cycle length, a change in cycle length can cause a change in the speed of progression. The changes in cycle length increased the speed of progression from 25 to 29.2 mph in the AM period and to 30.5 mph in the PM period. Conversely, the speed of progression in the off-peak period decreased from 30 to 23.5 mph. The latter change was observed to be inappropriate, and a revised offset plan was implemented to increase the speed of progression to 30 mph (Fig. G-163).

The offset plans (Figs. G-160, G-161, and G-163) were considered as Stage 2 of the experiment, with advance phases at Central Avenue, Orange Street, and State Street, as shown in Figure G-17.

The effects of the signal progression plans on Broad Street were determined by "before" and "after" comparisons of:

1. The number of vehicles stopped during the red signal phase.
2. The number of vehicles through during the following green signal phase.
3. The time required to travel the facility (trip time).
4. The time stopped during trips along this facility (delay time).
5. The number of stops occurring during each trip along the facility.

The actual number of vehicles stopped during the red signal phase and the number of vehicles through on the following green signal phase were counted manually for northbound and southbound movements on Broad Street at Central Avenue and at Orange Street after the Stage 1 (25-mph progression) improvement only. These counts were taken between the hours of 7:30 and 8:30 AM and 4:30 and 5:30 PM.

Speed and delay runs were made between Clay Street and Central Avenue, a distance of 3,180 ft (0.6 mile), between 7:00 and 9:00 AM, and 1:00 and 3:00 PM. Between 4:00 and 6:00 PM, runs were made only between Central Avenue and Orange Street, a distance of 1,550 ft (0.3 mile). A summary of all surveillance activity is given in Table G-134.

Analysis

The "after" measurements of Experiment B78 were used as "before" data for Experiment B100. Tables G-135 and G-136 summarize the results of the statistical analyses between the "before" and "after" measurements.

For the AM time period, a comparison of the conditions "before" and "after" the Stage 1 (25-mph progression) improvement indicated the following to be statistically significant:

1. Decreases for southbound traffic of 26.2 sec (16.5 percent) in total trip time, of 27.7 sec (51.4 percent) in delay time, and of 0.6 stop (33.3 percent) in the number of stops.
2. Decreases of 97.6 vehicles (9.7 percent) and 308.7 vehicles (64.6 percent) in the number of southbound vehicles stopped at the Orange Street and Central Avenue intersections, respectively.
3. Increases of 104.3 vehicles (5.4 percent) and 444.0 vehicles (25.0 percent) in the number of southbound vehicles through at the Orange Street and Central Avenue intersections, respectively.
4. Increases for northbound traffic of 21.4 sec (30.4 percent) in delay time and of 0.7 stop (30.4 percent) in the number of stops.
5. Decreases of 156.4 vehicles (32.5 percent) and 84.0 vehicles (49.1 percent) in the number of northbound vehicles stopped at the Orange Street intersection and the Park Place approach to the Central Avenue intersection, respectively.

Describing the improvements in terms of quality of flow for southbound traffic, the changes are reductions of 0.7 min per mile in trip time, 0.8 min per mile in delay time, and 1.0 stop per mile. In addition, average speed increased from 13.67 to 16.38 mph.

The more than 50-percent decrease in delay time for southbound traffic and the 25-percent increase in southbound vehicles through the Central Avenue intersection observed after the Stage 1 improvement reflect the elimination of the "bottleneck" at this location, which was experienced in Experiment B78. This change was primarily accomplished by the added time provided for the southbound advance phase.

Whereas some of the mean and percentage values increased to the detriment of northbound flow as much as others decreased to the improvement of the southbound flow, a comparison of these values weighted as to the relative numbers of vehicles affected reveals the measured improvements given in Table G-137. Using the total vehicle-hours of trip time for the 7:30 to 8:30 AM time period, as represented by mean times recorded for speed and delay runs and mean volumes observed at Orange Street, the increase of 8.72 veh-hr of trip time for north-

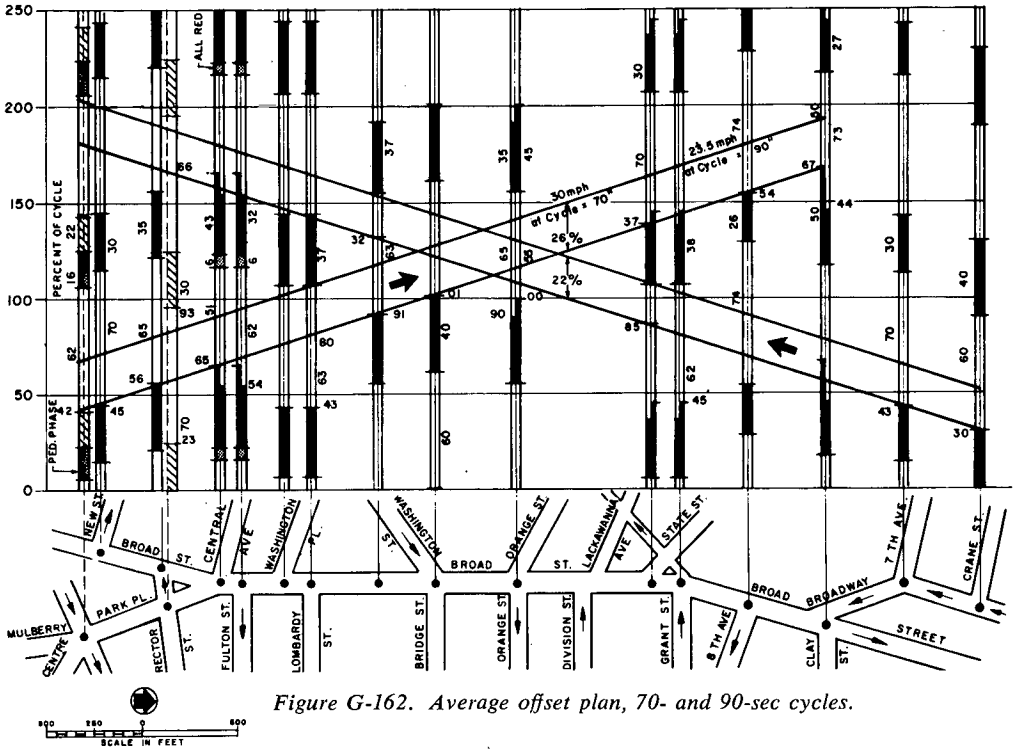


Figure G-162. Average offset plan, 70- and 90-sec cycles.

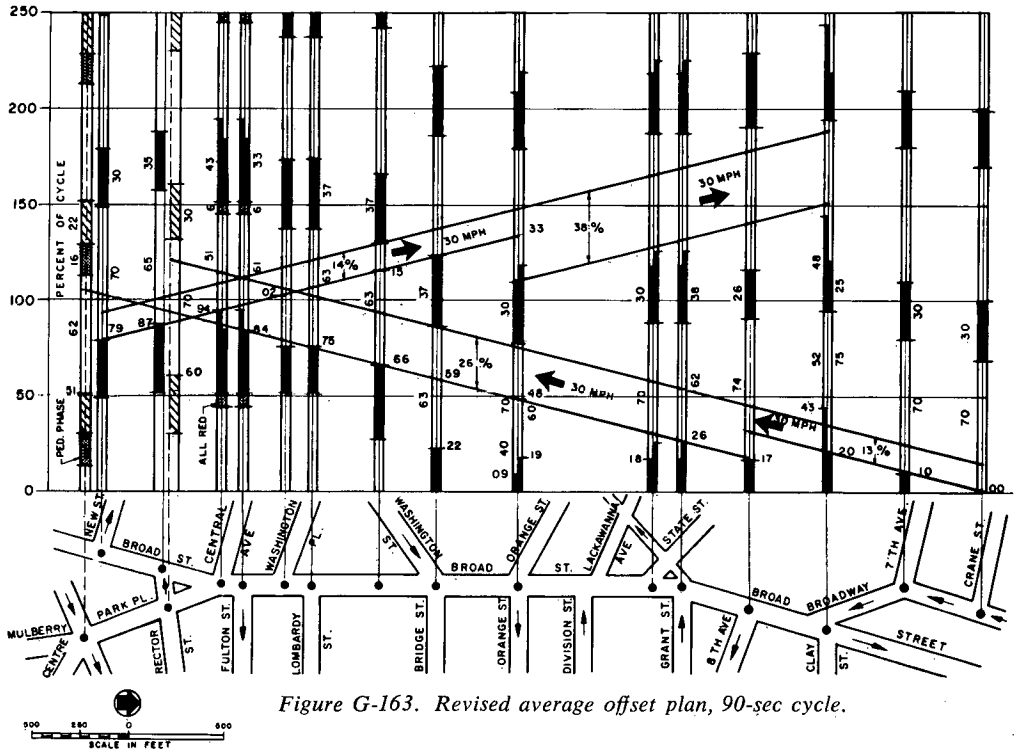


Figure G-163. Revised average offset plan, 90-sec cycle.

bound traffic does not offset the decrease of 10.34 veh-hr of trip time for southbound traffic. For each vehicle-hour of increase in trip time suffered by northbound traffic, a decrease of 1.2 veh-hr was realized by southbound traffic. In a similar manner, the increase of 7.33 veh-hr of delay time for northbound traffic and the decrease of 14.22 veh-hr of delay time for southbound traffic indicated that for each vehicle-hour increase in delay time suffered by northbound traffic, a decrease of 1.9 veh-hr was realized by southbound traffic.

TABLE G-134

SUMMARY OF SURVEILLANCE

"AFTER" CONDITION	SPEED AND DELAY RUNS: NO. OF RUNS NB	SB	DATES OF RUNS	MANUAL VOLUME SURVEYS: AT ORANGE ST. DAY 1	DAY 2	DAY 3	AT CENTRAL AVE. DAY 1	DAY 2	DAY 3
(*a*) AM time period [a]									
				Tues.	Thurs.	Fri.	Tues.	Thurs.	Fri.
25 mph (100-sec cycle)	25	24	6/16–6/20, incl.	6/17	6/19	6/20	6/17	6/19	6/20
29.2 mph (90-sec cycle)	24	25	7/8–7/11, incl.						
(*b*) Midday time period [b]									
30 mph (70-sec cycle)	11	13	6/17, 6/18						
(*c*) PM time period [c]									
				Mon.	Tues.	Wed.	Mon.	Tues.	Wed.
25 mph (110-sec cycle)	35	31	6/16–6/20, incl.	6/16 [d]	6/17	6/18	6/16	6/17	6/18

Note: All dates in 1969.
[a] 25-mph progression implemented 6/11; 29.2-mph progression implemented 6/23.
[b] 30-mph progression implemented 6/11; 23.5-mph progression—90-sec cycle implemented 6/23; 30-mph—90-sec cycle implemented 7/1.
[c] 25-mph progression implemented 6/11; 30.5-mph—90-sec cycle implemented 6/23.
[d] Signal malfunction—data not used.

TABLE G-135

SUMMARY OF ANALYSIS

CONDITION	NORTHBOUND TRAFFIC: SPEED AND DELAY: TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	VEHICLES PER HOUR: ORANGE STREET THROUGH	ORANGE STREET STOP	PARK PLACE THROUGH	PARK PLACE STOP	CENTRAL AVENUE THROUGH	CENTRAL AVENUE STOP
(*a*) AM time period									
"Before"	184.0	70.3	2.3	1026.7	480.7	224.7	171.0	634.0	179.3
"After" 25 mph	205.0	91.7	3.0	1074.7	324.3	238.8	87.0	692.3	202.0
Net change	+21.0	+21.4	+0.7	+48.0	−156.4	+13.6	−84.0	+58.3	+22.7
Percent change	+11.4	+30.4	+30.4	+4.7	−32.5	+6.1	−49.1	+9.2	+12.7
Sig. level	NS	0.05	0.05	NS	0.005	NS	0.005	NS	NS
"After" 30 mph	189.2	73.9	2.8						
Net change (25 to 30)	−15.8	−17.8	−0.2						
Percent change	−7.7	−19.4	−6.7						
Sig. level	NS	0.05	NS						
Net change ("before" to 30)	+5.2	+3.6	+0.5						
Percent change	+2.8	+5.1	+21.7						
Sig. level	NS	NS	0.05						
(*b*) Midday time period									
"Before"	196.0	68.8	2.7						
"After" 30 mph (70-sec cycle)	178.7	62.8	2.7						
Net change	−17.3	−6.0	0.0						
Percent change	−8.8	−8.7	0.0						
(*c*) PM time period									
"Before"	118.5	53.4	1.9	2753.0	1453.5	838.3		1244.3	287.3
"After" 25 mph	74.0	16.7	0.8	2911.0	782.5	888.3		1224.0	376.0
Net change	−44.5	−36.7	−1.1	+158.0	−671.0	+50.0		−20.3	+88.7
Percent change	−37.6	−68.7	−57.9	+5.7	−46.2	+6.0		−1.6	+30.9
Sig. level	0.0005	0.0005	0.005	0.05	0.005	NS		NS	0.005

NS = not significant at $\alpha = 0.10$.

TABLE G-136

SUMMARY OF ANALYSIS

CONDITION	SOUTHBOUND TRAFFIC: SPEED AND DELAY: TRIP TIME (SEC)	DELAY TIME (SEC)	NO. OF STOPS	VEHICLES PER HOUR: ORANGE STREET: THROUGH	ORANGE STREET: STOP	CENTRAL AVENUE: THROUGH	CENTRAL AVENUE: STOP
(*a*) AM time period							
"Before"	158.6	53.9	1.8	1946.7	1007.3	1777.7	478.0
"After"25 mph	132.4	26.2	1.2	2051.0	909.7	2221.7	169.3
Net change	−26.2	−27.7	−0.6	+104.3	−97.6	+444.0	−308.7
Percent change	−16.5	−51.4	−33.3	+5.4	−9.7	+25.0	−64.6
Sig. level	0.05	0.05	0.05	0.10		0.005	0.005
"After" 30 mph	125.4	23.5	0.9				
Net change (25 to 30)	−7.0	−2.7	−0.3				
Percent change	−5.3	−10.3	−25.0				
Sig. level	NS	NS	NS				
Net change ("before" to 30)	−33.2	−30.4	−0.9				
Percent change	−20.9	−56.4	−50.0				
Sig. level	0.05	0.05	0.05				
(*b*) Midday time period							
"Before"	176.6	48.0	2.1				
"After" 30 mph (70-sec cycle)	163.6	29.8	1.8				
Net change	−13.0	−18.2	−0.3				
Percent change	−7.4	−37.9	−14.3				
(*c*) PM time period							
"Before"	106.9	46.3	1.4	755.0	662.5	1229.5	443.0
"After" 25 mph	125.9	62.2	1.7	681.0	439.5	1119.0	449.0
Net change	+19.0	+15.9	+0.3	−74.0	−223.0	−110.5	+6.0
Percent change	+17.8	+34.3	+21.4	−9.8	−33.7	−9.0	+1.4
Sig. level	0.05	0.025	0.10	0.10	0.005	0.025	NS

NS = not significant at $\alpha = 0.10$.

TABLE G-137

COMPARISON OF VEHICLE-HOURS[a] FOR TRIP TIME AND DELAY TIME

CONDITION	NB (VEH-HR): TRIP TIME	NB (VEH-HR): DELAY TIME	SB (VEH-HR): TRIP TIME	SB (VEH-HR): DELAY TIME	BOTH (VEH-HR): TRIP TIME	BOTH (VEH-HR): DELAY TIME
(*a*) AM time period—Clay to Central						
"After" reversible lanes	52.48	20.05	85.76	29.15	138.24	49.20
"After" 25-mph	61.20	27.38	75.42	14.93	136.62	42.31
Net change	+8.72	+7.33	−10.34	−14.22	−1.62	−6.89
Percent change	+16.6	+36.5	−12.1	−48.8	−1.2	−14.0
Net change from B78 "before"	+21.53	+13.95	−55.51	−47.21	−33.97	−33.26
Percent change	+54.3	+103.8	−42.4	−76.0	−19.9	−44.0
(*b*) PM time period—Orange to Central						
"After" reversible lanes	90.62	40.84	22.42	9.71	113.04	50.55
"After" 25 mph	59.84	13.50	23.82	11.77	83.66	25.27
Net change	−30.78	−27.34	+1.40	+2.06	−29.38	−25.28
Percent change	−34.0	−66.9	+6.2	+21.2	−26.0	−50.0
Net change from B78 "before"	−31.40	−24.62	+6.80	+6.06	−24.59	−18.56
Percent change	−34.4	−64.6	+40.0	+106.0	−22.7	−42.3

[a] Volumes measured at Orange St.

It is important to note that 9 of the 24 speed and delay runs (37.5 percent) were made without stops in the "after" condition, whereas none of the 21 runs made during the "before" condition was made without stops. The average speed of the runs without stops was 21.17 mph.

Comparing the "after" Stage 1 (25 mph progression) conditions to the "after" Stage 2 (±30 mph progression) conditions, the analysis indicated the following to be statistically significant: a decrease of 17.8 sec (19.4 percent) in delay time for northbound traffic.

Although none of the factors for southbound traffic changed significantly, the trip time, delay time, and number of stops decreased.

The "before" conditions of this experiment ("after" B78) compared to the "after" Stage 2 (±30 mph progression) conditions indicate reductions of over 0.5 min each in trip time and delay time for southbound traffic, with a corresponding reduction of almost 1 stop for each vehicle. This improvement was realized with only a 0.5-stop-per-vehicle increase for each northbound vehicle.

Again, it is of interest to note that 9 of the 25 speed and delay runs (36.0 percent) were made without stops after the 30-mph progression was implemented. These runs had an average speed of 25.63 mph—an increase of 4.46 mph over similar runs with the 25-mph progression.

The "before" conditions of Experiment B78 compared to the "after" Stage 2 conditions indicate total reductions of 142.4 sec (53.2 percent) in trip time, 103.6 sec (81.5 percent) in delay time, and 3.2 stops (78.0 percent) per trip for southbound traffic. These improvements were realized with increases of 51.9 sec (37.8 percent) in trip time, 27.4 sec (58.9 percent) in delay time, and 1.2 stops (75.0 percent) per trip for northbound traffic.

For the PM time period, a comparison of the "before" and "after" Stage 1 conditions indicated the following to be statistically significant:

1. Decreases for northbound traffic of 44.5 sec (37.6 percent) in total trip time, of 36.7 sec (68.7 percent) in delay time, and of 1.1 stops (57.9 percent) in the number of stops.
2. A decrease of 671.0 vehicles (46.2 percent) in the number of northbound vehicles stopped at Orange Street.
3. An increase of 88.7 vehicles (30.9 percent) in the number of northbound vehicles stopped on Broad Street at Central Avenue.
4. An increase of 158.0 vehicles (5.7 percent) in the number of northbound vehicles through at Orange Street.
5. Increases for southbound traffic of 19.0 sec (17.8 percent) in total trip time, of 15.9 sec (34.3 percent) in delay time, and of 0.3 stop (21.4 percent) in the number of stops.
6. A decrease of 223.0 vehicles (33.7 percent) in the number of southbound vehicles stopped at Orange Street.
7. Decreases of 74.0 vehicles (9.8 percent) and 110.5 vehicles (9.0 percent) in the number of southbound vehicles through at the Orange Street and Central Avenue intersections, respectively.

Again describing the improvements in terms of quality of flow for northbound traffic, the changes are savings of 2.5 min per mile in trip time, 2.1 min per mile in delay time, and 3.8 stops per mile, with an increase from 8.92 to 14.28 mph in average speed.

Even with the increase of 158.0 northbound vehicles (5.7 percent) through the Orange Street intersection, Table G-10 indicates a large saving of 24.36 veh-hr in trip time, compared to the small increase of 1.74 veh-hr imposed on southbound traffic. For each vehicle-hour of trip time added to southbound traffic movement, 14.0 veh-hr were saved for northbound movement. In a similar manner, the increase of 2.29 veh-hr in delay time for southbound traffic and the decrease of 27.11 veh-hr in delay time for northbound traffic indicated that for each vehicle-hour increase in delay time suffered by southbound traffic, a decrease of 11.8 veh-hr was realized by northbound traffic.

Of added interest is the fact that 22 of the 35 speed and delay runs (62.9 percent) were made without stops in the "after" condition, compared to one of 27 (3.7 percent) made in the "before" condition. The average speed of the "after" runs was 21.31 mph, with the designed speed of progression being 25 mph.

During the midday time period, a number of speed and delay runs were made after the Stage 1 improvement (Table G-134). Tables G-135 and G-136 indicate decreases in trip time and delay time for both northbound and southbound traffic and a decrease in the number of stops for southbound traffic. Of interest is the fact that 7 of the 13 southbound runs were made without stopping between Orange Street and Central Avenue after the Stage 1 improvement, compared to none under the "before" conditions. The average speed of these runs was 19.72 mph, with the designed speed of progression being 30 mph, indicating that perhaps a lower speed of progression might have been more effective.

Conclusions

The 25-mph preferential progression generally resulted in more vehicles proceeding through the experimental area in less time and with less delay for the predominant flow. A saving of 0.7 min of trip time, 0.8 min of delay time, 1.0 stop per mile of travel, and an increase of from 13.67 to 16.38 mph were realized for southbound traffic in the AM time period. A saving of 2.5 min of trip time, 2.1 min of delay time, 3.8 stops per mile of travel, and an increase of from 8.92 to 14.28 mph in average speed were realized for northbound traffic in the PM time period. The effects of the 29.2-mph progression in the AM time period were not as pronounced as the change from the simultaneous offset to the 25-mph progression; but positive indications of a higher quality of flow were apparent, including additional reductions in trip time, delay time, and the number of stops.

Table G-137 compares the total changes in vehicle-hours of trip time and delay time resulting from all improvements made in both Experiment B78 and Experiment B100 (Stage 1). For the 7:30 to 8:30 AM period, these changes resulted in a net saving of almost 34 veh-hr of trip time and more than 33 veh-hr of delay time. For the 4:30 to 5:30 PM period, similar net savings of almost 25 and 19 veh-hr were realized for trip time and delay time, respectively. Expressed in terms of miles of travel, the dominant southbound flow in the AM period experienced over-all reductions

of 3.9 min per mile in trip time, 2.9 min per mile in delay time, and 5.3 stops for each mile of travel, and an increase in over-all average speed of from 8.1 to 17.29 mph. Similarly, the dominant northbound flow in the PM time period experienced over-all reductions of 2.6 min per mile in trip time, 1.9 min per mile in delay time, and 3.3 stops for each mile of travel, and an increase in over-all average speed of from 8.8 to 14.3 mph.

Oak Street Signal Progression—Experiment E40

Experiment E40 investigates the effects of alternate signal progression plans on traffic characteristics on Oak Street in Louisville (Fig. G-164). Oak Street is a one-way street in the fringe area of the CBD. It is part of the crosstown arterial system, providing for eastbound trips from residential areas to industrial and commercial districts.

Experimental Area

Oak Street is a one-way street, permitting travel in an eastbound direction east of Eighth Street, and is a two-way street west of Eighth Street. Table G-138 gives the parking restrictions during the AM and PM peak hours. Table G-139 gives the street widths of Oak Street between various intersections.

An on-ramp to I-65 for southbound vehicles is located on Oak Street between Floyd Street and Preston Street. Traffic volumes on the ramp are 550 vehicles during the AM peak hour and 400 vehicles during the PM peak hour. Peak traffic flows on Oak Street occur during the PM peak hour, with approximately 1,300 vph on Oak Street west of the I-65 on-ramp and approximately 900 vph east of the I-65 on-ramp. During the AM peak hour there are approximately 900 vehicles west of and 350 vehicles east of the I-65 on-ramp on Oak Street. Figures G-165 through G-167 show the 1965 average weekday traffic volumes for the AM, midday, and PM time periods.

Traffic signals on Oak Street are part of the Trafflex system south of Broadway. The local intersections are controlled by a master controller at the intersection of St. Catherine Street and Brook Street. The local controllers have two dials. Dial 1 operates between the hours of 6:00 PM and 2:30 PM; Dial 2 operates from 2:30 to 6:00 PM.

Design of Experiment

Analysis of speed and delay runs on Oak Street indicated large delays between many of the signalized intersections (Table G-140). As indicated, the period of greatest delay occurred during the PM peak hour, where average delay time was equal to 31.6 percent of the total trip time. The signal timing was recalculated for each intersection, using the volumes shown in Figures G-165 through G-167. The offset relationships were recalculated to provide for the maximum possible green band width, considering the delays given in Table G-140. The new settings were installed during the week of February 16, 1969. Subsequent speed and delay runs indicated that certain adjustments were required. These adjustments were installed during the week of April 27, 1969. Tables G-141 and G-142 give the signal phasing and offsets for each condition. A 60-sec cycle was in effect during this study. Figures G-168 through G-173 show the time-space diagrams for Dial 1 and Dial 2 of each condition.

The surveillance system for this experiment consisted of speed and delay runs and manual intersection counts. Speed and delay runs were taken during the following three time periods of the day: AM—7:30 to 9:00; midday—9:00 to 12:00 noon; and PM—3:30 to 5:30.

The vehicle counts consisted of counting the number of vehicles through per cycle and the number of vehicles stopped per cycle at all approaches to the intersections of: (1) Oak Street and First Street, (2) Oak Street and Brook Street, and (3) Oak Street and Preston Street. The vehicle

TABLE G-138

PARKING RESTRICTIONS ON OAK STREET

	PEAK-HOUR PARKING RESTRICTIONS		NO. OF TRAVEL LANES	
OAK ST. FROM	SOUTH SIDE	NORTH SIDE	AM	PM
Eighth to Seventh Sts.	No stopping 7:00 to 9:00 AM, except Saturdays and Sundays	None	3	2
Seventh to Third Sts.	No stopping 7:00 to 9:00 AM, 4:00 to 6:00 PM, except Saturdays and Sundays	None	3	3
Third and Second Sts.	No stopping at any time	No stopping at any time	4	4
Second to Brook Sts.	No stopping 7:00 to 9:00 AM, 4:00 to 6:00 PM, except Saturdays and Sundays	None	3	3
Brook to Shelby Sts.	No stopping 4:00 to 6:00 PM, except Saturdays and Sundays	None	2	3
Shelby to Logan Sts.	No stopping at any time	No stopping at any time	2	2

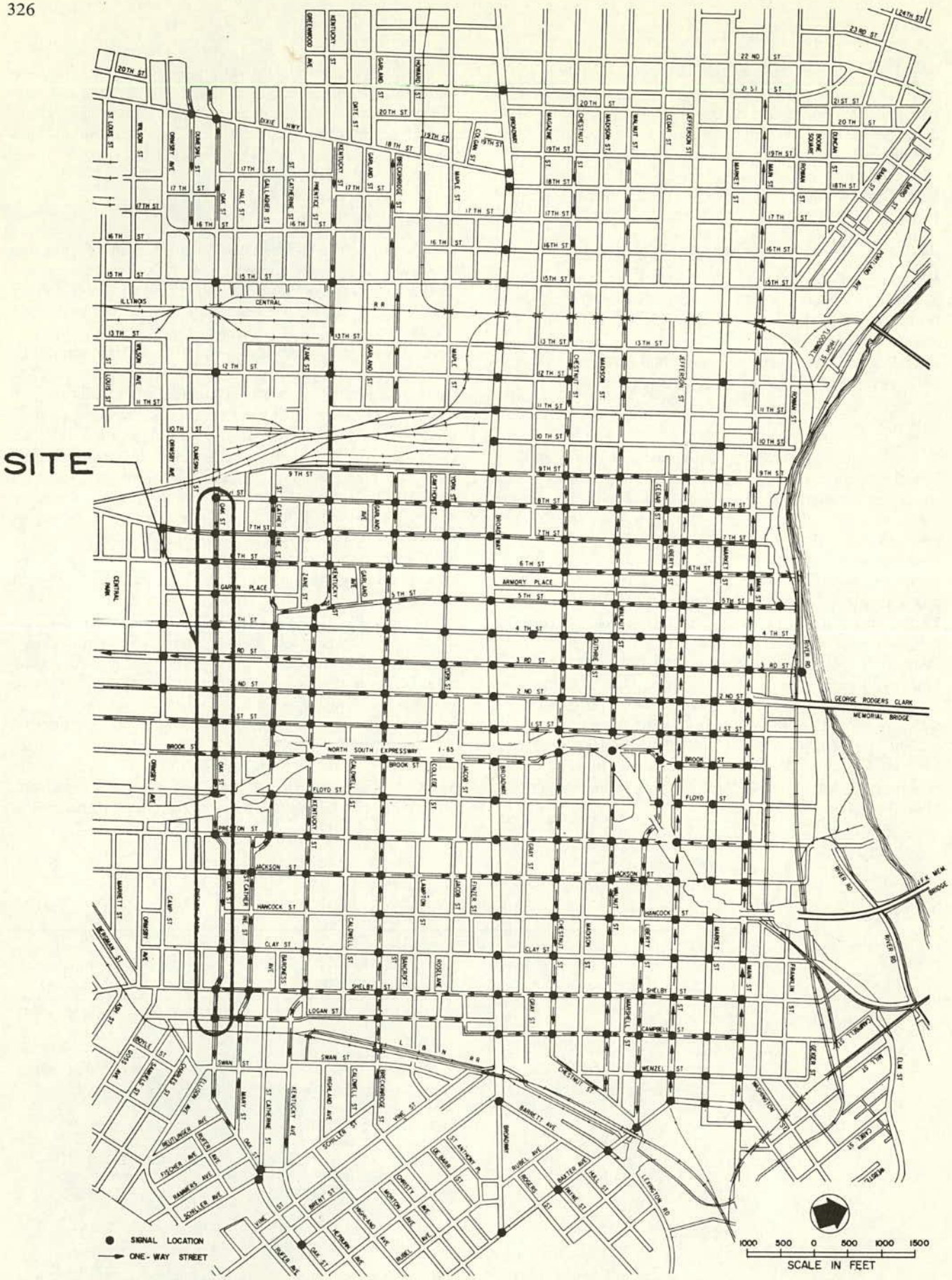


Figure G-164. Location map, Experiment E40.

TABLE G-139

STREET WIDTHS

BETWEEN	STREET WIDTH (FT)
Eighth St. and Sixth St.	42
Sixth St. and Third St.	36
Third St. and Preston St.	42
Preston St. and Shelby St.	36
Shelby St. and Logan St.	26

counts were made only before and after Phase 1 during the following time periods: AM—7:30 to 8:30; midday—1:30 to 2:30; and PM—4:30 to 5:30.

Dates of implementation and measurements are as follows:

CONDITION	DATES
Data group 1	1/29/69, 1/30/69, 1/31/69
Implementation of Phase 1	Week of 2/16/69
Data group 2	2/26/69, 2/27/69, 3/14/69
Implementation of Phase 2	Week of 4/27/69
Data group 3	5/7/69, 5/8/69, 5/9/69

Analysis

Analysis of the intersection counts during the morning and midday time periods indicated no significant differences between data groups for the number of vehicles through per cycle or the number of vehicles stopped per cycle.

TABLE G-140

AVERAGE DELAY TIME PER VEHICLE BETWEEN CHECKPOINTS ON OAK STREET

BETWEEN CHECKPOINTS	AVERAGE DELAY TIME PER VEHICLE (SEC)		
	AM	MIDDAY	PM
Eighth St.			
Seventh St.	0	0.8	0
Sixth St.	0	0	0.2
Garvin Place	0	0.2	2.8
Fourth St.	0	0.2	10.5
Third St.	0	0.8	13.8
Second St.	0.7	1.5	20.1
First St.	0.3	1.8	34.6
Brook St.	8.5	8.9	26.1
Floyd St.	0	0	0
Preston St.	6.1	5.8	11.9
Jackson St.	2.7	5.7	2.8
Clay St.	0	0.6	0
Shelby St.	0	0	4.3
Logan St.	0	0	0
Eighth St. to Logan St.	18.3	26.3	127.1
Average delay time as a percentage of total trip time from Eighth St. to Logan St.	7.3	9.7	31.6

Analysis of number of vehicles through per cycle during the PM period indicated no significant difference between data groups 1 and 2 (Table G-143). Significant changes were determined in the number of vehicles stopped per cycle (Table G-144). At the intersection of Oak Street with First Street, the number of vehicles stopped per cycle decreased on Oak Street and increased on First Street. This

TABLE G-141

GREEN TIME PLUS AMBER TIME ON OAK STREET

CROSS STREET	GREEN TIME PLUS AMBER TIME (SEC)					
	DIAL 1			DIAL 2		
	ORIGINAL	PHASE I	PHASE II	ORIGINAL	PHASE I	PHASE II
Eighth St.	38	36	36	42	38	38
Seventh St.	33	29	29	35	30	30
Sixth St.	33	24	24	35	30	33
Garvin Place	41	37	37	42	35	35
Fourth St.	35	28	34	32	27	33
Third St.	30	26	31	30	31	33
Second St.	32	28	28	30	35	35
First St.	30	31	31	30	36	33
Brook St.	26	29	29	30	33	33
Floyd St.	38	33	33	38	35	35
Preston St.	33	28	32	33	33	33
Jackson St.	32	34	34	30	31	31
Clay St.	38	37	37	38	36	36
Shelby St.	31	30	30	31	30	30
Logan St.	33	27	27	33	31	31

TABLE G-142

SYSTEM OFFSETS ON OAK STREET

CROSS STREET	SYSTEM OFFSETS (%)					
	DIAL 1			DIAL 2		
	ORIGINAL	PHASE I	PHASE II	ORIGINAL	PHASE I	PHASE II
Eighth St.	68	82	82	68	85	85
Seventh St.	93	4	4	93	5	5
Sixth St.	13	30	30	13	25	25
Garvin Place	46	49	49	40	46	46
Fourth St.	68	75	68	63	74	68
Third St.	93	93	93	93	95	95
Second St.	21	21	21	20	16	16
First St.	48	42	42	43	40	40
Brook St.	69	62	62	68	59	59
Floyd St.	86	87	87	86	83	83
Preston St.	93	3	3	93	0	5
Jackson St.	13	22	22	14	25	28
Clay St.	66	67	67	66	68	68
Shelby St.	82	90	90	81	95	95
Logan St.	10	13	13	10	10	10

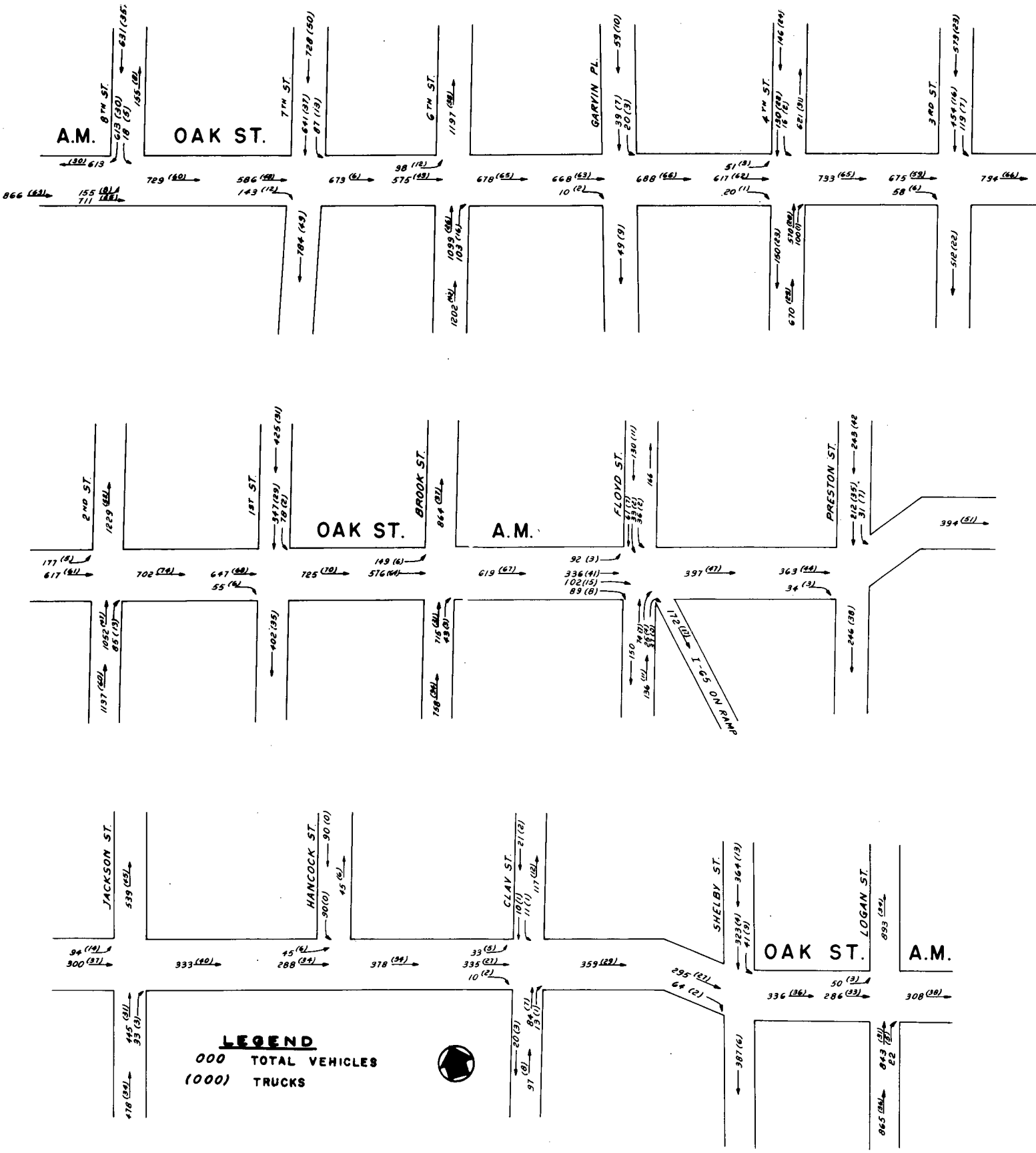


Figure G-165. 1968 AWDT, AM peak hour.

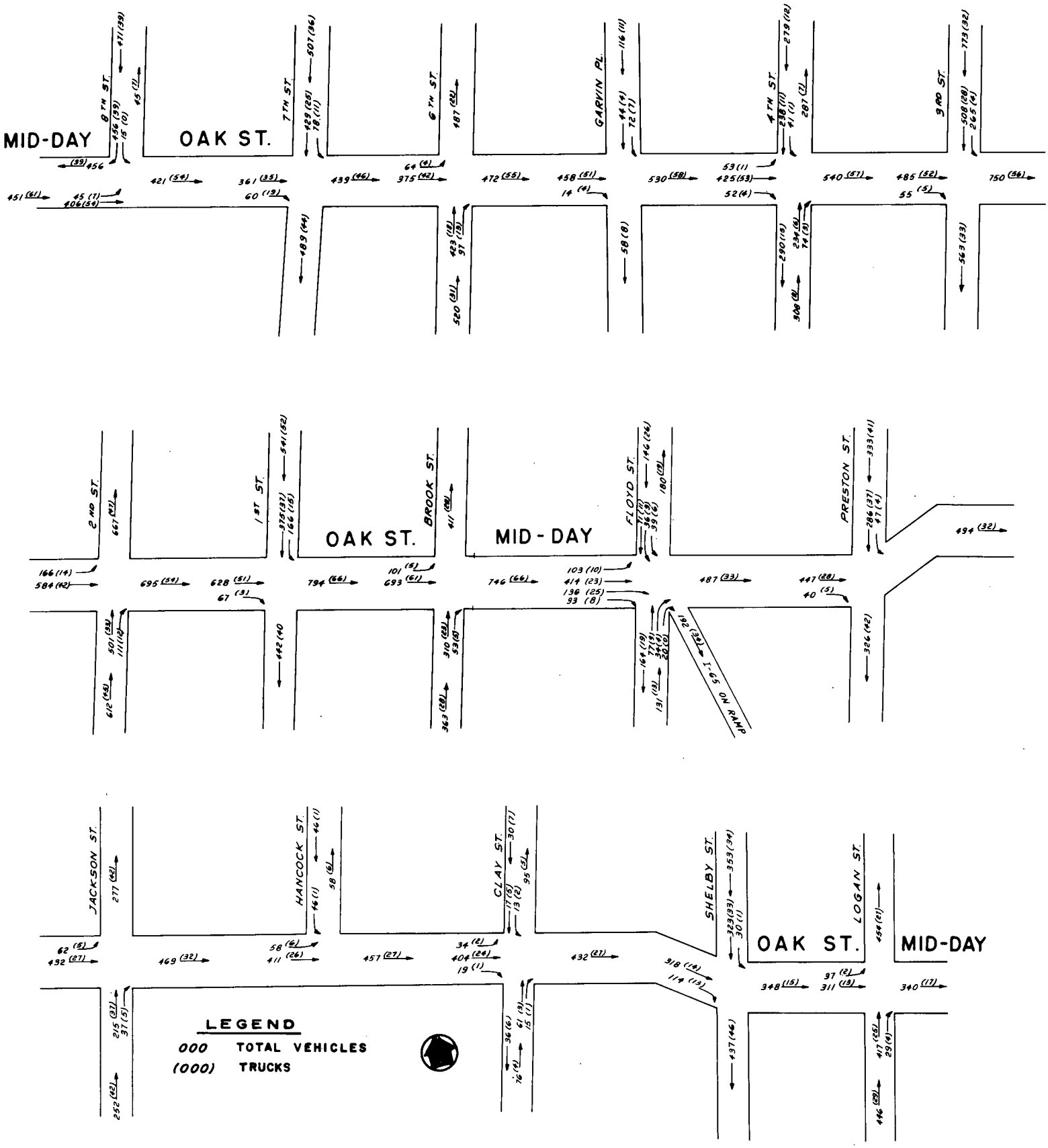


Figure G-166. 1968 AWDT, midday hour.

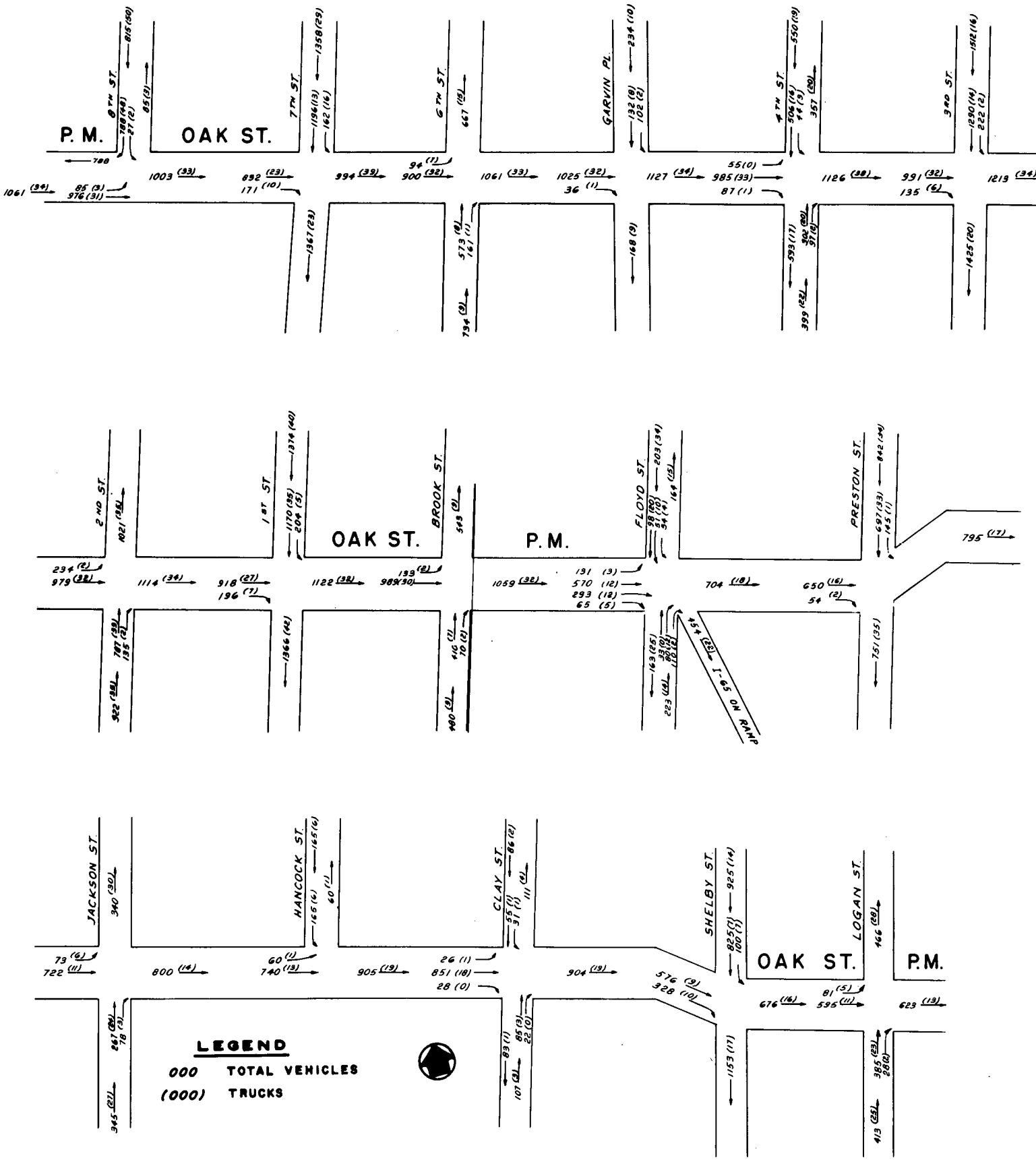


Figure G-167. 1968 AWDT, PM *peak hour.*

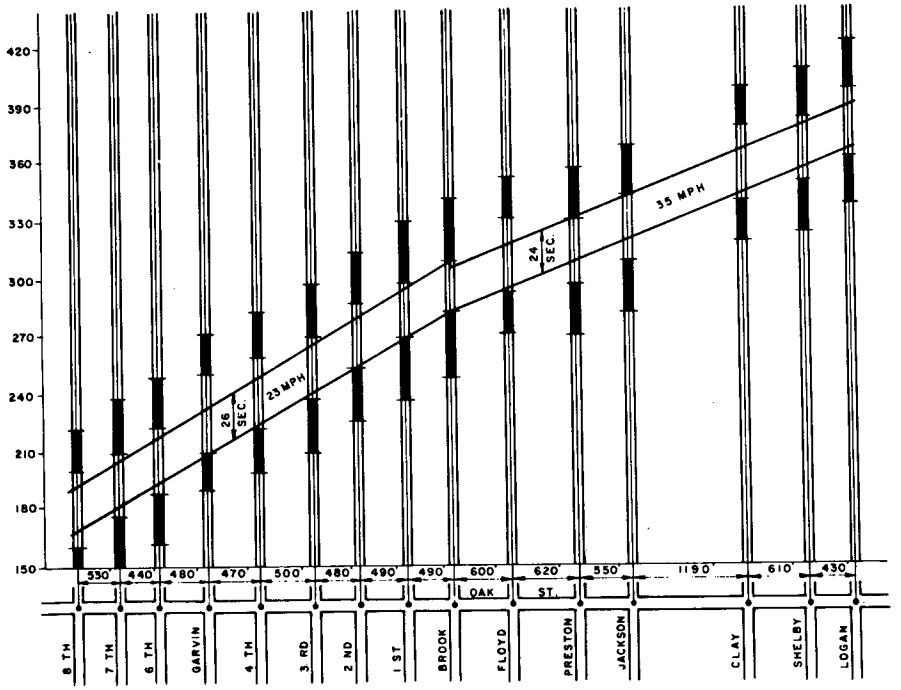


Figure G-168. Original settings, Dial 1.

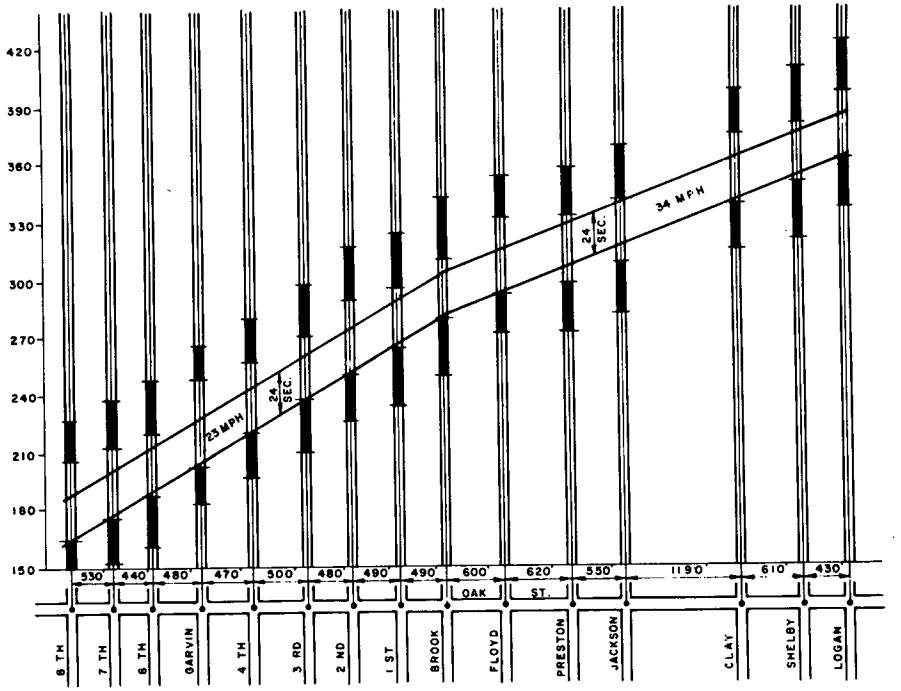


Figure G-169. Original settings, Dial 2.

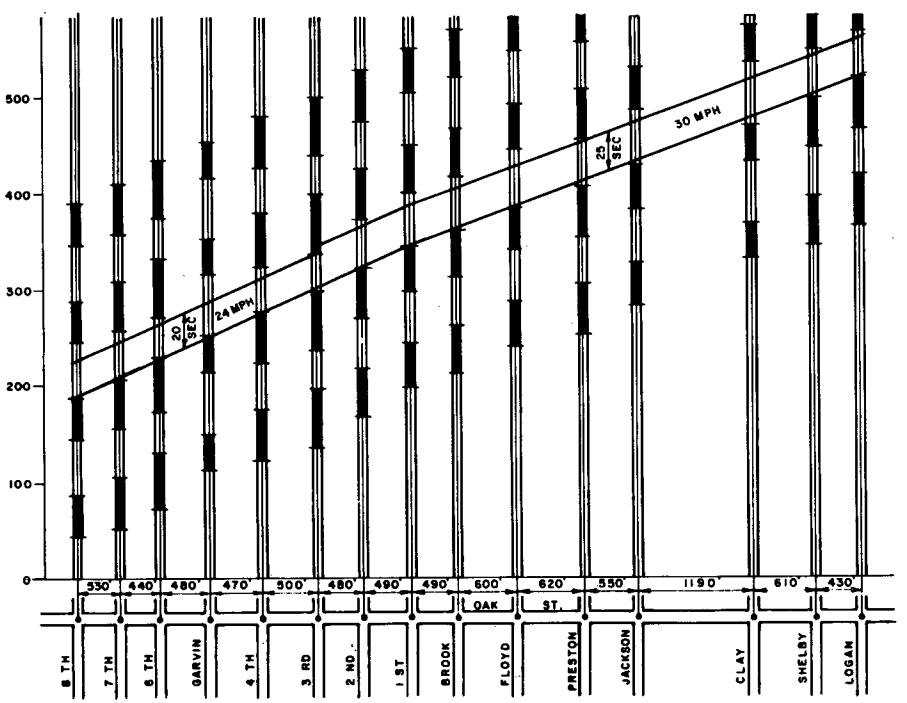


Figure G-170. Phase I settings, Dial 1.

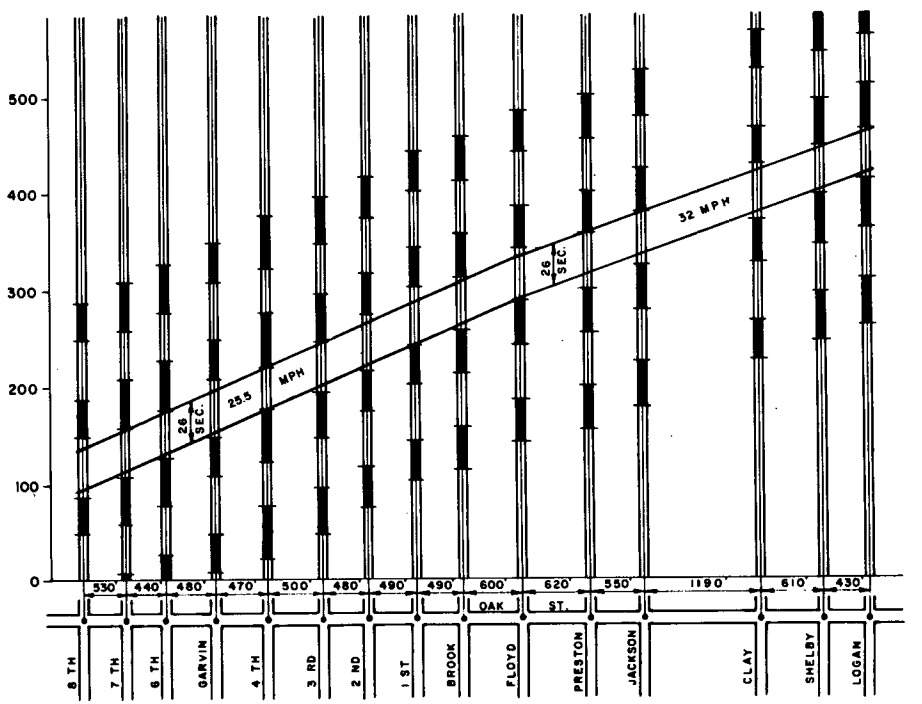


Figure G-171. Phase I settings, Dial 2.

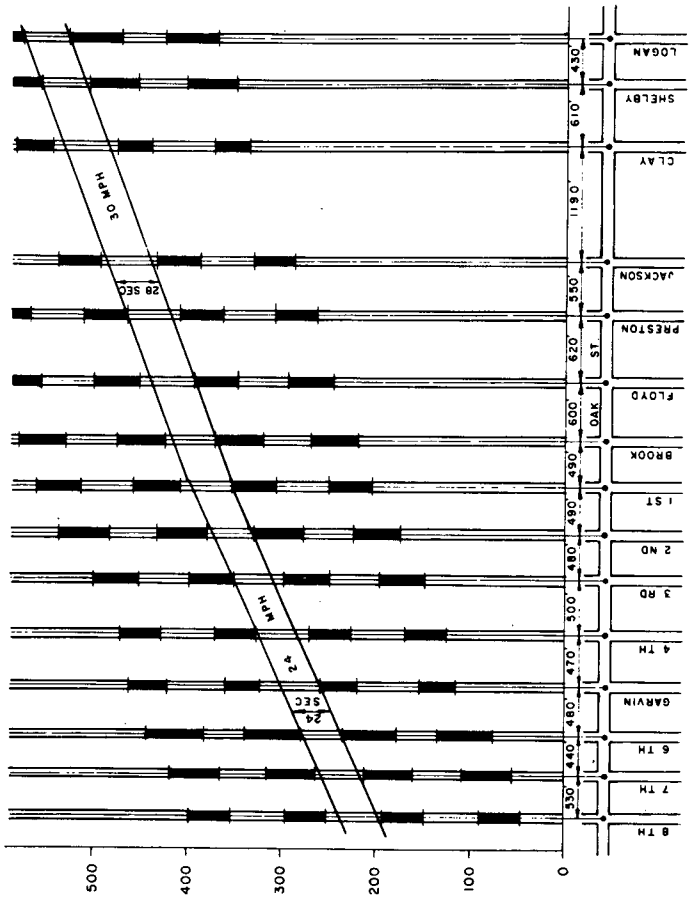


Figure G-172. Phase II settings, Dial 1.

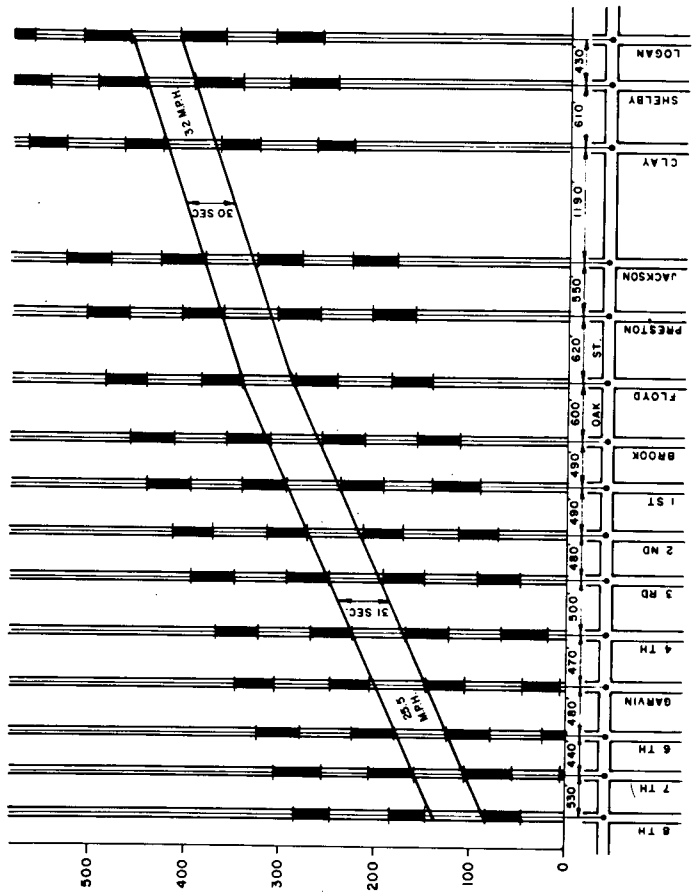


Figure G-173. Phase II settings, Dial 2.

TABLE G-143

NUMBER OF VEHICLES THROUGH PER CYCLE, PM PERIOD—MEAN VALUES

INTERSECTION OF OAK ST. WITH	APPROACH	MEAN VALUE (VEH) "BEFORE"	"AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$
First St.	Oak St.	24.5	24.8	0.3(1.2%)	No
	First St.	22.8	22.8	0.0(0.0%)	No
Brook St.	Oak St.	25.3	25.2	−0.1(−0.4%)	No
	Brook St.	7.6	7.8	0.2(2.6%)	No
Preston St.	Oak St.	17.4	17.1	−0.3(−1.7%)	No
	Preston St.	15.1	14.7	−0.4(−2.6%)	No

TABLE G-144

NUMBER OF VEHICLES STOPPED PER CYCLE, PM PERIOD—MEAN VALUES

INTERSECTION OF OAK ST. WITH	APPROACH	MEAN VALUE (VEH) "BEFORE"	"AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$
First St.	Oak St.	8.0	3.9	−4.1(−51.3%)	Yes
	First St.	13.4	20.5	7.1(53.0%)	Yes
Brook St.	Oak St.	5.7	7.1	1.4(24.6%)	Yes
	Brook St.	1.4	1.6	0.2(14.3%)	No
Preston St.	Oak St.	9.4	4.3	−5.1(−54.3%)	Yes
	Preston St.	6.8	3.3	−3.5(−51.5%)	Yes

TABLE G-145

NUMBER OF VEHICLES STOPPED PER CYCLE, PM PERIOD—VARIANCES

INTERSECTION OF OAK ST. WITH	APPROACH	VARIANCE (VEH) "BEFORE"	VARIANCE (VEH) "AFTER"	NO. OF OBS. "BEFORE"	NO. OF OBS. "AFTER"	F RATIO
First St.	Oak St.	19.7	8.3	180	180	2.37 [a]
	First St.	46.4	86.6	179	175	1.87 [a]
Brook St.	Oak St.	8.6	9.5	180	180	1.10
	Brook St.	2.0	2.1	179	178	1.05
Preston St.	Oak St.	11.6	6.4	179	180	1.81 [a]
	Preston St.	10.7	8.1	162	179	1.32 [a]

[a] Significant at the $\alpha = 0.05$ level.

result could have been expected, because the signal phasing was changed to give 6 sec of green time from First Street to Oak Street (Table G-141). The change in variance (Table G-145) also showed a similar result. The number of vehicles stopped per cycle at the Oak Street approach to its intersection with Brook Street was significantly increased (Table G-144). This increase was not expected, as more green time was given to the Oak Street approach; however, the offset relationship was changed from 68 percent in the original setting to 59 percent for the Phase 1 setting. Both approaches to the Oak Street-Preston Street intersection indicated a significant reduction in the mean values and variances of the number of vehicles stopped per cycle.

Information obtained from the speed and delay run data included speed, delay time, number of stops, and delay time per stop. These data were analyzed by use of the ANOVA technique. Tables G-146 and G-147 give the mean values and variances for the speed and delay run data. Tukey's limits for multiple comparisons were calculated at the 5-percent level and were drawn around the mean values (Fig. G-174). As indicated, the only significant changes were:

TABLE G-146

SUMMARY OF SPEED AND DELAY ANALYSIS, MEAN VALUES

TIME PERIOD	DATA GROUP	SPEED (MPH)	DELAY TIME (SEC)	NO. OF STOPS	DELAY TIME PER STOP (SEC)
AM	1	20.2	23.3	1.5	17.0
	2	21.0	21.6	0.9	25.8
	3	20.7	24.8	1.0	24.4
Midday	1	20.7	22.6	1.2	21.2
	2	21.7	16.5	0.7	24.2
	3	20.5	24.3	1.0	25.8
PM	1	16.1	69.9	2.6	26.5
	2	15.6	72.2	2.8	26.6
	3	17.2	56.0	1.8	29.1

1. A reduction in speed for the midday time period between data groups 2 and 3.
2. An increase in delay time per stop between data groups 1 and 2 and data groups 1 and 3.

TABLE G-147

SUMMARY OF SPEED AND DELAY ANALYSIS, VARIANCES

TIME PERIOD	DATA GROUP	SPEED (MPH)	DELAY TIME (SEC)	NO. OF STOPS	DELAY TIME PER STOP (SEC)	NO. OF OBS. CATEGORY "A"	NO. OF OBS. CATEGORY "B"
AM	1	2.2	127.8	0.9	51.0	11	10
	2	8.2	260.8	0.6	47.8	18	13
	3	10.7	453.6	0.6	72.8	24	18
Midday	1	6.2	302.5	1.0	64.9	16	12
	2	7.0	254.9	0.6	52.6	19	11
	3	8.0	314.7	0.5	14.7	20	15
PM	1	7.4	1,961.0	1.5	72.8	12	12
	2	7.8	1,624.5	2.7	21.6	17	17
	3	11.8	1,343.4	1.1	11.2	19	18

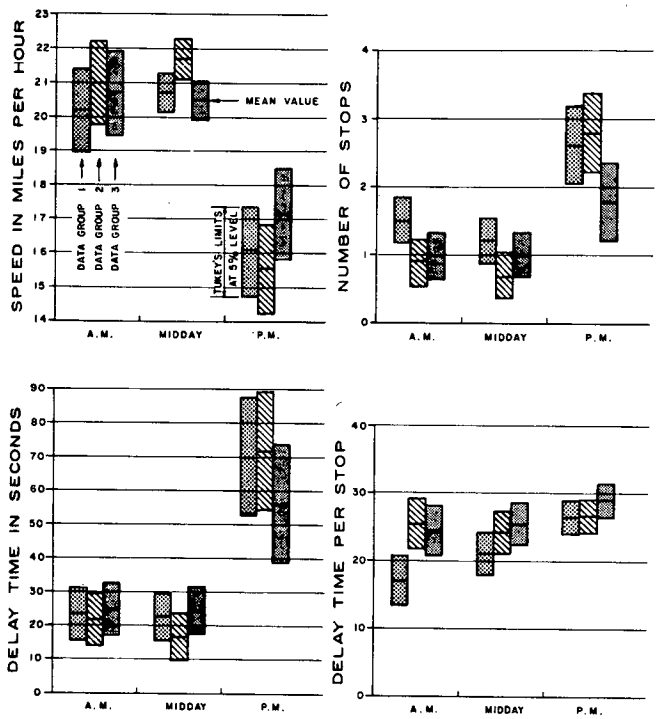


Figure G-174. Analysis of speed and delay run data.

The delay time per vehicle was analyzed further by investigating the delay time between checkpoints from the speed and delay run data. Table G-148 gives the average delay time per vehicle in seconds between checkpoints for each data group. Analysis of this table reinforces the findings of the vehicle counts. During the PM time period at First Street, where the number of vehicles stopped per cycle was reduced by 51.3 percent, the average delay time per vehicle was reduced from 12.5 to 5.8 sec (53.6 percent). At the intersection of Oak Street and Preston Street, where the number of vehicles stopped per cycle was reduced by 51.5 percent on the Oak Street approach, a corresponding reduction in delay time is given in Table G-148. Similarly, at the Oak Street approach to its intersection with Brook Street, the number of vehicles stopped per cycle increased slightly, whereas the average delay time per vehicle increased slightly.

Conclusions

Based on the foregoing analysis, it is concluded that:

1. The number of vehicles through per cycle did not change significantly at any of the three intersections.
2. The number of vehicles stopped per cycle was significantly reduced on Oak Street at two of the three intersections monitored. The variability of the number of vehicles stopped per cycle was also reduced for the same two approaches.
3. From speed and delay run data, generally the variables of speed, delay time, number of stops, and delay time per stop did not change significantly between data groups, except for a reduction in speed during the midday time period between data groups 2 and 3 and increases in delay time per stop during the AM time period between data groups 1 and 2 and data groups 1 and 3.
4. The reduction in vehicles stopped per cycle on Oak Street at First Street and at Preston Street resulted in corresponding reductions in the delay time per vehicle.

Network Signal Coordination—Experiment E35

Experiment E35 evaluates the effects of coordinating all traffic signal controllers in downtown Louisville under one common control. These effects were evaluated by speed and delay run data taken on many of the downtown streets.

Experimental Area

The 281 traffic-signal-controlled intersections (Fig. G-175) can be divided into four systems, as follows:

TABLE G-148

AVERAGE DELAY TIME PER VEHICLE BETWEEN CHECKPOINTS

		AVERAGE DELAY TIME (SEC), BY CROSS STREET															
TIME PERIOD	DATA GROUP	SEVENTH	SIXTH	GARVIN	FOURTH	THIRD	SECOND	FIRST	BROOK	FLOYD	PRESTON	JACKSON	CLAY	SHELBY	LOGAN	TOTAL DELAY TIME (SEC)
AM	1					0.7				7.9	9.2	5.5				23.3
	2	0.3			1.6	0.2		6.8	1.6	4.7	3.8		1.5	1.0		21.5
	3	3.3			1.2	0.2	1.0	3.0		3.3	9.3		2.3	1.3		24.9
Midday	1		0.4					4.5		8.9	4.6	2.7	1.3			22.4
	2			1.3	1.7		0.3	1.5		6.7	1.5		3.6			16.6
	3				1.8	1.5	1.5	4.1	2.4	4.3	6.0		2.7			24.3
PM	1		0.3	3.0	15.0	5.8	12.5	8.3	2.3	14.7	5.3		2.8			70.0
	2	6.5	3.9	19.4	9.8	1.4	5.8	10.2		9.1	3.4	1.2	1.5			72.2
	3		2.5	6.7	8.6	5.4	6.4	8.8		4.2	8.8	1.4	3.2			56.0

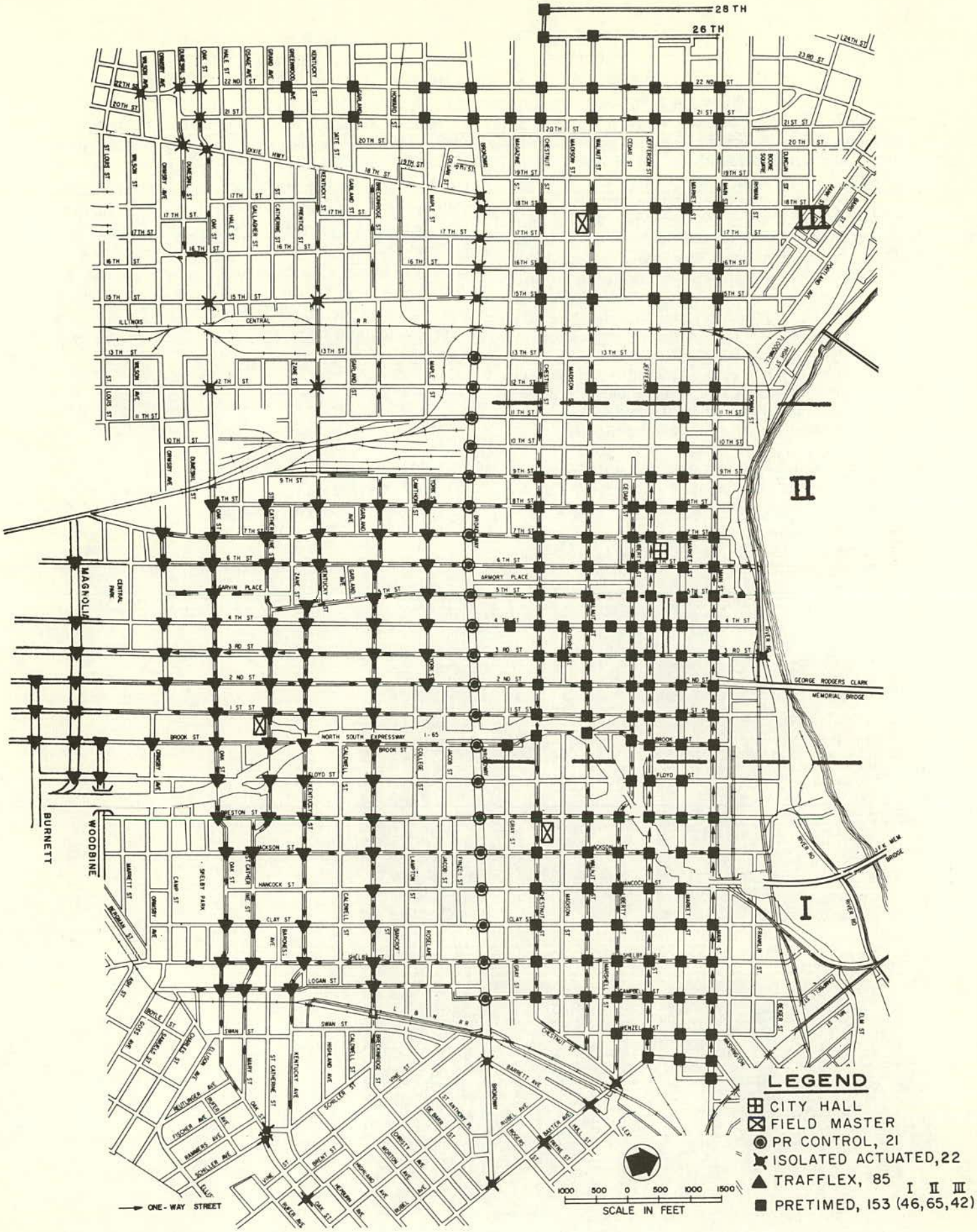


Figure G-175. Location map, Experiment E35.

1. PR system—21 local controllers are located on Broadway between Campbell Street and 13th Street. These are coordinated by a master controller located in City Hall and operated on a 90-sec cycle with an average offset plan. The location and separate operation of this system disrupt north-south progressive traffic movement.

2. Trafflex system—85 local Trafflex controllers are located in the area bounded by York Street on the north, Logan Street on the east, Burnett Street on the south, and Eighth Street on the west. These are coordinated by a master controller located at the intersection of Brook Street with St. Catherine Street. The master controller electronically establishes the cycle length and offset plan for each of the local units. The local controllers are equipped with two dials, each operating on a 60-sec cycle.

3. Synchronous system—153 pretimed two-dial controllers are located in the areas north and south of Broadway (Fig. G-175). This system is divided into three subsystems, each controlled by a master controller in the field. These masters are controlled by a supermaster located in City Hall. The local units operate one dial on a 60-sec cycle.

4. Isolated actuated controllers—22 isolated actuated controllers are located within the study area (Fig. G-175).

Each of these four systems operates independent of the others without coordination. Some of the local units are shown in Figure G-176.

Design of Experiment

Observation of vehicle flows indicated that a potential improvement could be gained by integrating all the systems under one control. It was first thought that the coordination could be done with one common cycle length. Discussions with personnel of the Department of Traffic Engineering indicated that a short cycle length would not be sufficient to accommodate pedestrian movements and traffic volumes on Broadway, and a long cycle length could not be tolerated in the areas north and south of Broadway. Therefore, it was decided to use a common cycle length for the areas north and south of Broadway and to use double this cycle length for Broadway, as follows:

TIME	AREA	CYCLE LENGTH (SEC)
6:00-2:30 PM	North of Broadway	50
	Broadway	100
	South of Broadway	50
2:30-6:00 PM	North of Broadway	60
	Broadway	120
	South of Broadway	60

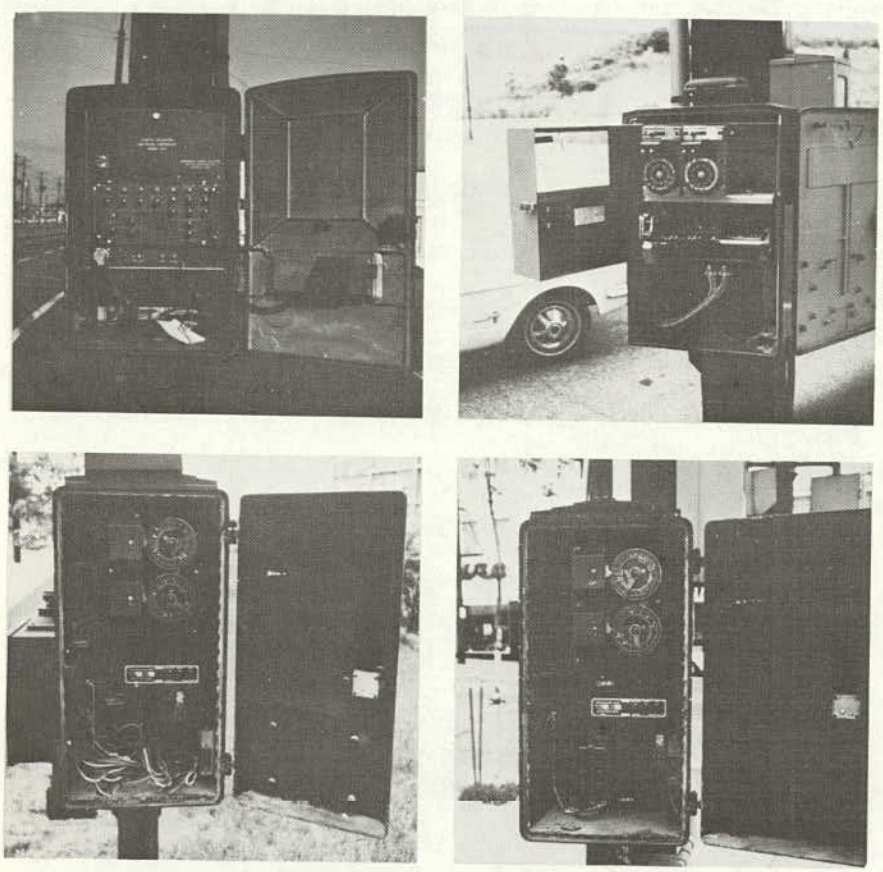


Figure G-176. Local controllers.

The physical method of interconnecting the various systems was based on using the PR master controller in City Hall to coordinate all systems. A slave PR master, also in City Hall, received a coordination pulse from the PR master and transmitted another coordination pulse to all local controllers north of Broadway in areas I and II (Fig. G-175). Area III was not included in the experimentation. The local controllers were set to run on a 45-sec cycle plus a 5-sec dwell per cycle between the hours of 6:00 PM and 2:30 PM, and on a 55-sec cycle plus a 5-sec dwell per cycle between the hours of 2:30 and 6:00 PM. The 5-sec dwell was necessary to compensate for any variation in the coordination pulse from the PR master controller. No change was made to the interconnection between the PR master controller in City Hall and the PR local controllers on Broadway. A slave PR controller was installed at the intersection of Broadway and Preston Street. It was used to coordinate the Trafflex system south of Broadway with the PR master in City Hall. The Trafflex system operated with its normal 10-percent dwell. The traffic-actuated controllers were not interconnected with the other systems.

The development of time-space diagrams to control traffic during two time periods of the day involved an immense amount of data collection. Information was gathered on physical dimensions, parking characteristics, and traffic volume counts. Traffic counts were conducted at alternate intersections in the area bounded by Main Street on the north, Baxter Avenue on the east, Oak Street on the south, and Ninth Street on the west. These were adjusted to represent an average day in 1968.

Time-space diagrams were developed in three steps. Step 1 involved the use of SIGOP.* Step 2 involved displaying the SIGOP output on a three-dimensional time-space model (see Appendix D) and making manual adjustments. Step 3 consisted of plotting the information from the three-dimensional time-space model on graph paper and fine-tuning the system.

Because the SIGOP's capacity was limited to 150 intersections, it was not possible to make one run for the area bounded by Main Street on the north, Baxter Avenue on the east, Oak Street on the south, and Ninth Street on the west. It became necessary to divide this area into two subareas. The first subarea was bounded by Main Street on the north, Baxter Avenue on the east, Breckinridge Street on the south, and Ninth Street on the west. The second subarea was bounded by Broadway on the north, Logan Street on the east, Oak Street on the south, and Ninth Street on the west. The offsets and splits for common intersections determined for the SIGOP run of the first area were used as input for the SIGOP run of the second area. Specifically these included the intersections on Broadway, York Street, and Breckingridge Street. The macro input used for the SIGOP runs is given in Table G-149. The variables are explained in the following manuals:

* *Traffic Signal Optimization Program*, prepared for the Bureau of Public Roads by Peat, Marwick, Livingston & Co.

TABLE G-149

SIGOP MACRO INPUT

VARIABLE	VALUE	DIMENSION
ALANE	0	Lanes
ALPHA	0.75	—
AMBER	3.0	Seconds
ARMIN	4.6	Seconds
CRIMP	0	—
CYCLE	60	Seconds
CYCLE	120	Seconds
CYCLE	50	Seconds
CYCLE	100	Seconds
DISCH	2.25	Seconds
DLANE	0	Lanes
FTRDN	0.10	Vehicles
FTRUK	2.0	Vehicles
FTRUP	0.12	Vehicles
GREMI	5	Seconds
"NODES"	149	Intersections
"NODES"	89	Intersections
NUQUIT	500	Errors
NUFAZ	2	Phases
OTP	2	Time periods
SPEED	25	Mph
STIMP	0.5	—
VOLYM	30,000	Vph

TABLE G-150

ALPHA VALUES

STREET	ALPHA VALUE
Baxter Ave.	1.00
Breckinridge St.	1.00
Broadway	2.00
Brook St.	0.75
Campbell St.	1.00
Chestnut St.	1.00
Clay St.	0.75
Floyd St.	0.75
Hancock St.	0.75
Jackson St.	1.00
Jefferson St.	1.00
Kentucky St.	1.00
Liberty St.	1.00
Logan St.	1.00
Main St.	1.00
Market St.	1.00
Oak St.	1.00
Preston St.	1.00
St. Catherine St.	1.00
Shelby St.	1.00
Walnut St.	1.00
Wenzel St.	0.75
York St.	0.75
Numbered Streets:	
First St.	0.75
Second St.	1.00
Third St.	1.00
Fourth St.:	
North of Broadway	0.50
South of Broadway	0.75
Fifth St.	0.75
Sixth St.	1.00
Seventh St.	1.00
Eighth St.	0.75
Ninth St.	0.75

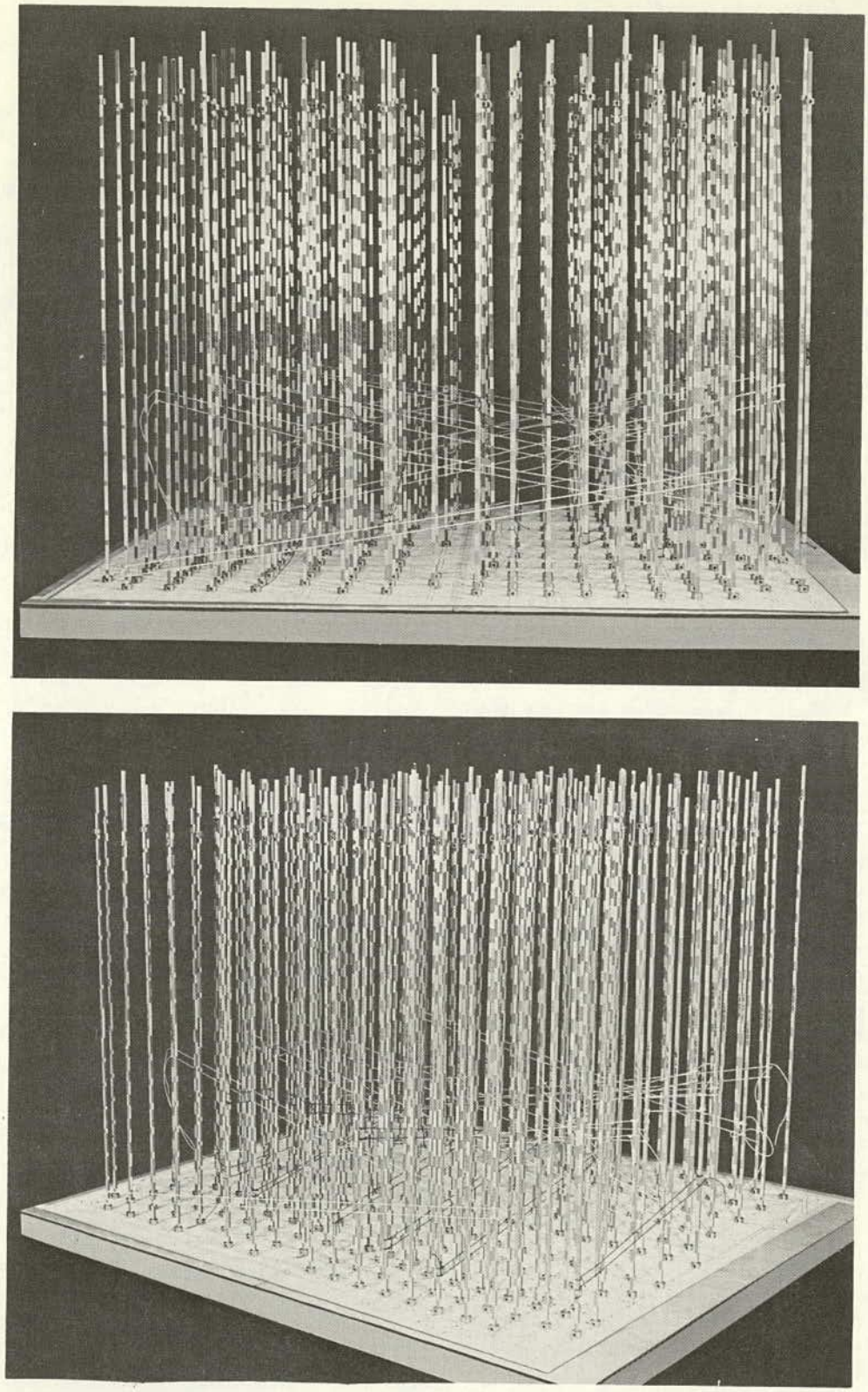


Figure G-177. Three-dimensional time-space model.

1. *SIGOP: Traffic Signal Optimization Program.* Peat, Marwick, Livingston & Co. (Sept. 1966).

2. *SIGOP: Traffic Signal Optimization Program Users' Manual.* Peat, Marwick, Livingston & Co. (Dec. 1968).

3. *SIGOP: Traffic Signal Optimization Program Field Test and Sensitivity Studies.* Peat, Marwick, Livingston & Co. (Dec. 1968).

The alpha values, which express the value of each individual link, are given in Table G-150.

The splits and offsets determined from the SIGOP output were displayed on a specially constructed three-dimensional time-space model (Fig. G-177). Basically the model consisted of aluminum alloy rods fitted into holes in a wooden base. Elastic thread was used to show the green

band width from one intersection to the next; pressure-sensitive tape was used to indicate the red intervals at each intersection. Figure G-178 shows the publicity that was received by the use of this model in the City Hall of Louisville.

The last step in the determination of splits and offsets was the task of plotting on graph paper the splits and offsets determined from the three-dimensional time-space model. Fine-tuning and minor adjustments were made that could not be readily seen on the three-dimensional model. The offsets and splits in the "before" and "after" conditions are given in Table G-151.

The new phasings and offsets were installed on the following dates (1969):

DATE	WORK ACCOMPLISHED
May 24	Dial 2 south of Broadway installed and turned on.
June 8	Dial 1 south of Broadway and Dials 1 and 2 on Broadway installed.
Week of June 8	Testing and correction of Dial 1 south of Broadway and Dials 1 and 2 on Broadway.
June 24 and 25	Dial 2 north of Broadway installed.
June 30	Dial 2 north of Broadway turned on and adjusted.
July 2 and 3	Dial 1 north of Broadway installed.
Week of July 6	Dial 1 north of Broadway turned on and adjusted. North of Broadway, south of Broadway, and Broadway interconnected.

Because some of the equipment had not been used for many years, considerable difficulty was experienced due to equipment failures. The worst problems occurred in the area north of Broadway. Because only one dial had been in operation for several years, many relays continuously failed in switching control from one dial to another. The excellent work by the maintenance forces of the Louisville and Jefferson County Department of Traffic Engineering soon solved and corrected the problems.

The surveillance system for this experiment was limited to speed and delay runs on many of the north-south and east-west streets in the downtown area (Table G-152). Data were gathered during the following time periods: AM—7:30 to 9:30 AM; midday—9:30 AM to 3:30 PM; and PM—3:30 to 5:30 PM. The speed and delay runs were made between June 11, 1968, and May 20, 1969, for the "before" condition, and between July 8, 1969, and July 24, 1969, for the "after" condition.

Analysis

To simplify the analysis of data, the streets given in Table G-152 were grouped into four subareas: (1) east-west streets north of Broadway, (2) Broadway, (3) east-west streets south of Broadway, and (4) north-south streets.

The variables that were analyzed from delay run data included speed, delay time, number of stops, delay time per stop, and stops per mile. Mean values and differences between the "before" and "after" conditions for individual streets are summarized in Tables G-153 through G-156. Table G-157 gives the mean values for each of the four areas, as well as average values for all streets included in the study. Tables G-158 through G-162 summarize the variances that correspond to the aforementioned mean values.

The analysis of variance technique and t tests were used to determine the differences between the mean values of the "before" and "after" conditions. Two levels of significance were determined, one at $\alpha \leq 0.05$ and another at $0.10 \geq \alpha > 0.05$. Differences in variability for each of the measurements were analyzed, using the F test for individual streets. Because of the large number of observations, nonparametric tests were used to detect differences between the variables of the four subareas.

Referring to the foregoing tables, the following differences were significant at the $\alpha = 0.05$ level:

AM period.—

1. On the east-west streets north of Broadway, speed was reduced by 1.3 mph (6.1 percent), the number of stops was increased by 5.0 per vehicle (41.7 percent), delay time per stop was reduced by 4.3 sec per vehicle (18.3 percent), and the number of stops per mile was increased by 0.3 per vehicle (42.9 percent).

2. On Broadway, speed increased by 1.6 mph (9.6 percent), delay time decreased by 47.0 sec per vehicle (49.2 percent), number of stops decreased by 2.2 per vehicle (51.2 percent), and the number of stops per mile decreased by 1.5 per vehicle (51.7 percent).

3. On the east-west streets south of Broadway, speed was reduced by 1.9 mph (9.0 percent), delay time was increased by 13.0 sec per vehicle (48.3 percent), number of stops was increased by 0.9 per vehicle (81.8 percent), average delay time per stop was reduced by 3.9 sec per vehicle (16.4 percent), and the number of stops per mile was increased by 0.6 per vehicle (75.0 percent).

4. On the north-south streets, the number of stops increased by 0.3 per vehicle (11.1 percent), and the delay time per stop decreased by 5.9 sec per vehicle (22.2 percent).

5. The variability of the "before" and "after" measurements on Broadway decreased significantly for delay time, number of stops, and stops per mile and increased significantly for delay time per stop. The variability of delay time per stop was reduced for the east-west streets south of Broadway and for the north-south streets.

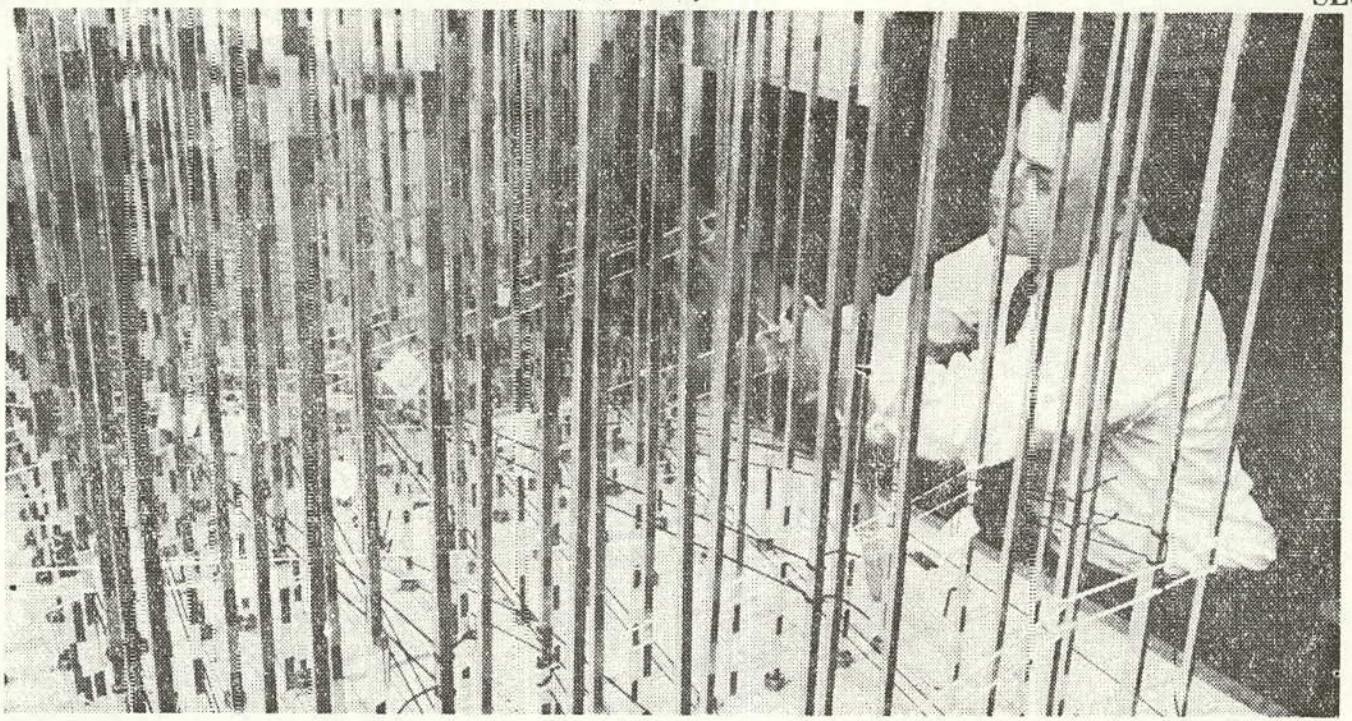
Midday period.—

1. On the east-west streets north of Broadway, the number of stops increased significantly by 0.5 per vehicle (41.7 percent), delay time per stop decreased by 5.1 sec per vehicle (20.7 percent), and stops per mile increased by 0.3 per vehicle (42.9 percent).

2. On Broadway, average delay time decreased by 21.5 sec per vehicle (26.2 percent), number of stops decreased

The Courier-Journal

MONDAY, MAY 19, 1969
SECTION B



Staff Photo

WALTER KRAFT'S 'ERECTOR SET' HAS BECOME CITY HALL CONVERSATION PIECE.

'Whatzit' Is a Model of City Traffic-Signal System

Engineers at City Hall are playing with a big Erector set—and it's not just for fun.

Visitors to the Traffic Engineering Department wonder what they've built and those who know have a hard time explaining it. But without doubt it's THE conversation piece of local government.

The engineers' gadget of aluminum rods and strings is a model of Louisville's traffic-signal system; it will help them take some of the kinks out of the city's traffic flow.

The model was assembled by Walter H. Kraft, senior traffic engineer of Edwards & Kelcey, Newark, N.J., traffic consultants; John Wilbanks, also of the Newark firm, and James Pasikowski, city traffic signal engineer.

The project is part of traffic research that Edwards & Kelcey has been doing in Louisville for more than a year under the auspices of the National Highway Research Board. The study is being made under a $1 million research-and-demonstration project conducted by the National Cooperative Highway Research Program to find ways to improve traffic movement.

The aluminum rods are mounted on a map of Louisville at each of some 280 intersections that have traffic signals. Each rod has red tape markings designating time intervals of light changes.

Engineers can take sightings on the rods and markings down any street and tell where the red-green light changes should be adjusted to get a smoother, faster flow of traffic.

Traffic Engineer Arthur R. Daniel Jr. said he plans to use information supplied by the model to tie together the entire traffic system.

Traffic signals now operate under three different systems—one north of Broadway, one south, and one for Broadway itself, he explained. Consequently, there are variations in the timing of vehicle movement.

With the entire system tied together, it should be possible to travel from one end of a street to the other without hitting a red light.

The model is the brainchild of Kraft, the Edwards & Kelcey traffic expert, and was designed at the firm's Newark office. It took four men a week just to put the time data on the rods, Kraft said.

Time cycles of traffic signals throughout the city and other pertinent traffic data were processed by computer before being transferred to the model rods.

Daniel hopes eventually to have a completely computerized traffic system. Information now being gathered can be used toward that end, he said.

Figure G-178. Newspaper publicity.

TABLE G-151

TRAFFIC SIGNAL TIMING

MAIN STREET (PHASE A)	CROSS STREET (PHASE B)	A.M.				P.M.			
		SYSTEM OFFSET, %		MAIN ST. % GREEN & AMBER		SYSTEM OFFSET, %		MAIN ST.% GREEN & AMBER	
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
BRECKINRIDGE ST.	LOGAN ST.	97	34	50	50	97	95	50	45
	SHELBY ST.	13	60	58	50	22	8	49	45
	CLAY ST.	34	88	63	50	38	38	64	65
	HANCOCK ST.	55	27	58	50	54	61	61	55
	JACKSON ST.	88	56	50	50	88	3	50	62
	PRESTON ST.	13	93	59	50	12	5	50	60
	FLOYD ST.	32	27	58	60	32	32	58	62
	BROOK ST.	47	43	45	50	47	66	45	55
	FIRST ST.	69	82	50	56	68	99	50	50
	SECOND ST.	95	96	48	50	94	31	50	48
	THIRD ST.	21	33	45	50	21	55	44	55
	FOURTH ST.	46	54	49	50	46	63	48	53
	FIFTH ST.	68	78	55	50	68	83	55	50
	SIXTH ST.	91	95	53	46	91	12	51	55
	SEVENTH ST.	18	24	45	56	19	35	40	45
	EIGHTH ST.	45	48	42	50	45	43	49	50
BROADWAY ST.	CAMPBELL ST.	0	0	64	64	0	0	64	54
	SHELBY ST.	13	12	61	61	13	2	61	56
	CLAY ST.	97	9	64	64	97	87	64	56
	HANCOCK ST.	48	48	69	67	48	69	69	56
	JACKSON ST.	59	60	68	61	59	64	68	56
	PRESTON ST.	46	58	64	64	46	64	64	56
	FLOYD ST.	86	97	68	68	86	43	68	56
	BROOK ST.	16	12	57	57	16	40	57	56
	FIRST ST.	0	0	61	61	0	36	61	56
	SECOND ST.	2	98	61	61	2	14	61	56
	THIRD ST.	47	45	63	63	47	7	63	56
	FOURTH ST.	54	63	60	60	54	0	60	56
	FIFTH ST.	45	53	66	66	45	95	66	56
	SIXTH ST.	94	97	60	60	94	3	60	56
	SEVENTH ST.	99	98	60	60	99	5	60	56
	EIGHTH ST.	6	14	64	64	6	98	64	56
	NINTH ST.	2	0	68	64	2	92	68	67
	TENTH ST.	58	92	68	67	58	85	68	67
	ELEVENTH ST.	71	43	71	67	71	46	71	65
	TWELFTH ST.	78	55	68	67	78	55	68	56
	THIRTEENTH ST.	60	59	68	67	60	61	68	56
BURNETT ST.	SECOND ST.	54	38	46	46	54	64	48	48
	FIRST ST.	23	35	43	44	23	58	42	43
	BROOK ST.	65	17	42	42	65	38	42	43

MAIN STREET (PHASE A)	CROSS STREET (PHASE B)	A.M.				P.M.			
		SYSTEM OFFSET, %		MAIN ST. % GREEN & AMBER		SYSTEM OFFSET, %		MAIN ST.% GREEN & AMBER	
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"
CHESTNUT ST.	12TH. ST.	60	76	55	56	60	33	55	55
	NINTH ST.	25	33	65	56	25	90	65	67
	EIGHTH ST.	48	64	50	60	48	90	50	60
	SEVENTH ST.	75	88	50	60	75	38	50	52
	SIXTH ST.	98	18	50	46	98	58	50	50
	FIFTH ST.	25	45	50	64	25	90	50	58
	FOURTH ST.	48	64	50	50	48	17	50	50
	THIRD ST.	75	88	50	54	75	41	50	48
	SECOND ST.	98	16	50	52	98	46	50	62
	FIRST ST.	25	38	65	50	25	75	65	55
	BROOK ST.	40	42	50	46	40	90	50	50
	PRESTON ST.	75	35	55	54	75	67	55	67
	JACKSON ST.	98	61	50	50	98	90	50	50
	HANCOCK ST.	95	88	55	50	95	28	55	55
	CLAY ST.	48	31	55	58	48	54	55	62
	SHELBY ST.	75	53	55	56	75	89	55	50
	CAMPBELL ST.	98	88	55	46	98	7	55	60
CONGRESS ALLEY	FOURTH ST.	50	50	40	38	50	50	40	40
GRAY ST.	BROOK ST.	90	76	45	40	90	65	45	43
GUTHRIE ST.	FOURTH ST.	60	60	36	28	60	60	36	29
	THIRD ST.	13	88	34	40	13	82	34	34
JEFFERSON ST.	BAXTER AVE.	25	38	50	50	25	56	50	52
	WENZEL ST.	44	63	64	60	44	59	64	62
	CAMPBELL ST.	70	88	55	46	70	89	55	47
	SHELBY ST.	98	19	55	66	98	7	55	52
	CLAY ST.	25	69	55	56	25	32	55	50
	HANCOCK ST.	50	88	57	50	50	47	57	60
	JACKSON ST.	75	38	57	50	75	67	57	55
	PRESTON ST.	95	66	55	50	95	18	55	48
	FLOYD ST.	10	8	55	60	10	49	55	67
	BROOK ST.	20	30	55	58	20	79	55	62
	FIRST ST.	50	61	57	56	50	10	57	50
	SECOND ST.	70	80	43	60	70	38	43	38
	THIRD ST.	98	14	55	50	98	69	55	50
	FOURTH ST.	20	39	55	50	20	90	55	63
	FIFTH ST.	49	62	58	50	49	22	58	50

TABLE G-151 (*continued*)

MAIN STREET (PHASE A)	CROSS STREET (PHASE B)	A.M. SYSTEM OFFSET, % "BEFORE"	A.M. SYSTEM OFFSET, % "AFTER"	A.M. MAIN ST. % GREEN & AMBER "BEFORE"	A.M. MAIN ST. % GREEN & AMBER "AFTER"	P.M. SYSTEM OFFSET, % "BEFORE"	P.M. SYSTEM OFFSET, % "AFTER"	P.M. MAIN ST. % GREEN & AMBER "BEFORE"	P.M. MAIN ST. % GREEN & AMBER "AFTER"
JEFFERSON ST. (CONT'D)	SIXTH ST.	70	88	50	60	70	45	50	43
	SEVENTH ST.	98	18	55	50	98	79	55	50
	EIGHTH ST.	20	40	47	48	20	90	47	55
	NINTH ST.	45	64	48	60	45	21	48	47
	12TH. ST.	10	26	55	56	10	83	55	55
KENTUCKY ST.	EIGHTH ST.	27	7	45	54	22	58	49	62
	SEVENTH ST.	45	78	53	50	47	75	59	45
	SIXTH ST.	68	41	46	44	68	77	55	50
	FIFTH ST.	7	77	40	56	14	7	50	52
	FOURTH ST.	22	91	49	52	22	13	58	50
	THIRD ST.	47	2	49	52	47	23	49	55
	SECOND ST.	66	40	50	46	68	60	50	57
	FIRST ST.	95	61	50	50	95	62	50	53
	BROOK ST.	18	93	40	50	18	4	49	62
	FLOYD ST.	39	16	61	56	39	17	61	63
	PRESTON ST.	48	54	49	54	47	50	49	52
	JACKSON ST.	68	80	50	56	68	70	55	60
	CLAY ST.	13	38	64	56	12	17	63	52
	SHELBY ST.	61	68	56	50	36	46	64	58
	LOGAN ST.	63	86	49	50	63	53	50	60
LIBERTY ST.	EIGHTH ST.	98	30	55	50	98	74	55	57
	SEVENTH ST.	20	42	45	46	20	90	45	60
	SIXTH ST.	50	80	57	52	50	20	57	50
	FIFTH ST.	70	88	45	48	70	28	45	62
	FOURTH ST.	98	29	50	50	98	60	50	52
	THIRD ST.	20	42	50	46	20	72	50	60
	SECOND ST.	52	75	55	50	52	17	55	50
	FIRST ST.	70	88	50	46	70	22	50	57
	BROOK ST.	98	23	55	54	98	55	55	52
	FLOYD ST.	15	62	69	60	15	88	69	67
	PRESTON ST.	30	8	55	48	30	20	55	67
	JACKSON ST.	51	35	55	56	51	48	55	60
	HANCOCK ST.	73	73	54	55	73	73	54	55
	CLAY ST.	98	8	57	50	98	7	57	67
	SHELBY ST.	25	51	55	56	25	29	55	50
	CAMPBELL ST.	50	73	57	46	50	54	57	63
	WENZEL ST.	8	8	50	50	8	81	50	50

MAIN STREET (PHASE A)	CROSS STREET (PHASE B)	A.M. SYSTEM OFFSET, % "BEFORE"	A.M. SYSTEM OFFSET, % "AFTER"	A.M. MAIN ST. % GREEN & AMBER "BEFORE"	A.M. MAIN ST. % GREEN & AMBER "AFTER"	P.M. SYSTEM OFFSET, % "BEFORE"	P.M. SYSTEM OFFSET, % "AFTER"	P.M. MAIN ST. % GREEN & AMBER "BEFORE"	P.M. MAIN ST. % GREEN & AMBER "AFTER"
MAGNOLIA ST.	SIXTH ST.	9	51	59	60	9	10	59	50
	FOURTH ST.	81	50	42	42	81	37	42	43
	THIRD ST.	61	3	43	44	61	59	39	38
	SECOND ST.	65	61	42	42	95	90	40	50
	FIRST ST.	74	2	34	34	74	23	34	50
	BROOK ST.	70	40	51	52	70	45	51	50
	FLOYD ST.	2	75	54	54	7	74	63	50
MAIN ST.	JOHNSON ST.	69	88	60	52	69	7	60	52
	BAXTER ST.	75	14	50	46	75	21	50	33
	WENZEL ST.	98	38	67	70	98	44	67	67
	CAMPBELL ST.	25	73	55	50	25	73	55	45
	SHELBY ST.	50	10	57	60	50	88	57	67
	CLAY ST.	75	28	55	60	75	23	55	67
	JACKSON ST.	25	88	55	60	25	75	55	63
	PRESTON ST.	45	31	53	56	45	88	53	50
	BROOK ST.	73	88	50	60	73	37	50	53
	FIRST ST.	98	18	55	50	98	60	55	57
	SECOND ST.	25	56	49	52	25	81	49	48
	THIRD ST.	51	73	50	50	51	7	50	55
	FOURTH ST.	75	88	50	60	75	25	50	67
	FIFTH ST.	98	28	55	60	98	61	55	58
	SIXTH ST.	25	35	45	50	25	85	45	48
	SEVENTH ST.	48	74	60	46	48	11	60	52
	12TH ST.	70	86	55	56	70	48	55	58
MARKET ST.	12TH ST.	50	66	56	56	50	23	56	57
	11TH ST.	—	90	—	55	—	52	—	55
	TENTH ST.	98	14	65	60	98	71	65	65
	NINTH ST.	25	38	60	70	25	17	60	67
	EIGHTH ST.	48	64	50	60	48	27	50	55
	SEVENTH ST.	75	83	50	60	75	52	50	67
	SIXTH ST.	98	13	50	56	98	61	50	48
	FIFTH ST.	25	39	55	50	25	80	55	67
	FOURTH ST.	48	65	50	56	48	7	50	58
	THIRD ST.	75	83	50	56	75	28	50	62
	SECOND ST.	90	11	50	56	90	58	50	57
	FIRST ST.	25	31	55	58	25	87	55	63
	BROOK ST.	48	63	50	56	48	9	50	60
	FLOYD ST.	70	88	55	60	70	47	55	60
	PRESTON ST.	80	36	55	52	80	74	55	60

TABLE G-151 (*continued*)

MAIN STREET (PHASE A)	CROSS STREET (PHASE B)	A.M. SYSTEM OFFSET, % "BEFORE"	A.M. SYSTEM OFFSET, % "AFTER"	A.M. MAIN ST. % GREEN & AMBER "BEFORE"	A.M. MAIN ST. % GREEN & AMBER "AFTER"	P.M. SYSTEM OFFSET, % "BEFORE"	P.M. SYSTEM OFFSET, % "AFTER"	P.M. MAIN ST. % GREEN & AMBER "BEFORE"	P.M. MAIN ST. % GREEN & AMBER "AFTER"
MARKET ST. (CONT'D)	JACKSON ST.	98	69	55	56	98	90	55	60
	HANCOCK ST.	75	8	58	60	75	11	58	60
	CLAY ST.	50	49	56	60	50	41	56	67
	SHELBY ST.	75	79	55	56	75	75	55	60
	CAMPBELL ST.	98	13	55	60	98	7	55	67
	WENZEL ST.	25	38	55	50	25	44	55	63
	BAXTER ST.	50	73	65	60	50	60	65	68
MID BLOCK	4th.-WALNUT TO								
	LIBERTY ST.	23	23	35	40	23	23	35	40
	4th BROADWAY								
	TO CHESTNUT ST.	75	75	35	40	75	75	35	40
OAK ST.	EIGHTH ST.	82	20	60	60	85	42	63	63
	SEVENTH ST.	4	46	48	50	5	67	50	50
	SIXTH ST.	30	70	40	40	25	88	55	55
	GARVIN ST.	49	96	62	62	46	16	58	65
	FOURTH ST.	68	8	57	58	68	41	55	55
	THIRD ST.	93	46	52	52	95	72	55	55
	SECOND ST.	21	73	47	48	16	91	58	58
	FIRST ST.	42	91	52	52	40	21	55	55
	BROOK ST.	62	33	48	48	59	43	55	55
	FLOYD ST.	87	64	55	56	83	74	58	58
	PRESTON ST.	3	96	53	54	5	10	55	55
	JACKSON ST.	22	26	57	58	28	38	52	52
	CLAY ST.	67	75	62	58	68	91	60	60
	SHELBY ST.	90	96	50	50	95	18	50	50
	LOGAN ST.	13	29	45	46	10	36	52	52
ORMSBY ST.	SEVENTH ST.	96	0	37	38	97	12	39	38
	SIXTH ST.	47	68	41	42	46	58	41	48
	FOURTH ST.	93	62	51	52	93	62	50	50
	THIRD ST.	68	0	48	48	68	8	47	47
	SECOND ST.	80	25	51	52	82	62	49	52
	BROOK ST.	45	94	31	40	45	14	32	40
ST. CATHERINE ST.	LOGAN ST.	43	52	43	50	43	46	42	47
	SHELBY ST.	69	63	50	50	68	1	50	40

MAIN STREET (PHASE A)	CROSS STREET (PHASE B)	A.M. SYSTEM OFFSET, % "BEFORE"	A.M. SYSTEM OFFSET, % "AFTER"	A.M. MAIN ST. % GREEN & AMBER "BEFORE"	A.M. MAIN ST. % GREEN & AMBER "AFTER"	P.M. SYSTEM OFFSET, % "BEFORE"	P.M. SYSTEM OFFSET, % "AFTER"	P.M. MAIN ST. % GREEN & AMBER "BEFORE"	P.M. MAIN ST. % GREEN & AMBER "AFTER"
ST. CATHERINE ST. (CONT'D)	CLAY ST.	93	1	64	58	93	23	63	52
	JACKSON ST.	34	64	47	58	35	51	49	57
	PRESTON ST.	68	4	52	52	48	93	52	43
	FLOYD ST.	80	36	64	60	80	28	63	58
	BROOK ST.	30	76	43	52	97	69	51	60
	FIRST ST.	23	92	49	50	22	95	44	48
	SECOND ST.	46	3	49	58	47	31	49	53
	THIRD ST.	68	33	52	56	68	43	50	48
	FOURTH ST.	93	48	50	56	93	68	51	57
	SIXTH ST.	46	97	50	52	44	17	50	60
	SEVENTH ST.	68	17	50	50	68	28	50	43
	EIGHTH ST.	93	40	55	54	93	51	59	67
WALNUT ST.	CAMPBELL ST.	25	42	55	46	25	36	55	40
	SHELBY ST.	48	88	55	60	48	61	55	63
	CLAY ST.	75	13	57	56	75	90	57	50
	HANCOCK ST.	98	52	55	60	98	7	55	50
	JACKSON ST.	25	88	55	48	25	42	55	45
	PRESTON ST.	50	8	57	60	50	56	57	57
	BROOK ST.	85	88	45	46	85	37	45	40
	FIRST ST.	98	10	53	56	98	58	53	50
	SECOND ST.	25	50	50	58	25	90	50	33
	THIRD ST.	48	63	48	56	48	15	48	48
	FOURTH ST.	75	88	50	60	75	51	50	62
	FIFTH ST.	98	16	50	56	98	72	50	55
	SIXTH ST.	25	42	50	46	25	90	50	45
	SEVENTH ST.	48	72	50	50	48	21	50	55
	EIGHTH ST.	75	88	50	52	75	42	50	52
	NINTH ST.	98	9	55	52	98	70	55	55
	12TH ST.	60	76	56	56	60	33	56	55
WOODBINE ST.	BROOK ST.	9	66	50	50	4	72	45	50
	FLOYD ST.	23	70	50	50	23	57	50	50
YORK ST.	EIGHTH ST.	72	90	41	50	72	81	41	43
	SEVENTH ST.	9	85	32	56	9	9	32	52
	SIXTH ST.	49	23	34	48	49	72	34	38
	FIFTH ST.	59	51	36	56	70	44	31	50
	FOURTH ST.	81	77	41	56	6	22	41	50
	THIRD ST.	8	97	36	36	7	44	40	37
	SECOND ST.	33	49	34	40	32	67	33	60

TABLE G-152

LIMITS OF SPEED AND DELAY RUNS

STREET	LIMITS	DISTANCE (FT)
Breckinridge St.	Logan St. to Eighth St.	7,710
Broadway	Campbell St. to Eighth St.	7,880
Brook St.	Oak St. to Main St.	8,100
Chestnut St.	Ninth St. to Campbell St.	8,310
Jefferson St.	Baxter Ave. to Ninth St.	9,345
Kentucky St.	Eighth St. to Logan St.	7,820
Liberty St.	Ninth St. to Wenzel St.	8,950
Main St.	Baxter Ave. to Ninth St.	9,370
Market St.	Ninth St. to Baxter Ave.	9,370
Oak St.	Seventh St. to Logan St.	7,300
St. Catherine St.	Logan St. to Eighth St.	7,910
Walnut St.	Campbell St. to Ninth St.	8,300
First St.	Main St. to Oak St.	7,940
Second St.	Oak St. to Walnut St.	5,880
Third St.	Main St. to Oak St.	7,950
Fourth St.	Oak St. to Main St.	7,960
Fifth St.	Main St. to Kentucky St.	6,210
Sixth St.	Oak St. to Main St.	7,840
Seventh St.	Main St. to Oak St.	8,020

by 1.0 per vehicle (26.3 percent), and stops per mile decreased by 0.7 per vehicle (26.9 percent).

3. On the east-west streets south of Broadway, speed decreased by 2.1 mph (9.4 percent), delay time increased by 18.0 sec per vehicle (104.0 percent), number of stops increased by 0.9 per vehicle (112.5 percent), and stops per mile increased by 0.7 per vehicle (140.0 percent).

TABLE G-163

1968 AVERAGE WEEKDAY TRAFFIC VOLUMES

STREET	LIMITS	PEAK-HR VEHICLES AM	PEAK-HR VEHICLES PM
Breckinridge St.	Logan St. to Eighth St.	1,100	600
Broadway EB	Eighth St. to Campbell St.	850	1,550
Broadway WB	Campbell St. to Eighth St.	1,250	700
Brook St.	Oak St. to Main St.	750	700
Chestnut St.	Ninth St. to Campbell St.	950	1,150
Fifth St.	Main St. to Kentucky St.	650	800
First St.	Main St. to Oak St.	750	1,300
Fourth St. NB	Oak St. to Main St.	450	400
Jefferson St.	Baxter Ave. to Ninth St.	1,300	850
Kentucky St.	Eighth St. to Logan St.	400	1,150
Liberty St.	Ninth St. to Wenzel St.	600	1,200
Main St.	Baxter Ave. to Ninth St.	1,300	1,100
Market St. EB	Ninth St. to Baxter Ave.	850	1,100
Oak St.	Seventh St. to Logan St.	550	950
St. Catherine St.	Logan St. to Eighth St.	1,150	850
Second St.	Oak St. to Walnut St.	1,300	1,200
Seventh St.	Main St. to Oak St.	650	1,050
Sixth St.	Oak St. to Main St.	1,000	1,050
Third St.	Main St. to Oak St.	900	1,200
Walnut St.	Campbell St. to Ninth St.	900	700
All		17,650	19,600

4. On the north-south streets, speed increased by 1.5 mph (9.1 percent), delay time decreased by 10.6 sec per vehicle (14.0 percent), and delay time per stop decreased by 2.7 sec per vehicle (11.3 percent).

5. On the east-west streets north of Broadway, the variability of the "before" and "after" measurements increased significantly for the number of stops and the number of stops per mile. On the east-west streets south of Broadway, the variability of delay time per stop decreased significantly. On the north-south streets, the variability of speed increased significantly, whereas the variability of stops per mile decreased significantly.

PM period.—

1. On the east-west streets north of Broadway, delay time per stop decreased by 3.8 sec per vehicle (15.4 percent).

2. On Broadway, delay time increased by 32.7 sec per vehicle (28.4 percent), delay time per stop increased by 20.3 sec per vehicle (93.5 percent), and stops per mile decreased by 1.1 per vehicle (30.6 percent).

3. On the east-west streets south of Broadway, speed decreased by 2.0 mph (11.4 percent), delay time increased by 19.4 sec per vehicle (33.8 percent), number of stops increased by 1.4 per vehicle (70 percent), delay time per stop decreased by 5.4 sec per vehicle (18.8 percent), and stops per mile increased by 0.9 per vehicle (64.3 percent).

4. On the north-south streets, the number of stops increased by 0.6 per vehicle (16.2 percent), delay time per stop decreased by 2.9 sec per vehicle (10.7 percent), and stops per mile increased by 0.4 per vehicle (15.4 percent).

5. The variability of the "before" and "after" measurements indicated a significant increase in delay time per stop on Broadway, in delay time on the north-south streets, and in stops per mile on the east-west streets south of Broadway.

The 1968 average weekday traffic volumes for the morning and afternoon peak hours developed for SIGOP input were used to weigh the differences between the "before" and "after" measurements. This task was accomplished by determining the average volume on each street (Table G-163). Table G-164 gives the delay time, number of stops, and stops per mile for all vehicles traveling on the indicated streets during the AM and PM peak hours. Table G-165 is a summary for the four subareas. Generally, delay time was reduced during the AM time period, whereas all other variables increased during the AM and PM time periods.

Conclusions

Based on the analysis of data from the travel time survey of all arterials, it can be concluded that:

1. During the AM time period, average speed decreased by 0.5 mph (2.6 percent), delay time decreased by 3.4 sec per vehicle (6.7 percent), number of stops increased by 0.2 per vehicle (9.5 percent), delay time per stop decreased by 4.4 sec per vehicle (17.9 percent), and the number of stops per mile increased by 0.2 (14.3 percent). In terms of total travel during the AM peak hour, delay time decreased

TABLE G-153

SPEED AND DELAY DATA, MEAN VALUES, EAST-WEST STREETS NORTH OF BROADWAY

TIME PERIOD	STREET	SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE
AM	MAIN ST.	22.1	22.8	0.7 (3.2%)	27.1	23.3	-3.8 (-14.0%)	1.3	1.3	0.0 (0.0%)	20.1	16.2	-3.9 (-19.4%)	0.8	0.7	-0.1 (-12.5%)
	MARKET ST.*	21.9	18.7	-3.2 (-14.6%) b	17.0	39.1	22.1 (130.0%) b	0.7	2.1	1.4 (200.0%) b	23.6	19.3	-4.3 (-18.2%) b	0.4	1.2	0.8 (200.0%) b
	JEFFERSON ST	22.5	21.0	-1.5 (-6.7%) b	20.0	15.8	-4.2 (-21.0%)	0.7	0.9	0.2 (28.6%)	27.5	20.4	-7.1 (-25.8%) b	0.4	0.5	0.1 (25.0%)
	LIBERTY ST.	22.2	19.1	-3.1 (-14.0%) b	20.5	40.6	20.1 (98.0%) b	0.9	2.3	1.4 (155.6%) b	22.1	18.9	-3.2 (-14.5%)	0.5	1.3	0.8 (160.0%) b
	WALNUT ST.	18.4	17.9	-0.5 (-2.7%)	45.5	38.0	-7.5 (-16.5%)	1.9	2.0	0.1 (5.3%)	26.1	20.2	-5.9 (-22.6%) b	1.2	1.3	0.1 (8.3%)
	CHESTNUT ST.	20.4	19.8	-0.6 (-2.9%)	28.4	34.9	6.5 (22.9%)	1.4	1.9	0.5 (35.7%)	21.2	20.5	-0.7 (-3.3%)	0.9	1.2	0.3 (33.3%)
	AVERAGE	21.2	19.9	-1.3 (-6.1%) b	26.4	31.9	5.5 (20.8%)	1.2	1.7	0.5 (41.7%) b	23.5	19.2	-4.3 (-18.3%) b	0.7	1.0	0.3 (42.9%) b
MIDDAY	MAIN ST.	22.4	23.3	0.9 (4.0%)	22.2	22.4	0.2 (0.9%)	1.1	1.1	0.0 (0.0%)	20.8	21.2	0.4 (1.9%)	0.6	0.6	0.0 (0.0%)
	MARKET ST.*	19.6	19.7	0.1 (0.5%)	36.6	25.9	-10.7 (-29.2%)	1.5	1.3	-0.2 (-13.3%)	24.8	20.5	-4.3 (-17.3%) a	0.8	0.8	0.0 (0.0%)
	JEFFERSON ST	24.1	22.3	-1.8 (-7.5%) b	9.5	13.3	3.8 (40.0%)	0.4	1.4	1.0 (250.0%) b	24.4	9.6	-14.8 (-60.7%) b	0.2	0.8	0.6 (300.0%) b
	LIBERTY ST.	21.0	18.9	-2.1 (-10.0%) b	31.4	46.8	15.4 (49.0%)	1.2	2.0	0.8 (66.7%) a	26.6	24.2	-2.4 (-9.0%)	0.7	1.2	0.5 (71.4%) b
	WALNUT ST.	19.2	18.5	-0.7 (-3.6%)	34.0	39.3	5.3 (15.6%)	1.2	2.0	0.8 (66.7%) b	27.9	21.2	-6.7 (-24.0%) b	0.8	1.3	0.5 (62.5%) b
	CHESTNUT ST.	18.6	19.9	1.3 (7.0%)	42.2	42.1	-0.1 (-0.2%)	1.9	2.4	0.5 (26.3%)	23.1	20.2	-2.9 (-12.6%)	1.2	1.5	0.3 (25.0%)
	AVERAGE	20.8	20.4	-0.4 (-1.9%)	29.3	31.6	2.3 (7.8%)	1.2	1.7	0.5 (41.7%) b	24.6	19.5	-5.1 (-20.7%) b	0.7	1.0	0.3 (42.9%) b
PM	MAIN ST.	21.6	23.9	2.3 (10.6%) b	34.6	15.0	-19.6 (-56.6%) b	1.7	0.7	-1.0 (-58.8%) b	20.1	22.5	2.4 (11.9%)	0.9	0.4	-0.5 (-55.6%) b
	MARKET ST.*	16.4	21.3	4.9 (29.9%) b	87.9	15.6	-72.3 (-82.3%) b	3.5	1.2	-2.3 (-65.7%) b	26.0	14.2	-11.8 (-45.4%) b	2.0	0.7	-1.3 (-65.0%) b
	JEFFERSON ST	23.8	17.9	-5.9 (-24.8%) b	11.1	65.5	54.4 (490.1%) b	0.5	3.3	2.8 (560.0%) b	24.7	20.2	-4.5 (-18.2%)	0.3	1.8	1.5 (500.0%) b
	LIBERTY ST.	18.5	18.0	-0.5 (-2.7%)	54.9	53.7	-1.2 (-2.2%)	2.3	2.2	-0.1 (-4.3%)	25.1	25.5	0.4 (1.6%)	1.3	1.3	0.0 (0.0%)
	WALNUT ST.	17.0	17.4	0.4 (2.4%)	65.5	36.0	-29.5 (-45.0%) b	2.5	1.6	-0.9 (-36.0%) b	27.3	24.5	-2.8 (-10.3%)	1.6	1.0	-0.6 (-37.5%) b
	CHESTNUT ST.	15.7	13.7	-2.0 (-12.7%)	87.7	113.8	26.1 (29.8%)	3.6	6.2	2.6 (72.2%) b	24.8	18.7	-6.1 (-24.6%) b	2.3	3.9	1.6 (69.6%) b
	AVERAGE	18.8	18.7	-0.1 (-0.5%)	56.9	49.9	-7.0 (-12.3%)	2.3	2.5	0.2 (8.7%)	24.7	20.9	-3.8 (-15.4%) b	1.4	1.5	0.1 (7.1%)

a – SIGNIFICANT AT $.10 \geq \alpha > .05$

b – SIGNIFICANT AT $\alpha \leq .05$

* EASTBOUND DIRECTION ONLY

TABLE G-154

SPEED AND DELAY DATA, MEAN VALUES, BROADWAY

TIME PERIOD	DIRECTION	SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE
AM	EASTBOUND	19.2	21.1	1.9 (9.9%) b	65.3	30.3	-35.0 (-53.6%) b	3.2	1.4	-1.8 (-56.3%) b	19.9	18.9	-1.0 (-5.0%)	2.1	0.9	-1.2 (-57.1%) b
	WESTBOUND	14.3	15.5	1.2 (8.4%) b	125.7	66.8	-58.9 (-46.9%) b	5.4	2.8	-2.6 (-48.1%) b	23.3	25.2	1.9 (8.2%) a	3.6	1.8	-1.8 (-50.0%) b
	AVERAGE	16.7	18.3	1.6 (9.6%) b	95.5	48.5	-47.0 (-49.2%) b	4.3	2.1	-2.2 (-51.2%) b	21.6	22.0	0.4 (1.9%)	2.9	1.4	-1.5 (-51.7%) b
MIDDAY	EASTBOUND	18.7	19.9	1.2 (6.4%)	68.5	39.0	-29.5 (-43.1%) b	3.1	1.8	-1.3 (-41.9%) b	22.4	22.2	-0.2 (-0.9%)	2.1	1.2	-0.9 (-42.9%) b
	WESTBOUND	15.5	14.4	-1.1 (-7.1%) a	99.5	81.9	-17.6 (-17.7%) a	4.6	3.8	-0.8 (-17.4%) a	22.4	21.4	-1.0 (-4.5%)	3.1	2.5	-0.6 (-19.4%) a
	AVERAGE	17.1	17.2	0.1 (0.6%)	82.0	60.5	-21.5 (-26.2%) b	3.8	2.8	-1.0 (-26.3%) b	22.4	21.8	-0.6 (-2.7%)	2.6	1.9	-0.7 (-26.9%) b
PM	EASTBOUND	14.4	17.9	3.5 (24.3%) b	113.2	75.7	-37.5 (-33.1%) b	5.6	2.1	-3.5 (-62.5%) b	20.5	41.8	21.3 (103.9%) b	3.8	1.4	-2.4 (-63.2%) b
	WESTBOUND	14.5	11.1	-3.4 (-23.4%) b	116.8	219.7	102.9 (88.1%) b	5.3	5.3	0.0 (0.0%)	22.9	42.3	19.4 (84.7%) b	3.5	3.6	0.1 (2.9%)
	AVERAGE	14.4	14.5	0.1 (0.7%)	115.0	147.7	32.7 (28.4%) b	5.4	3.7	-1.7 (-31.5%) b	21.7	42.0	20.3 (93.5%) b	3.6	2.5	-1.1 (-30.6%) b

a – SIGNIFICANT AT .10 ≥ α > .05

b – SIGNIFICANT AT α ≤ 05

TABLE G-155

SPEED AND DELAY DATA, MEAN VALUES, EAST-WEST STREETS SOUTH OF BROADWAY

TIME PERIOD	STREET	SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE
AM	BRECKINRIDGE ST.	20.7	18.3	-2.4 (-11.6%) b	33.1	40.5	7.4 (22.4%)	1.4	2.1	0.7 (50.0%) b	25.6	20.0	-5.6 (-21.9%)	0.9	1.5	0.6 (66.7%) b
	KENTUCKY ST.	24.0	22.0	-2.0 (-8.3%) b	9.9	25.2	15.3 (154.5%) b	0.6	1.8	1.2 (200.0%) b	18.0	13.8	-4.2 (-23.3%)	0.4	1.2	0.8 (200.0%) b
	ST. CATHERINE ST.	19.3	17.3	-2.0 (-10.4%) b	40.1	57.7	17.6 (43.9%) b	1.6	2.8	1.2 (75.0%) b	27.2	20.9	-6.3 (-23.2%) b	1.1	1.9	0.8 (72.7%) b
	OAK ST.	20.7	19.7	-1.0 (-4.8%)	24.8	36.2	11.4 (46.0%) a	1.0	1.5	0.5 (50.0%) b	24.4	25.2	0.8 (3.3%)	0.7	1.1	0.4 (57.1%) b
	AVERAGE	21.2	19.3	-1.9 (-9.0%) b	26.9	39.9	13.0 (48.3%) b	1.1	2.0	0.9 (81.8%) b	23.8	19.9	-3.9 (-16.4%) b	0.8	1.4	0.6 (75.0%) b
MIDDAY	BRECKINRIDGE ST.	21.9	19.9	-2.0 (-9.1%) b	22.2	38.9	16.7 (75.2%) b	1.1	2.0	0.9 (81.8%) b	21.1	21.1	0.0 (0.0%)	0.8	1.4	0.6 (75.0%) b
	KENTUCKY ST.	23.9	21.7	-2.2 (-9.2%) b	11.2	28.8	17.6 (157.1%) b	0.5	1.5	1.0 (200.0%) b	23.3	18.9	-4.4 (-18.9%)	0.3	1.0	0.7 (233.3%) b
	ST. CATHERINE	23.0	19.6	-3.4 (-14.8%) b	11.5	41.4	29.9 (260.0%) b	0.5	1.9	1.4 (280.0%) b	24.6	22.2	-2.4 (-9.8%)	0.3	1.3	1.0 (333.3%) b
	OAK ST.	20.5	19.8	-0.7 (-3.4%)	24.3	32.3	8.0 (32.9%) a	1.0	1.4	0.4 (40.0%) b	25.8	23.2	-2.6 (-10.1%) b	0.7	1.0	0.3 (42.9%) b
	AVERAGE	22.4	20.3	-2.1 (-9.4%) b	17.3	35.3	18.0 (104.0%) b	0.8	1.7	0.9 (112.5%) b	23.7	21.3	-2.4 (-10.1%) a	0.5	1.2	0.7 (140.0%) b
PM	BRECKINRIDGE ST.	17.9	17.4	-0.5 (-2.8%)	54.8	49.9	-4.9 (-8.9%)	2.0	2.3	0.3 (15.0%)	28.7	22.8	-5.9 (-20.6%) b	1.4	1.5	0.1 (7.1%)
	KENTUCKY ST.	18.2	16.4	-1.8 (-9.9%) a	61.6	75.4	13.8 (22.4%)	2.3	3.7	1.4 (60.9%) b	28.3	20.6	-7.7 (-27.2%) b	1.5	2.5	1.0 (66.7%) b
	ST. CATHERINE ST.	16.9	14.5	-2.4 (-14.2%) b	57.3	84.1	26.8 (46.8%) b	2.1	3.7	1.6 (76.2%) b	28.6	24.6	-4.0 (-14.0%) b	1.4	2.5	1.1 (78.6%)
	OAK ST.	17.2	14.1	-3.1 (-18.0%) b	56.0	98.8	42.8 (76.4%) b	1.8	3.8	2.0 (111.1%) b	29.1	25.1	-4.0 (-13.7%) b	1.3	2.8	1.5 (115.4%) b
	AVERAGE	17.6	15.6	-2.0 (-11.4%) b	57.4	76.8	19.4 (33.8%) b	2.0	3.4	1.4 (70.0%) b	28.7	23.3	-5.4 (-18.8%) b	1.4	2.3	0.9 (64.3%) b

a – SIGNIFICANT AT .10 ≥ α > .05

b – SIGNIFICANT AT α ≤ .05

TABLE G-156

SPEED AND DELAY DATA, MEAN VALUES, NORTH-SOUTH STREETS

TIME PERIOD	STREET	SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE
AM	BROOK ST.	20.9	21.3	0.4 (1.9%)	45.5	33.1	-12.4 (-27.3%) b	1.7	2.0	0.3 (17.6%)	28.1	17.0	-11.1 (-39.5%) b	1.1	1.3	0.2 (18.2%)
	1st. ST.	18.6	18.1	-0.5 (-2.7%)	69.3	70.2	0.9 (1.3%)	2.6	3.7	1.1 (42.3%) b	25.9	19.0	-6.9 (-26.6%) b	1.7	2.5	0.8 (47.1%) b
	2nd. ST.	17.0	19.7	2.7 (15.9%) b	65.7	33.3	-32.4 (-49.3%) b	2.3	1.3	-1.0 (-43.5%) b	29.9	24.6	-5.3 (-17.7%) b	2.1	1.2	-0.9 (-42.9%) b
	3rd. ST.	16.8	17.3	0.5 (3.0%)	85.9	70.7	-15.2 (-17.7%)	3.3	3.2	-0.1 (-3.0%)	26.9	22.3	-4.6 (-17.1%) b	2.2	2.1	-0.1 (-4.5%)
	4th. ST.*	14.8	13.9	-0.9 (-6.1%)	100.8	105.3	4.5 (4.5%)	3.9	4.8	0.9 (23.1%) a	27.1	22.1	-5.0 (-18.5%) b	2.6	3.2	0.6 (23.1%) a
	5th. ST.	17.6	17.1	-0.5 (-2.8%)	57.7	60.0	2.3 (4.0%)	2.2	3.0	0.8 (36.4%) b	27.1	19.4	-7.7 (-28.4%) b	1.9	2.5	0.6 (31.6%) a
	6th. ST.	16.6	17.1	0.5 (3.0%)	81.1	74.3	-6.8 (-8.4%)	3.0	3.7	0.7 (23.3%)	25.8	20.2	-5.6 (-21.7%) b	2.1	2.5	0.4 (19.0%)
	7th. ST.	19.3	19.6	0.3 (1.6%)	57.1	57.1	0.0 (0.0%)	2.5	2.4	-0.1 (-4.0%)	22.4	21.1	-1.3 (-5.8%)	1.6	1.6	0.0 (0.0%)
	AVERAGE	17.7	18.0	0.3 (1.7%)	70.4	63.0	-7.4 (-10.5%)	2.7	3.0	0.3 (11.1%) b	26.6	20.7	-5.9 (-22.2%) b	1.9	2.1	0.2 (10.5%) a
MIDDAY	BROOK ST.	21.1	20.5	-0.6 (-2.8%)	37.6	48.8	11.2 (29.8%) a	2.0	2.7	0.7 (35.0%) b	18.9	17.4	-1.5 (-7.9%)	1.3	1.8	0.5 (38.5%) b
	1st. ST.	19.3	21.2	1.9 (9.8%) a	46.3	41.6	-4.7 (-10.2%)	2.3	2.3	0.0 (0.0%)	20.3	17.1	-3.2 (-15.8%) a	1.5	1.5	0.0 (0.0%)
	2nd. ST.	16.6	20.4	3.8 (22.9%) b	65.7	24.8	-40.9 (-62.3%) b	2.5	1.0	-1.5 (-60.0%) b	27.6	21.2	-6.4 (-23.2%) b	2.2	0.9	-1.3 (-59.1%) b
	3rd. ST.	16.2	16.3	0.1 (0.6%)	72.8	85.9	13.1 (18.0%) a	3.2	3.3	0.1 (3.1%)	21.5	26.6	5.1 (23.7%) b	2.1	2.2	0.1 (4.8%)
	4th. ST.*	11.6	11.7	0.1 (0.9%)	142.9	127.7	-15.2 (-10.6%)	5.9	6.0	0.1 (1.7%)	24.4	21.2	-3.2 (-13.1%) b	3.9	4.0	0.1 (2.6%)
	5th. ST.	14.6	16.6	2.0 (13.7%) a	80.4	64.1	-16.3 (-20.3%)	3.1	3.0	-0.1 (-3.2%)	26.7	20.1	-6.6 (-24.7%) b	2.7	2.5	-0.2 (-7.4%)
	6th. ST.	15.4	17.9	2.5 (16.2%) b	86.9	60.6	-26.3 (-30.3%) b	3.3	3.0	-0.3 (-9.1%)	27.2	20.4	-6.8 (-25.0%) b	2.2	2.0	-0.2 (-9.1%)
	7th. ST.	17.4	19.1	1.7 (9.8%) a	72.3	66.4	-5.9 (-8.2%)	2.9	2.4	-0.5 (-17.2%)	24.6	25.6	1.0 (4.1%)	1.9	1.6	-0.3 (-15.8%)
	AVERAGE	16.5	18.0	1.5 (9.1%) b	75.6	65.0	-10.6 (-14.0%) b	3.1	3.0	-0.1 (-3.2%)	23.9	21.2	-2.7 (-11.3%) b	2.2	2.1	-0.1 (-4.5%)
PM	BROOK ST.	20.8	20.9	0.1 (0.5%)	47.4	32.0	-15.4 (-32.5%) a	1.7	1.4	-0.3 (-17.6%)	26.7	21.0	-5.7 (-21.3%) a	1.1	0.9	-0.2 (-18.2%)
	1st. ST.	16.4	16.7	0.3 (1.8%)	92.7	84.6	-8.1 (-8.7%)	3.5	4.0	0.5 (14.3%)	26.4	22.1	-4.3 (-16.3%) b	2.3	2.7	0.4 (17.4%)
	2nd. ST.	15.9	15.8	-0.1 (-0.6%)	78.6	58.3	-20.3 (-25.8%) a	2.7	2.1	-0.6 (-22.2%) b	30.1	28.4	-1.7 (-5.6%)	2.4	1.9	-0.5 (-20.8%) a
	3rd. ST.	14.0	17.1	3.1 (22.1%)	124.4	77.1	-47.3 (-38.0%) b	4.5	3.4	-1.1 (-24.4%)	28.4	24.7	-3.7 (-13.0%) a	3.0	2.3	-0.7 (-23.3%)
	4th. ST.*	12.0	10.6	-1.4 (-11.7%) b	145.0	194.4	49.1 (34.1%) b	6.0	8.3	2.3 (38.3%) b	24.1	23.5	-0.6 (-2.5%)	4.0	5.5	1.5 (37.5%) b
	5th. ST.	15.7	14.6	-1.1 (-7.0%)	76.6	76.5	-0.1 (-0.1%)	2.7	2.8	0.1 (3.7%)	27.8	29.2	1.4 (5.0%)	2.3	2.3	0.0 (0.0%)
	6th. ST.	13.7	11.4	-2.3 (-16.8%) a	160.4	169.7	9.3 (5.8%)	5.2	7.7	2.5 (48.1%) a	29.4	22.9	-6.5 (-22.1%) b	3.5	5.2	1.7 (48.6%) a
	7th. ST.	17.9	15.6	-2.3 (-12.8%) b	68.1	101.5	33.4 (49.0%) a	2.8	4.5	1.7 (60.7%) b	23.8	21.9	-1.9 (-8.0%)	1.9	3.0	1.1 (57.9%) b
	AVERAGE	15.8	15.3	-0.5 (-3.2%)	99.1	99.3	0.2 (0.2%)	3.7	4.3	0.6 (16.2%) b	27.1	24.2	-2.9 (-10.7%) b	2.6	3.0	0.4 (15.4%) b

a – SIGNIFICANT AT .10 ≥ α > .05

b – SIGNIFICANT AT α ≤ .05

* NORTHBOUND DIRECTION ONLY

TABLE G-157

SPEED AND DELAY DATA, MEAN VALUES, SUMMARY

TIME PERIOD	STREET	SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE	"BEFORE"	"AFTER"	DIFFERENCE
AM	EAST-WEST STREETS	20.5	19.4	-1.1 (-5.4%) b	38.1	37.3	-0.8 (-2.1%)	1.7	1.9	0.2 (11.8%) b	23.3	19.9	-3.4 (-14.6%) b	1.1	1.2	0.1 (9.1%) a
	NORTH OF BROADWAY	21.2	19.9	-1.3 (-6.1%) b	26.4	31.9	5.5 (20.8%)	1.2	1.7	0.5 (41.7%) b	23.5	19.2	-4.3 (-18.3%) b	0.7	1.0	0.3 (42.9%) b
	BROADWAY	16.7	18.3	1.6 (9.6%) b	95.5	48.5	-47.0 (-49.2%) b	4.3	2.1	-2.2 (-51.2%) b	21.6	22.0	0.4 (1.9%)	2.9	1.4	-1.5 (-51.7%) b
	SOUTH OF BROADWAY	21.2	19.3	-1.9 (-9.0%) b	26.9	39.9	13.0 (48.3%) b	1.1	2.0	0.9 (81.8%) b	23.8	19.9	-3.9 (-16.4%) b	0.8	1.4	0.6 (75.0%) b
	NORTH-SOUTH STREETS	17.7	18.0	0.3 (1.7%)	70.4	63.0	-7.4 (-10.5%)	2.7	3.0	0.3 (11.1%) b	26.6	20.7	-5.9 (-22.2%) b	1.9	2.1	0.2 (10.5%) a
	ALL STREETS	19.4	18.9	-0.5 (-2.6%) b	51.0	47.6	-3.4 (-6.7%) a	2.1	2.3	0.2 (9.5%) b	24.6	20.2	-4.4 (-17.9%) b	1.4	1.6	0.2 (14.3%) b
MIDDAY	EAST-WEST STREETS	20.7	19.8	-0.9 (-4.3%) b	34.1	37.6	3.5 (10.3%) a	1.5	1.9	0.4 (26.7%) b	23.9	20.5	-3.4 (-14.2%) b	1.0	1.2	0.2 (20.0%) b
	NORTH OF BROADWAY	20.8	20.4	-0.4 (-1.9%)	29.3	31.6	2.3 (7.8%)	1.2	1.7	0.5 (41.7%) b	24.6	19.5	-5.1 (-20.7%) b	0.7	1.0	0.3 (42.9%) b
	BROADWAY	17.1	17.2	0.1 (0.6%)	82.0	60.5	-21.5 (-26.2%) b	3.8	2.8	-1.0 (-26.3%) b	22.4	21.8	-0.6 (-2.7%)	2.6	1.9	-0.7 (-26.9%) b
	SOUTH OF BROADWAY	22.4	20.3	-2.1 (-9.4%) b	17.3	35.3	18.0 (104.0%) b	0.8	1.7	0.9 (112.5%) b	23.7	21.3	-2.4 (-10.1%) a	0.5	1.2	0.7 (140.0%) b
	NORTH-SOUTH STREETS	16.5	18.0	1.5 (9.1%) b	75.6	65.0	-10.6 (-14.0%) b	3.1	3.0	-0.1 (-3.2%)	23.9	21.2	-2.7 (-11.3%) b	2.2	2.1	-0.1 (-4.5%)
	ALL STREETS	19.0	19.1	0.1 (0.5%)	50.7	48.6	-2.1 (-4.1%) a	2.1	2.3	0.2 (9.5%) b	23.9	20.8	-3.1 (-13.0%) b	1.5	1.6	0.1 (6.7%) b
PM	EAST-WEST STREETS	17.7	17.0	-0.7 (-4.0%) b	66.8	75.2	8.4 (12.6%) b	2.7	3.0	0.3 (11.1%) a	25.5	25.2	-0.3 (-1.2%)	1.8	1.9	0.1 (5.6%)
	NORTH OF BROADWAY	18.8	18.7	-0.1 (-0.5%)	56.9	49.9	-7.0 (-12.3%)	2.3	2.5	0.2 (8.7%)	24.7	20.9	-3.8 (-15.4%) b	1.4	1.5	0.1 (7.1%)
	BROADWAY	14.4	14.5	0.1 (0.7%)	115.0	147.7	32.7 (28.4%) b	5.4	3.7	-1.7 (-31.5%) b	21.7	42.0	20.3 (93.5%) b	3.6	2.5	-1.1 (-30.6%) b
	SOUTH OF BROADWAY	17.6	15.6	-2.0 (-11.4%) b	57.4	76.8	19.4 (33.8%) b	2.0	3.4	1.4 (70.0%) b	28.7	23.3	-5.4 (-18.8%) b	1.4	2.3	0.9 (64.3%) b
	NORTH-SOUTH STREETS	15.8	15.3	-0.5 (-3.2%)	99.1	99.3	0.2 (0.2%)	3.7	4.3	0.6 (16.2%) b	27.1	24.2	-2.9 (-10.7%) b	2.6	3.0	0.4 (15.4%) b
	ALL STREETS	16.9	16.3	-0.6 (-3.6%) b	79.7	84.8	5.1 (6.4%) a	3.1	3.5	0.4 (12.9%) b	26.2	24.8	-1.4 (-5.3%) b	2.1	2.4	0.3 (14.3%) b

a – SIGNIFICANT AT $.10 \geq \alpha > .05$

b – SIGNIFICANT AT $\alpha \leq .05$

TABLE G-158

SPEED AND DELAY DATA, VARIANCES, EAST-WEST STREETS NORTH OF BROADWAY

TIME PERIOD	STREET	NUMBER OF OBSERVATIONS				SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		CATEGORY "A"		CATEGORY "B"																
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	F - RATIO	"BEFORE"	"AFTER"	F - RATIO	"BEFORE"	"AFTER"	F - RATIO	"BEFORE"	"AFTER"	F - RATIO	"BEFORE"	"AFTER"	F - RATIO
AM	MAIN ST.	39	7	30	7	7.7	8.3	1.08	537.7	480.6	1.12	1.0	0.2	5.00 b	64.7	88.8	1.37	0.3	0.1	3.00 a
	MARKET ST. *	42	8	18	8	7.9	3.2	2.47	647.3	386.1	1.68	0.9	1.3	1.44	50.1	14.0	3.58 b	0.3	0.4	1.33
	JEFFERSON ST.	71	8	39	6	8.7	2.5	3.48 b	475.0	152.5	3.11 a	0.6	0.4	1.50	61.2	102.8	1.68	0.2	0.1	2.00 a
	LIBERTY ST.	81	8	58	6	6.5	11.2	1.72	366.6	840.0	2.29 b	0.6	2.8	4.67 b	67.4	12.5	5.39 b	0.2	1.0	5.00 b
	WALNUT ST.	48	8	45	8	7.5	2.8	2.68 b	852.3	146.9	5.80 b	1.7	0.6	2.83 a	43.7	32.1	1.36	0.7	0.2	3.50 b
	CHESTNUT ST.	28	8	22	7	10.5	8.6	1.22	598.0	336.4	1.78	1.2	1.6	1.33	59.3	26.7	2.22	0.5	0.6	1.20
	AVERAGE					7.9	6.1	1.30	547.0	388.2	1.41 a	0.9	1.2	1.33 a	58.6	44.2	1.33	0.3	0.4	1.33
MIDDAY	MAIN ST.	30	8	22	6	7.6	9.2	1.21	358.9	338.3	1.06	0.9	0.7	1.29	52.0	176.6	3.40 b	0.3	0.2	1.50
	MARKET ST. *	29	9	26	7	7.1	2.9	2.45 a	786.3	359.9	2.18	1.2	0.8	1.50	40.8	99.3	2.43 a	0.4	0.2	2.00
	JEFFERSON ST.	68	9	25	8	5.0	2.7	1.85	170.2	71.3	2.39 a	0.3	0.5	1.67	73.6	28.2	2.61 a	0.1	0.2	2.00 a
	LIBERTY ST.	77	9	59	8	9.0	10.6	1.18	726.6	1465.2	2.02 b	1.1	2.3	2.09 b	51.1	32.8	1.56	0.4	0.8	2.00 a
	WALNUT ST.	33	7	29	7	6.9	1.2	5.75 b	363.2	308.9	1.18	0.5	1.0	2.00 a	24.8	30.3	1.22	0.2	0.4	2.00 a
	CHESTNUT ST.	15	8	13	7	12.9	12.1	1.07	1007.0	919.3	1.10	2.0	4.8	2.40 a	44.6	17.6	2.53	0.8	2.0	2.50 a
	AVERAGE					7.5	6.5	1.15	507.2	587.0	1.16	0.8	1.7	2.13 b	48.0	59.3	1.24	0.3	0.6	2.00 b
PM	MAIN ST.	30	6	27	4	8.4	10.0	1.19	691.2	284.0	2.43	1.1	0.3	3.67 a	82.2	248.3	3.02 b	0.4	0.1	4.00 a
	MARKET ST. *	38	5	35	5	12.5	1.4	8.93 b	3025.7	101.3	29.87 b	5.2	0.2	26.00 b	32.4	116.7	3.60 b	1.6	0.1	16.00 b
	JEFFERSON ST.	76	4	28	4	5.4	8.2	1.52	285.1	1423.0	4.99 b	0.5	0.3	1.67	138.5	153.4	1.11	0.2	0.1	2.00
	LIBERTY ST.	79	6	73	5	10.5	11.2	1.07	1272.6	1738.7	1.37	2.3	3.0	1.30	35.2	9.6	3.67 a	0.8	1.0	1.25
	WALNUT ST.	36	5	35	5	13.1	0.3	43.67 b	2990.1	21.5	139.07 b	4.1	0.3	13.67 b	58.2	58.5	1.01	1.7	0.1	17.00 b
	CHESTNUT ST.	21	6	21	6	12.7	5.0	2.54	4285.7	2329.4	1.84	7.6	6.2	1.23	53.9	7.9	6.82 b	3.1	2.5	1.24
	AVERAGE					9.6	6.2	1.55 a	1616.8	1020.0	1.59	2.7	1.9	1.42	59.0	86.2	1.46	1.0	0.7	1.43

a – SIGNIFICANT AT $.10 \geq \alpha > .05$

b – SIGNIFICANT AT $\alpha \leq .05$

* EASTBOUND DIRECTION ONLY

CATEGORY "A" – NUMBER OF OBSERVATIONS FOR SPEED, DELAY TIME, NUMBER OF STOPS AND STOPS PER MILE

CATEGORY "B" – NUMBER OF OBSERVATIONS FOR DELAY TIME PER STOP

TABLE G-159

SPEED AND DELAY DATA, VARIANCES, BROADWAY

TIME PERIOD	DIRECTION	NUMBER OF OBSERVATIONS				SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		CATEGORY "A"		CATEGORY "B"																
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO
AM	EASTBOUND	30	16	30	10	8.4	12.7	1.51	965.0	1381.8	1.43	1.0	1.9	1.90 a	42.6	125.5	2.95 b	0.4	0.8	2.00 a
	WESTBOUND	28	16	28	16	9.3	2.7	3.44 b	4643.2	447.9	10.37 b	5.8	1.0	5.80 b	20.7	19.1	1.08	2.6	0.4	6.50 b
	AVERAGE					8.8	6.5	1.35	2738.4	914.9	2.99 b	3.3	1.4	2.36 b	32.1	59.0	1.84 b	1.5	0.6	2.50 b
MIDDAY	EASTBOUND	15	18	14	10	9.7	11.4	1.18	1161.3	1387.3	1.19	2.4	2.9	1.21	55.1	11.3	4.88 b	1.1	1.3	1.18
	WESTBOUND	14	18	14	18	4.2	5.6	1.33	1034.9	1108.1	1.07	3.5	1.4	2.50 b	27.7	36.4	1.31	1.6	0.6	2.67 b
	AVERAGE					7.1	8.5	1.20	1100.4	1247.7	1.13	2.9	2.1	1.38	41.4	27.8	1.49	1.3	1.0	1.30
PM	EASTBOUND	20	7	20	7	3.3	11.2	3.39 b	1106.2	1798.6	1.63	2.4	2.5	1.04	15.5	189.5	12.23 b	1.1	1.1	1.00
	WESTBOUND	21	6	21	6	6.5	0.3	21.67 b	2611.5	372.3	7.01 b	4.8	1.1	4.36 a	38.6	69.0	1.79	2.2	0.5	4.40 a
	AVERAGE					5.0	6.3	1.26	1878.1	1150.3	1.63	3.6	1.8	2.00	27.3	134.7	4.93 b	1.6	0.8	2.00

a — SIGNIFICANT AT $.10 \geq \alpha > .05$

b — SIGNIFICANT AT $\alpha \leq .05$

CATEGORY "A" – NUMBER OF OBSERVATIONS FOR SPEED, DELAY TIME, NUMBER OF STOPS AND STOPS PER MILE

CATEGORY "B" – NUMBER OF OBSERVATIONS FOR DELAY TIME PER STOP

TABLE G-160

SPEED AND DELAY DATA, VARIANCES, EAST-WEST STREETS SOUTH OF BROADWAY

TIME PERIOD	STREET	NUMBER OF OBSERVATIONS				SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		CATEGORY "A"		CATEGORY "B"																
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	BEFORE	"AFTER"	F-RATIO
AM	BRECKINRIDGE ST.	40	14	29	13	14.6	7.3	2.00 a	848.4	486.0	1.75	1.4	1.8	1.29	77.7	8.5	9.14 b	0.7	0.9	1.29
	KENTUCKY ST.	23	13	12	12	6.6	4.3	1.53	157.2	204.5	1.30	0.3	0.7	2.33 b	131.6	10.3	12.78 b	0.2	0.3	1.50
	ST. CATHERINE ST.	25	15	20	15	10.3	8.8	1.17	846.1	1109.7	1.31	1.7	2.6	1.53	35.5	8.5	4.18 b	0.7	1.2	1.71
	OAK ST.	24	13	18	11	10.7	12.7	1.19	453.6	454.6	1.00	0.6	0.8	1.33	72.8	24.8	2.94	0.3	0.4	1.33
	AVERAGE					11.2	8.3	1.35	623.0	583.6	1.07	1.1	1.5	1.36 a	73.8	12.4	5.95 b	0.5	0.7	1.40 a
MIDDAY	BRECKINRIDGE ST.	41	16	24	15	9.5	6.8	1.40	515.1	310.5	1.66	1.4	1.3	1.08	53.5	31.2	1.71	0.7	0.6	1.17
	KENTUCKY ST.	25	16	12	13	8.2	8.9	1.09	229.9	494.3	2.15 b	0.3	1.1	3.67 b	192.8	35.5	5.43 b	0.1	0.5	5.00 b
	ST. CATHERINE ST.	25	16	11	16	5.5	5.6	1.02	209.6	495.5	2.36 b	0.3	0.9	3.00 b	85.6	17.0	5.04 b	0.2	0.2	2.00 a
	OAK ST.	20	15	15	14	8.0	6.3	1.27	314.7	215.8	1.46	0.5	0.4	1.25	14.7	8.7	1.69	0.2	0.2	1.00
	AVERAGE					8.0	6.9	1.16	347.0	381.8	1.10	0.8	0.9	1.13	76.1	22.8	3.34 b	0.4	0.4	1.00
PM	BRECKINRIDGE ST.	30	12	29	12	6.8	4.0	1.70	1083.0	503.9	2.15 a	1.7	0.9	1.89	58.7	33.9	1.73	0.8	0.4	2.00
	KENTUCKY ST.	22	13	19	13	15.4	9.2	1.67	2906.1	1089.6	2.67 b	4.6	2.1	2.19 a	8.9	19.1	2.15 a	2.1	0.9	2.33 a
	ST. CATHERINE ST.	21	11	21	11	5.2	3.5	1.49	883.2	1567.7	1.78	1.6	4.8	3.00 b	21.2	22.7	1.07	0.7	2.1	3.00 b
	OAK ST.	19	11	18	11	11.8	11.0	1.07	1343.4	4593.8	3.42 b	1.1	5.8	5.27 b	11.2	47.7	4.26 b	0.6	3.0	5.00 b
	AVERAGE					9.5	7.0	1.36	1526.4	1865.9	1.22	2.2	3.3	1.50 a	29.2	30.4	1.04	1.0	1.6	1.60 b

a — SIGNIFICANT AT $.10 \geq \alpha > .05$

b — SIGNIFICANT AT $\alpha \leq .05$

CATEGORY "A" – NUMBER OF OBSERVATIONS FOR SPEED, DELAY TIME, NUMBER OF STOPS AND STOPS PER MILE

CATEGORY "B" – NUMBER OF OBSERVATIONS FOR DELAY TIME PER STOP

TABLE G-161

SPEED AND DELAY DATA, VARIANCES, NORTH-SOUTH STREETS

TIME PERIOD	STREET	NUMBER OF OBSERVATIONS				SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		CATEGORY "A"		CATEGORY "B"																
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO
AM	BROOK ST.	26	11	25	11	5.6	3.4	1.65	635.9	205.5	3.09 b	0.5	0.6	1.20	171.0	29.8	5.74 b	0.2	0.3	1.50
	1st. ST.	26	11	26	11	9.0	6.2	1.45	1176.1	758.2	1.55	1.5	1.6	1.07	33.3	24.7	1.35	0.6	0.7	1.17
	2nd. ST.	27	9	27	6	8.4	17.5	2.08	877.8	1311.5	1.49	1.0	2.0	2.00 a	109.3	18.4	5.94 b	0.8	1.6	2.00 a
	3rd. ST.	23	10	23	10	5.5	11.6	2.11 a	1401.0	1186.0	1.18	2.3	2.0	1.15	44.1	19.4	2.27	1.0	0.9	1.11
	4th. ST. *	27	9	27	9	4.8	2.4	2.00	1992.2	1252.8	1.59	2.6	1.4	1.86	69.1	23.9	2.89 a	1.1	0.6	1.83
	5th. ST.	22	9	21	9	8.2	3.9	2.10	1023.4	784.8	1.30	1.6	0.8	2.00	78.1	30.8	2.54 a	1.1	0.5	2.20
	6th. ST.	41	9	41	8	9.3	26.9	2.89 b	1617.3	2744.0	1.70	2.0	4.8	2.40 b	55.4	21.7	2.55 a	0.9	2.2	2.44 b
	7th. ST.	35	9	34	9	7.8	11.6	1.49	770.6	1252.4	1.63	0.9	1.3	1.44	37.5	34.4	1.09	0.4	0.6	1.50
	AVERAGE					7.5	10.1	1.35 a	1201.5	1146.0	1.05	1.6	1.8	1.13	72.0	25.8	2.79 b	0.8	0.9	1.13
MIDDAY	BROOK ST.	14	10	14	10	3.6	4.6	1.28	355.9	210.2	1.69	0.6	0.9	1.50	53.5	16.5	3.24 b	0.3	0.4	1.33
	1st. ST.	15	10	15	9	5.7	14.6	2.56 a	453.6	725.4	1.60	0.5	1.3	2.60 a	31.4	35.9	1.14	0.2	0.6	3.00 b
	2nd. ST.	15	10	14	8	12.9	7.1	1.82	1067.9	457.1	2.34	1.4	0.4	3.50 b	50.3	58.3	1.16	1.1	0.4	2.75 a
	3rd. ST.	13	8	13	8	1.3	4.4	3.38 b	387.9	373.6	1.04	0.5	0.2	2.50	16.1	36.9	2.29 a	0.2	0.1	2.00
	4th. ST. *	15	13	15	13	4.8	2.4	2.00	3007.8	1765.4	1.70	4.7	2.2	2.14 a	25.9	18.0	1.44	2.1	1.0	2.10
	5th. ST.	15	12	15	12	4.7	14.4	3.06 b	1094.1	1155.2	1.06	2.0	0.7	2.86 b	24.6	49.5	2.01	1.4	0.5	2.80 b
	6th. ST.	15	12	15	12	3.8	12.1	3.18 b	1008.6	651.0	1.55	1.8	1.1	1.64	16.2	54.6	3.37 b	0.8	0.5	1.60
	7th. ST.	15	10	15	10	4.3	10.2	2.37 a	1352.1	1408.0	1.04	1.1	1.4	1.27	42.8	45.8	1.07	0.5	0.6	1.20
	AVERAGE					5.2	8.8	1.69 b	1110.6	894.5	1.24	1.6	1.1	1.45 a	32.6	38.9	1.19	0.8	0.5	1.60 b
PM	BROOK ST.	27	10	25	7	9.3	8.6	1.08	832.0	1340.4	1.61	0.8	2.0	2.50 b	94.2	63.1	1.49	0.3	0.9	3.00 b
	1st. ST.	25	9	24	9	8.3	12.2	1.47	2660.3	983.3	2.71 a	3.3	2.3	1.43	40.3	39.6	1.02	1.5	1.0	1.50
	2nd. ST.	25	8	25	8	7.3	2.7	2.70 a	1040.1	365.6	2.84 a	0.8	0.4	2.00	123.3	75.1	1.64	0.7	0.3	2.33
	3rd. ST.	23	7	23	5	5.9	33.6	5.69 b	2591.7	5150.1	1.99	3.4	11.6	3.41 b	30.4	38.5	1.27	1.5	5.1	3.40 b
	4th. ST. *	22	7	22	7	4.0	1.6	2.50	4691.0	1804.3	2.60	4.2	2.2	1.91	58.6	8.1	7.23 b	1.9	1.0	1.90
	5th. ST.	19	4	19	4	6.6	6.2	1.06	1679.8	1237.7	1.36	1.5	1.6	1.07	58.4	51.0	1.15	1.1	1.1	1.00
	6th. ST.	24	6	24	6	14.4	4.5	3.20 a	15998.3	5483.1	2.92	11.7	12.7	1.09	48.3	34.0	1.42	5.3	5.7	1.08
	7th. ST.	24	6	24	6	4.5	10.3	2.29 a	739.2	2986.7	4.04 b	0.9	3.5	3.89 b	43.3	32.5	1.33	0.4	1.5	3.75 b
	AVERAGE					7.7	10.2	1.32 a	3762.1	2250.6	1.67 b	3.3	4.2	1.27 a	62.9	43.4	1.45 a	1.6	1.9	1.18

a — SIGNIFICANT AT $.10 \geq \alpha > .05$

b — SIGNIFICANT AT $\alpha \leq .05$

* — NORTHBOUND DIRECTION ONLY

CATEGORY "A" – NUMBER OF OBSERVATIONS FOR SPEED, DELAY TIME, NUMBER OF STOPS AND STOPS PER MILE

CATEGORY "B" – NUMBER OF OBSERVATIONS FOR DELAY TIME PER STOP

TABLE G-162

SPEED AND DELAY DATA, VARIANCES, SUMMARY

TIME PERIOD	STREET	NUMBER OF OBSERVATIONS				SPEED (MPH)			DELAY TIME (SECONDS)			NUMBER OF STOPS			DELAY TIME PER STOP (SECONDS/STOP)			STOPS PER MILE		
		CATEGORY "A"		CATEGORY "B"																
		"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO	"BEFORE"	"AFTER"	F-RATIO
AM	EAST-WEST STREETS	479	134	349	119	8.8	7.4	1.19	827.4	599.4	1.38	1.2	1.4	1.17	57.5	33.6	1.71	0.5	0.6	1.20
	NORTH OF BROADWAY	309	47	212	42	7.9	6.1	1.30	547.0	388.2	1.41 a	0.9	1.2	1.33 a	58.6	44.2	1.33	0.3	0.4	1.33
	BROADWAY	58	32	58	26	8.8	6.5	1.35	2738.4	914.9	2.99 b	3.3	1.4	2.36 b	32.1	59.0	1.84 b	1.5	0.6	2.50 b
	SOUTH OF BROADWAY	112	55	79	51	11.2	8.3	1.35	623.0	583.6	1.07	1.1	1.5	1.36 a	73.8	12.4	5.95 b	0.5	0.7	1.40 a
	NORTH-SOUTH STREETS	227	77	224	73	7.5	10.1	1.35 a	1201.5	1146.0	1.05	1.6	1.8	1.13	72.0	25.8	2.79 b	0.8	0.9	1.13
	ALL STREETS	706	211	573	192	8.3	8.4	1.01	946.8	796.8	1.19	1.3	1.5	1.15	63.2	30.6	2.07 b	0.6	0.7	1.17
MIDDAY	EAST-WEST STREETS	392	149	264	129	7.6	7.2	1.06	504.3	662.6	1.31	1.0	1.5	1.50	53.8	35.4	1.52	0.4	0.6	1.50
	NORTH OF BROADWAY	252	50	174	43	7.5	6.5	1.15	507.2	587.0	1.16	0.8	1.7	2.13 b	48.0	59.3	1.24	0.3	0.6	2.00 b
	BROADWAY	29	36	28	28	7.1	8.5	1.20	1100.4	1247.7	1.13	2.9	2.1	1.38	41.4	27.8	1.49	1.3	1.0	1.30
	SOUTH OF BROADWAY	111	63	62	58	8.0	6.9	1.16	347.0	381.8	1.10	0.8	0.9	1.13	76.1	22.8	3.34 b	0.4	0.4	1.00
	NORTH-SOUTH STREETS	117	85	116	82	5.2	8.8	1.69 b	1110.6	894.5	1.24	1.6	1.1	1.45 a	32.6	38.9	1.19	0.8	0.5	1.60 b
	ALL STREETS	509	234	380	211	7.1	7.8	1.10	639.4	746.0	1.17	1.1	1.3	1.18	47.4	36.8	1.29	0.5	0.6	1.20
PM	EAST-WEST STREETS	413	92	347	89	9.1	6.6	1.38	1622.4	1492.6	1.09	2.7	2.6	1.04	47.9	62.0	1.29	1.0	1.2	1.20
	NORTH OF BROADWAY	280	32	219	29	9.6	6.2	1.55 a	1616.8	1020.0	1.59	2.7	1.9	1.42	59.0	86.2	1.46	1.0	0.7	1.43
	BROADWAY	41	13	41	13	5.0	6.3	1.26	1878.1	1150.3	1.63	3.6	1.8	2.00	27.3	134.7	4.93 b	1.6	0.8	2.00
	SOUTH OF BROADWAY	92	47	87	47	9.5	7.0	1.36	1526.4	1865.9	1.22	2.2	3.3	1.50 a	29.2	30.4	1.04	1.0	1.6	1.60 b
	NORTH-SOUTH STREETS	189	57	186	52	7.7	10.2	1.32 a	3762.1	2250.6	1.67 b	3.3	4.2	1.27 a	62.9	43.4	1.45 a	1.6	1.9	1.19
	ALL STREETS	602	149	533	141	8.7	8.0	1.09	1172.8	1780.5	1.52	2.9	3.2	1.10	53.1	55.2	1.04	1.2	1.5	1.25

a – SIGNIFICANT AT $.10 \geq \alpha > .05$

b – SIGNIFICANT AT $\alpha \leq .05$

CATEGORY "A" – NUMBER OF OBSERVATIONS FOR SPEED, DELAY TIME, NUMBER OF STOPS AND STOPS PER MILE

CATEGORY "B" – NUMBER OF OBSERVATIONS FOR DELAY TIME PER STOP

TABLE G-164

TOTAL TRAVEL DURING PEAK HOURS—INDIVIDUAL STREETS

AREA	PEAK HOUR	STREET	DIFFERENCE BETWEEN "BEFORE" AND "AFTER" CONDITIONS: DELAY TIME (VEH-HR)	VEHICLE STOPS	VEHICLE STOPS/MILE
East-west streets north of Broadway	AM	Main St.	−1.4	0	−130
		Market St. EB	5.2	1,190	680
		Jefferson St.	−1.5	260	130
		Liberty St.	3.4	840	480
		Walnut St.	−1.9	90	90
		Chestnut St.	1.7	475	285
		All	5.5	2,855	1,535
	PM	Main St.	−6.0	−1,100	−550
		Market St. EB	−22.1	−2,530	−1,430
		Jefferson St.	12.8	2,380	1,275
		Liberty St.	−0.4	−120	0
		Walnut St.	−5.7	−630	−420
		Chestnut St.	8.3	2,990	1,840
		All	−13.1	990	715
Broadway	AM	Broadway EB	−8.3	−1,530	−1,020
		Broadway WB	−20.4	−3,250	−2,250
		All	−28.7	−4,780	−3,270
	PM	Broadway EB	−16.1	−5,425	−3,720
		Broadway WB	20.0	0	70
		All	3.9	−5,425	−3,650
East-west streets south of Broadway	AM	Breckinridge St.	2.3	770	440
		Kentucky St.	1.7	480	320
		St. Catherine St.	5.6	1,380	920
		Oak St.	1.7	275	220
		All	11.3	2,905	1,900
	PM	Breckinridge St.	−2.2	480	160
		Kentucky St.	2.3	840	60
		St. Catherine St.	12.7	2,720	1,870
		Oak St.	8.9	1,500	1,125
		All	21.7	5,540	3,215
North-south streets	AM	Brook St.	−2.6	225	150
		First St.	0.2	825	600
		Second St.	−11.7	−1,300	−1,170
		Third St.	−3.8	−90	−90
		Fourth St.	0.6	405	270
		Fifth St.	0.4	520	390
		Sixth St.	−1.9	700	400
		Seventh St.	0	−65	0
		All	−18.8	1,220	550
	PM	Brook St.	−3.0	−210	−140
		First St.	2.9	650	520
		Second St.	−6.8	−720	−600
		Third St.	−15.7	−1,320	−840
		Fourth St.	5.5	920	600
		Fifth St.	0.0	80	0
		Sixth St.	2.7	2,625	1,785
		Seventh St.	9.7	1,785	1,155
		All	−4.7	3,810	2,480

TABLE G-165

SUMMARY OF TOTAL TRAVEL DURING PEAK HOURS

PEAK HOUR	AREA	DIFFERENCE BETWEEN "BEFORE" AND "AFTER" CONDITIONS: DELAY TIME (VEH-HR)	VEHICLE STOPS	VEHICLE STOPS/MILE
AM	East-west streets	−11.9	980	165
	North of Broadway	5.5	2,855	1,535
	Broadway	−28.7	−4,780	−3,270
	South of Broadway	11.3	2,905	1,900
	North-south streets	−18.8	1,220	550
	All	−30.7	2,200	715
PM	East-west streets	12.5	1,105	280
	North of Broadway	−13.1	990	715
	Broadway	3.9	−5,425	−3,650
	South of Broadway	21.7	5,540	3,215
	North-south streets	−4.7	3,810	2,480
	All	7.8	4,915	2,760

by 30.7 veh-hr, vehicle stops increased by 2,200, and vehicle stops per mile increased by 715.

2. During the midday time period, the number of stops increased by 0.2 per vehicle (9.5 percent), delay time per stop decreased by 3.1 sec per vehicle (13.0 percent), and stops per mile increased by 0.1 per vehicle (6.7 percent).

3. During the PM time period, speed was reduced by 0.6 mph (3.6 percent), number of stops increased by 0.4 per vehicle (12.9 percent), delay time per stop decreased by 1.4 sec per vehicle (5.3 percent), and stops per mile increased by 0.3 per vehicle (14.3 percent). In terms of total travel during the PM peak hour, delay time increased by 7.8 veh-hr, vehicle stops increased by 4,915, and vehicle stops per mile increased by 2,760.

4. The variability of the "before" and "after" measurements for delay time per stop decreased significantly for all arterials during the AM time period.

TABLE G-166

SURVEY DATES AND WEATHER CONDITIONS

DATE (1969)	DAY	WEATHER CONDITION: AM PEAK	MORNING MIDDAY	PM PEAK
2/28	Fri.	Clear	Clear	Snow
3/11	Tues.	Ice	Rain	Clear
3/24	Mon.	Rain	Rain	Rain
3/25	Tues.	Rain	Rain	Rain
5/19	Mon.	Rain	Rain	Clear
5/20	Tues.	Clear	Clear	Clear
6/9	Mon.	Clear	Clear	Clear

INCLEMENT WEATHER EFFECTS

Inclement Weather Effects—Experiment E69

Experiment E69 studies the effects of inclement weather on traffic characteristics on Walnut Street and Liberty Street in Louisville (Fig. G-179). Both of these streets are major traffic carriers during the peak and off-peak hours.

Experimental Area

Walnut Street is a 36-ft-wide one-way arterial street providing for a maximum of four lanes of traffic movement in a westbound direction. Parking is permitted at all times between Chestnut Street and Floyd Street and prohibited on weekdays and Saturdays between Floyd Street and Seventh Street. Stopping is not permitted on both sides of the street in this latter section from 7 to 9 AM and 4 to 6 PM. Between Seventh Street and Ninth Street, parking is permitted at all times on the south side of Walnut Street and between 9 AM and 4 PM on the north side. Stopping is prohibited from 7 to 9 AM and 4 to 6 PM on the north side. Average weekday traffic volumes in 1968 on Walnut Street were about 6,000 vehicles between Chestnut Street and Floyd Street and ranged from 8,000 to 10,000 vehicles between Floyd Street and Ninth Street. The greatest traffic hour generally occurs during the morning peak hour.

Liberty Street is a four-lane roadway, generally 36 ft wide, permitting travel in an eastbound direction. Some sections in urban renewal areas have been widened to widths of between 40 and 56 ft. Between Ninth Street and Eighth Street, parking is permitted at all times on the north side of the street; stopping is permitted on the south side between 7 AM and 6 PM. Stopping is not permitted from 7 AM to 6 PM on both sides of the street between Eighth Street and Fifth Street; on the south side of the street between Fifth Street and Third Street; and on both sides of the street between Third Street and Brook Street. On the north side of the street between Fifth Street and Third Street, parking is not permitted between 7 AM and 4 PM; stopping is prohibited between 4 and 6 PM. East of Brook Street, parking is permitted on both sides of the street, except that stopping is prohibited on the south side between 4 and 6 PM. Average weekday traffic volumes in 1968 on Liberty Street ranged from 11,000 to 13,000 vehicles between Ninth Street and the I-65 on-ramp between Floyd Street and Preston Street. East of the latter location, traffic was about 5,000 vehicles per weekday. The greatest traffic occurs during the evening peak hour.

The traffic signals on Walnut Street and Liberty Street are part of an interconnected system operating on a 60-sec cycle. Time-space diagrams for both of these streets are shown in Figures G-180 and G-181.

Design of Experiment

Because experiment E69 did not involve any improvements and depended on obtaining traffic measurements during different types of weather, the surveillance system was designed so that counts could be taken on short notice. It was decided to limit the surveillance system to speed and delay runs and vehicle counts at selected intersections, as follows:

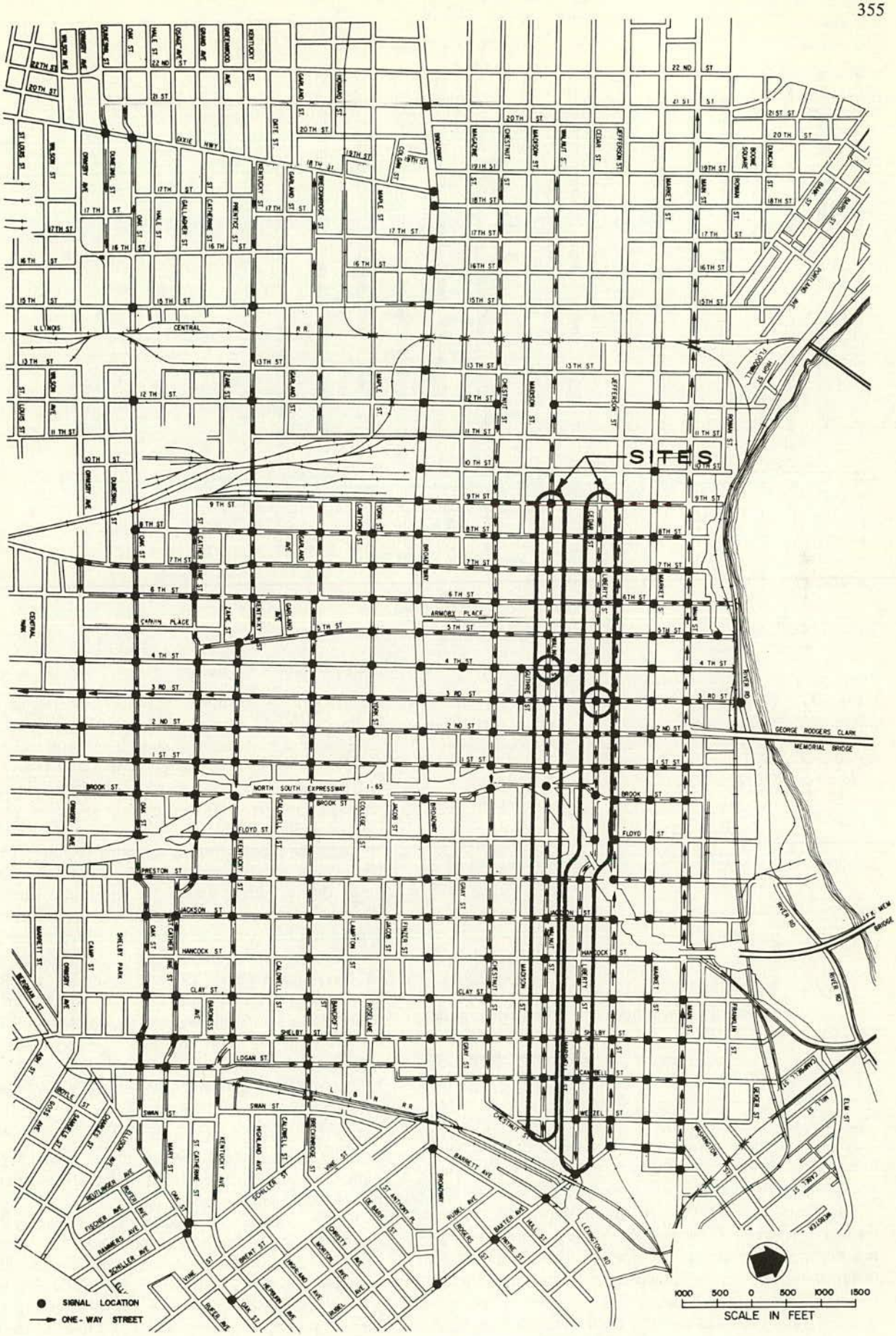


Figure G-179. Location map, Experiment E69.

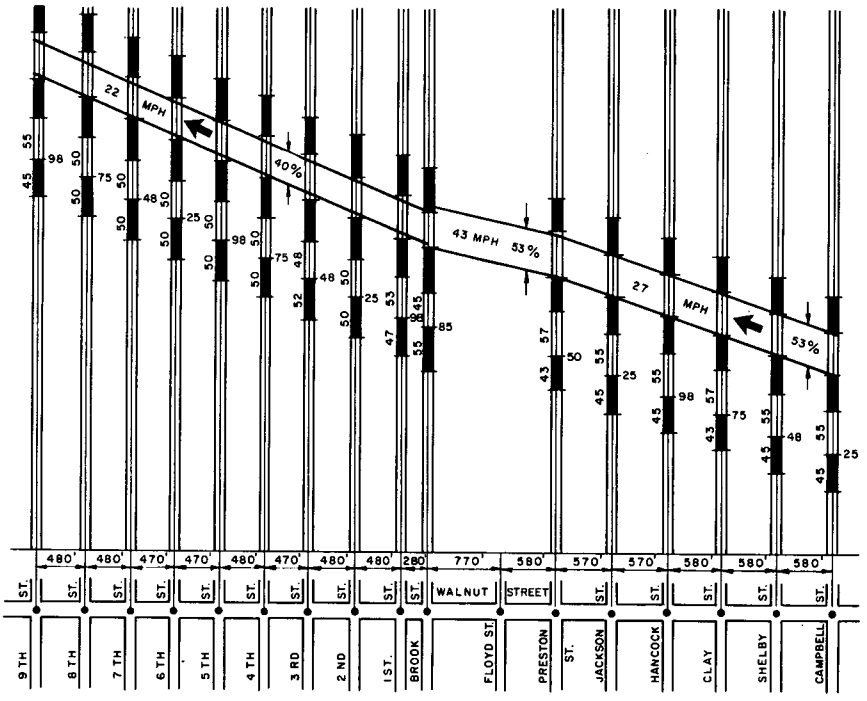


Figure G-180. Time-space diagram, Walnut Street.

TYPE OF MEASUREMENT	LOCATION	INFORMATION RECORDED
Speed and delay runs	Walnut St. (Cooper St. to Ninth St.)	Travel time Delay time Number of stops
	Liberty St. (Ninth St. to Baxter Ave.)	Travel time Delay time Number of stops
Vehicle counts	Walnut St. approach to Fourth St.	Vehicles through per cycle Vehicles stopped per cycle
	Liberty St. approach to Third St.	Vehicles through per cycle Vehicles stopped per cycle

ATR equipment counts were not taken because variations in traffic volume could be determined from the vehicle count measurements.

The dates and weather conditions when counts were taken are given in Table G-166. Surveys were taken during one snow condition and one ice condition; however, insufficient data were obtained for a proper analysis.

Additional speed and delay data recorded on weekdays between June 1968 and January 1969 were also used in this study.

Analysis and Conclusions

Analysis of speed and delay data indicated no significant difference in the mean values and variances of travel time, delay time, and number of stops between the clear and rain conditions. A summary is given in Table G-167.

There was no difference in the number of vehicles through per cycle between the rain and clear weather conditions, indicating that the same number of vehicles passed through the intersection in the same manner in both cases. There was also no difference in the number of vehicles stopped per cycle between the clear and rain conditions during the AM period at the Liberty Street approach and during the PM period at the Walnut Street approach. Differences were observed during the other time periods at both of these approaches (Tables G-168 and G-169). Table G-168 gives the individual values for the means and variances; Table G-169 gives grand means and pooled variances for the clear and rain conditions. Analysis of this latter table indicated that means and pooled variance for the rain category were significantly greater than the clear category for each of the four time periods.

Based on the foregoing analysis, it is concluded that:

1. There was no difference in travel time, delay time, and number of stops between rain and clear weather as determined from speed and delay data.
2. The number of vehicles through per cycle did not change at the two approaches studied.
3. The mean values and variances were greater for rainy weather than for clear weather at the following approaches for the time periods indicated: (1) Walnut Street at Fourth Street (AM and midday), and (2) Liberty Street at Third Street (midday and PM).

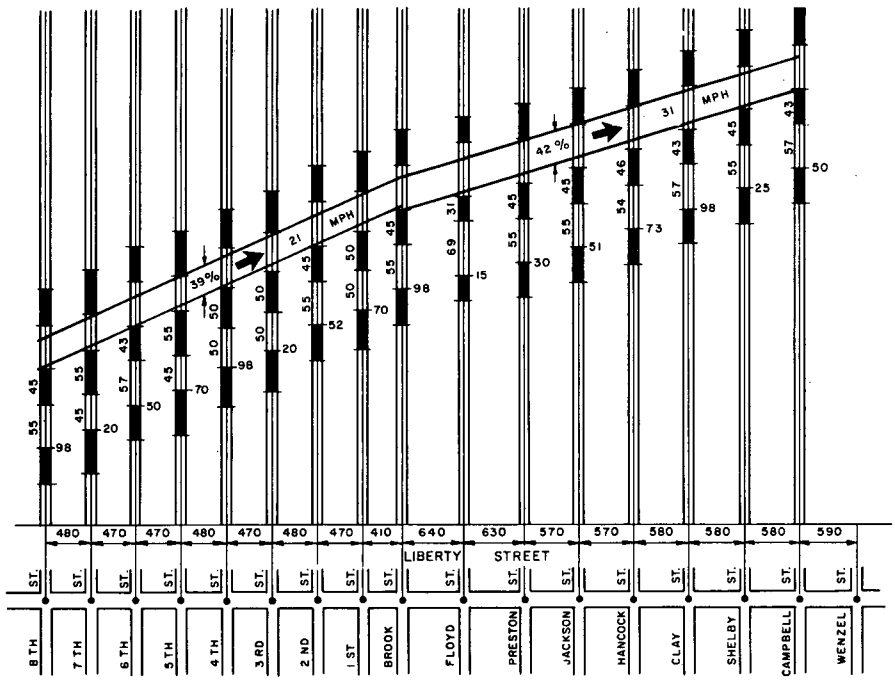


Figure G-181. Time-space diagram, Liberty Street.

Inclement Weather Effects—Experiment B99

Experiment B99 studies the effect of inclement weather on travel characteristics in Newark. Speed and delay runs were made on McCarter Highway, a major north-south arterial route through the city, on days when the pavement was dry and days when the pavement was wet. This experiment was designed to measure any changes in travel time, delay time, or number of stops caused by the wet pavement and/or reduced visibility that occurred during the inclement weather.

Experimental Area

McCarter Highway (N.J. 21) is an at-grade arterial route that connects N.J. 1, 9, and 22 on the south with N.J. 21 Freeway on the north (Fig. G-182). Traffic volumes range

TABLE G-167

SUMMARY OF SPEED AND DELAY ANALYSIS

STREET	TIME PERIOD	VARIABLE	MEAN VALUE CLEAR	MEAN VALUE RAIN	VARIANCE CLEAR	VARIANCE RAIN	NO. OF OBS. CLEAR	NO. OF OBS. RAIN
Walnut	AM	Travel time (sec)	416.7	436.3	4955.7	4538.4	27	20
		Delay time (sec)	84.4	87.1	2130.2	1707.7		
		No. of stops	2.8	3.2	1.7	2.6		
	Midday	Travel time (sec)	403.6	408.2	2093.1	2848.4	17	17
		Delay time (sec)	69.4	65.1	883.3	1100.0		
		No. of stops	2.5	2.4	1.0	1.1		
	PM	Travel time (sec)	443.9	430.2	8633.6	21700.6	29	6
		Delay time (sec)	93.7	81.8	4256.2	7425.4		
		No. of stops	3.4	3.2	4.8	10.6		
Liberty	AM	Travel time (sec)	326.9	339.6	1902.6	1954.6	63	19
		Delay time (sec)	47.2	48.4	1181.4	786.9		
		No. of stops	1.5	1.4	0.8	0.6		
	Midday	Travel time (sec)	352.8	351.0	4435.8	4270.0	64	12
		Delay time (sec)	61.0	55.2	2185.0	1961.4		
		No. of stops	2.0	1.7	2.1	1.5		
	PM	Travel time (sec)	398.6	375.5	5156.0	7028.3	78	6
		Delay time (sec)	85.2	64.0	2354.5	1943.2		
		No. of stops	3.2	2.5	2.8	5.1		

TABLE G-168

VEHICLES STOPPED PER CYCLE, DAILY VALUES

APPROACH TO	TIME PERIOD	STATISTIC	VEHICLES STOPPED PER CYCLE, BY DAY AND DATE (1969) MON. 3/24	TUES. 3/25	MON. 5/19	TUES. 5/20	MON. 6/9
Walnut St.	AM	Mean	6.1 (R)	7.8 (R)	10.5 (R)	6.8 (C)	4.4 (C)
		Variance	15.5 (R)	25.7 (R)	42.2 (R)	14.3 (C)	6.9 (C)
	Midday	Mean	5.2 (R)	4.5 (R)	6.5 (R)	4.7 (C)	4.8 (C)
		Variance	4.8 (R)	6.0 (R)	10.0 (R)	5.5 (C)	5.6 (C)
Liberty St.	Midday	Mean	6.1 (R)	4.2 (R)	6.3 (R)	3.8 (C)	4.3 (C)
		Variance	8.9 (R)	5.2 (R)	16.4 (R)	6.3 (C)	4.8 (C)
	PM	Mean	9.4 (C)	19.4 (R)	11.9 (C)	10.0 (C)	9.2 (C)
		Variance	19.0 (C)	71.9 (R)	48.1 (C)	22.7 (C)	17.8 (C)

(R) = rain; (C) = clear.

TABLE G-169

SUMMARY OF VEHICLES STOPPED PER CYCLE

APPROACH TO	TIME PERIOD	MEAN (VEH) CLEAR	RAIN	DIFFERENCE, RAIN-CLEAR	SIG. @ α LEVEL	VARIANCE (VEH) CLEAR	RAIN	RAIN/CLEAR	SIG. @ α LEVEL
Walnut St.	AM	5.6	8.1	+2.5(44.6%)	0.0005	10.6	27.8	2.62	0.0005
	Midday	4.8	5.4	+0.6(12.5%)	0.025	5.6	7.0	1.25	0.05
Liberty St.	Midday	4.0	5.5	+1.5(37.5%)	0.0005	5.6	10.2	1.82	0.0005
	PM	10.1	19.4	+9.3(92.1%)	0.0005	26.9	71.9	2.67	0.0005

from 20,000 to 30,000 vpd. During the morning and evening peak hours, traffic flow is highly directional, with peak flows inbound from both ends of the facility in the morning and outbound in the evening. Turning traffic in the midsection of the area (Fig. G-183) reduces the highly directional nature of the traffic flows so that the directional flows are approximately equal in that area. Figure G-184 shows the average peak traffic volumes and the number of lanes in each section of roadway.

Parking is prohibited on both sides of the roadway at all times; standing and stopping is prohibited on both sides during the morning peak hour of 7:00 to 9:30 and the evening peak hour of 4:00 to 6:00. Twenty-nine intersections along McCarter Highway are controlled by traffic signals which are part of the PR system in downtown Newark.

Experimental Design

No changes were made in traffic control systems or regulations for Experiment B99, because only the differences in traffic characteristics between wet and dry weather were desired. Speed and delay runs were made on McCarter Highway during the morning and evening peak hours during both fair and inclement weather. A sufficient sample of rain runs was obtained only during the morning peak hours. Speed and delay runs were made during the period from October 10, 1967, to May 20, 1969.

Analysis and Conclusions

Table G-170 gives the "before" and "after" mean values for travel time, delay time, and number of stops as summarized from the speed and delay run data for the AM period. Significant increases at the $\alpha = 0.025$ level were determined for the "before" and "after" mean values for all variables recorded during the southbound speed and delay runs. The significance level was greater and the percentage difference was less for similar statistics for the northbound direction.

Based on the analysis of these data, it is concluded that travel time, delay time, and the number of stops per run are greater during rainy weather than during fair weather (Table G-170).

BUS OPERATION

Bus Transit—Passenger Service Operations—Experiment F63

Reasonably accurate predictions of bus route time can be made if sufficient factual data are available describing three broad elements of bus transit service. These elements are

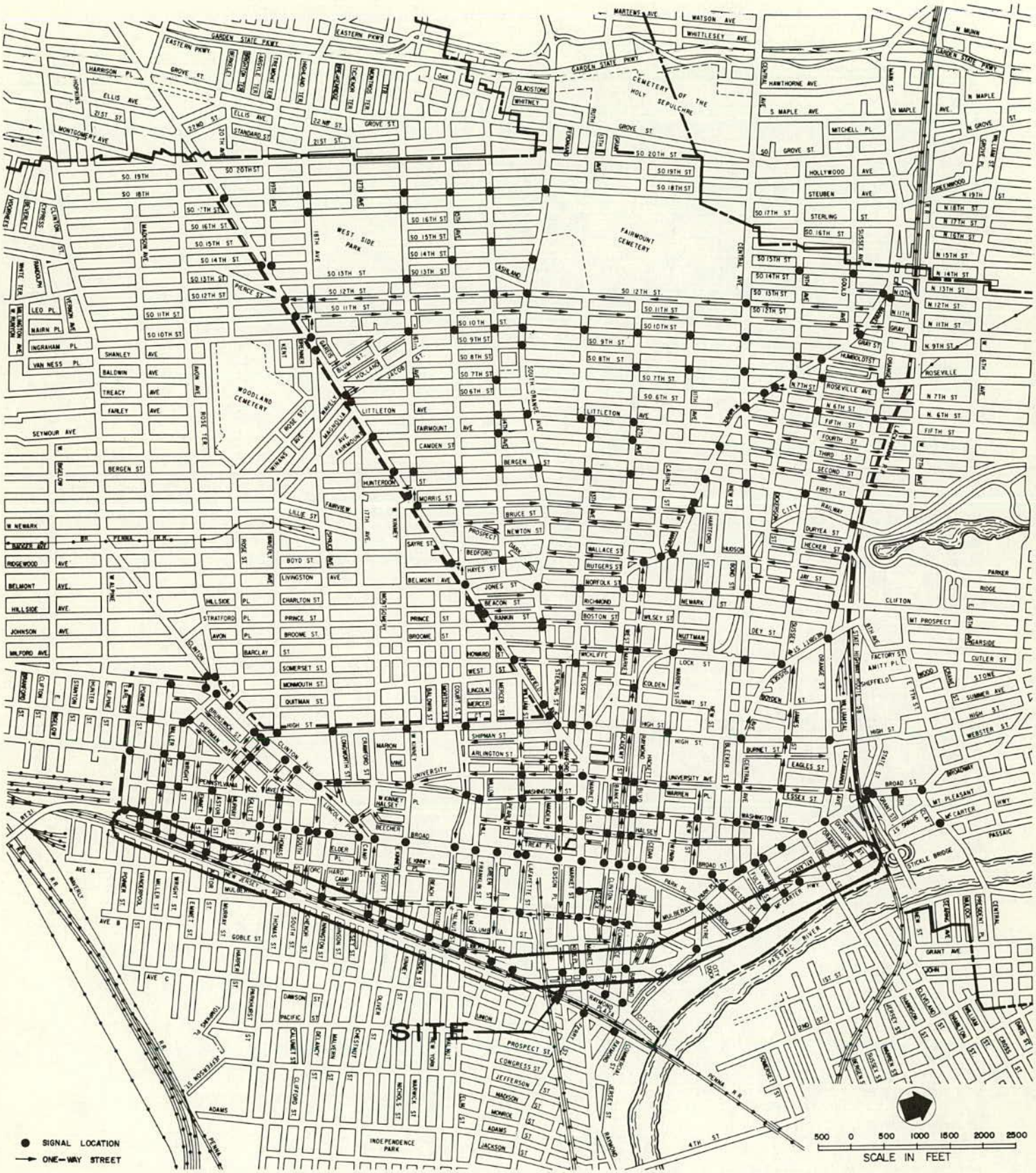


Figure G-182. Location map, Experiment B99.

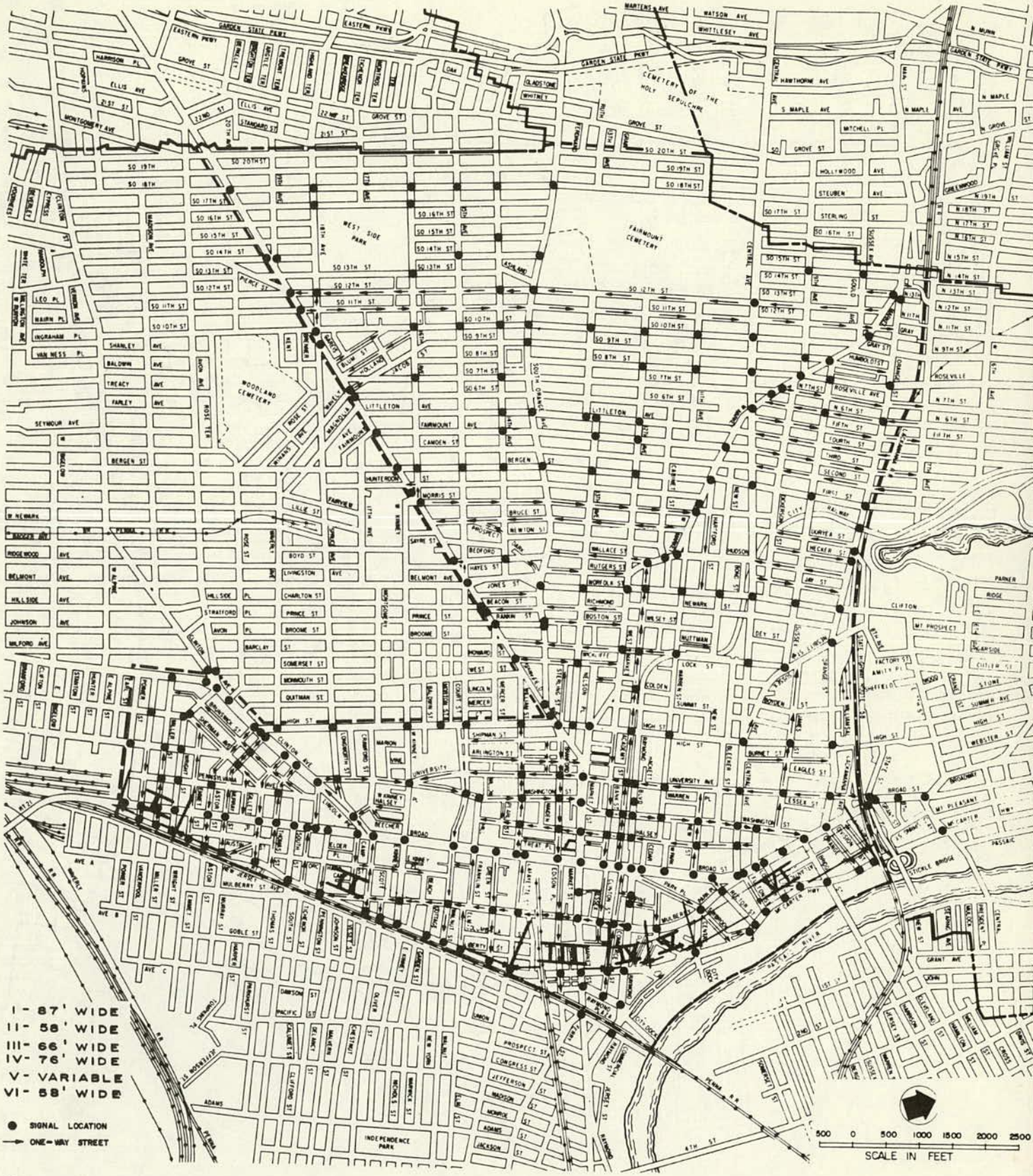


Figure G-183. Vicinity map.

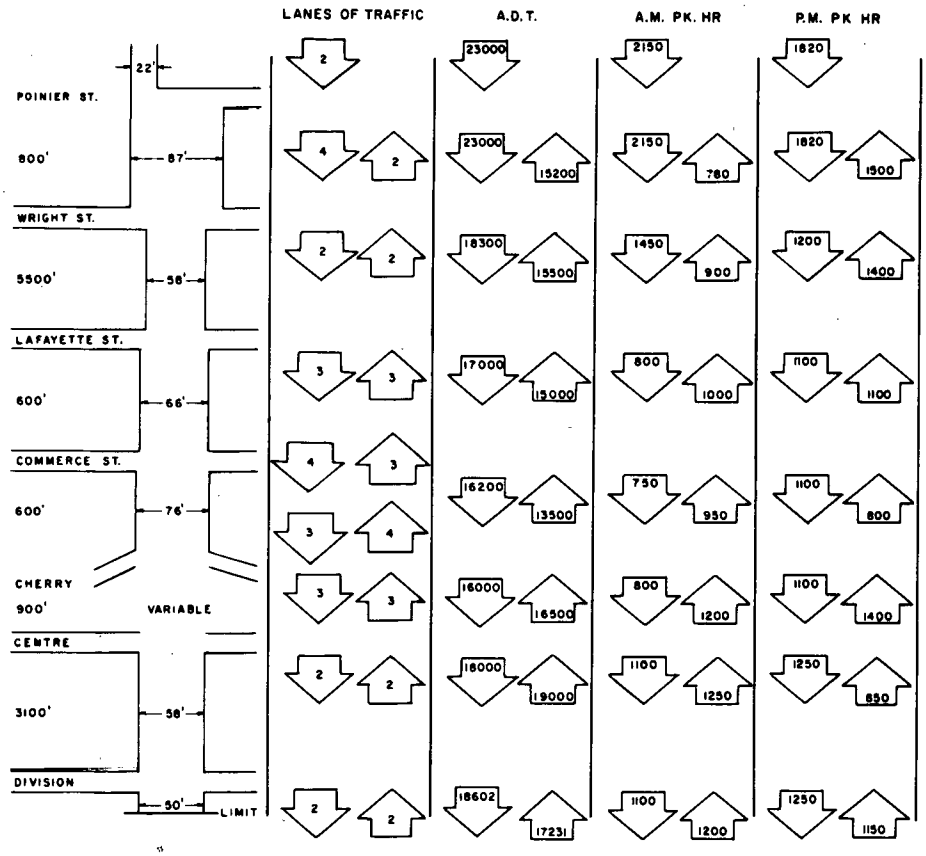


Figure G-184. 1968 traffic volumes.

bus stop operations, passenger service operations, and route time between bus stops. Accurate prediction of the time required for passenger service, together with accurate predictions of bus stop operation time and route time, can be used to optimize bus transit schedules for operational efficiency and passenger convenience.

The purpose of Experiment F63 is to define and describe the factors controlling the element of passenger service operations and to investigate the time required for such service. The interaction between transit operations and other traffic and the regulatory control of all traffic is included in the elements of bus stop operation and route time. The factors that control passenger service operations include the number of alighting and boarding passengers for each service performed, door use by alighting passengers, methods of fare collection, type of transfer activity, and types of passengers as represented by the time of day (i.e., inbound or outbound commuters or shoppers). Specific human factors such as age and physical condition of the passenger and the number and type of parcels being carried are considered to be uniformly reflected in the survey periods analyzed.

TABLE G-170

TRAVEL CHARACTERISTICS, SPEED AND DELAY DATA

DIRECTION	VARIABLE	AM PEAK PERIOD DRY MEAN	DRY OBS.	WET MEAN	WET OBS.	DIFFERENCE	SIG. @ α LEVEL INDICATED
NB	Travel time (sec)	615.8	18	687.7	10	+71.9 (11.7%)	0.10
	Delay time (sec)	222.5	17	249.8	10	+27.3 (12.3%)	0.15
	No. of stops	7.2	17	8.3	10	+1.1 (15.3%)	0.10
SB	Travel time (sec)	595.1	16	706.7	10	+111.6 (18.8%)	0.025
	Delay time (sec)	212.8	16	263.8	10	+51.0 (24.0%)	0.025
	No. of stops	6.8	16	8.6	10	+1.8 (26.5%)	0.025

Experiment F63 develops a method by which the time required for passenger service can be accurately estimated from the total number of passengers requiring service, whether they are alighting, boarding, or both. The data developed are applicable only to transit service operating under conditions similar to those observed. The method, however, can be used to develop similar data for transit service operating under a variety of conditions.

At the time the survey was made, the Louisville Transit Company charged a fare of $0.30, without zone limitations and with free transfer privileges between routes by means of paper transfers issued by the bus operators. A token system was also in effect and an additional different fare structure provided for students. Transfers were not available for token-paying passengers. Weekly or monthly commutation tickets were not used. For the purposes of this report, the previously described system is defined as the "cash and change" system.

Most buses were 40 ft in length, were equipped with 53 seats, and had a maximum legal load of 72 passengers.

This experiment also examines the observed delays to passenger service resulting from passengers alighting from the front door, through which passengers were waiting to board, to determine if a rear-door-alighting-only policy would reduce the time required for passenger service.

It is recognized that a "pay-leave" or a zone fare collection system requiring ticket surrender, either or both of which are used in other areas, would necessitate front-door alighting. Under the conditions observed, however, a rear-door-alighting-only policy is feasible.

The effects of such a policy were determined by simulation, using the times observed and recorded for each passenger service operation. The quantitative measurements of existing transit service in Louisville were made almost exclusively on buses of the Louisville Transit Company.

Experimental Area

Twelve different locations in the downtown Louisville area (Fig. G-185) were selected for intensive observation of passenger service activity. These locations were chosen because they had the highest concentration of passenger service activity in the city (Table G-171). The location of the bus stop, either near or far side, and the operational conditions present were assumed to have no effect on the time required to provide passenger service.

TABLE G-171

SURVEY LOCATIONS

STATION	BUS STREET	DIRECTION	CROSS STREET	DAILY PASS. [a]	RANK
1	Broadway	EB	Fourth St.	1,736	5
2	Broadway	WB	Fourth St.	1,911	4
3	Chestnut St.	EB	Fourth St.	1,699	6
4	Fourth St.	NB	Broadway	1,961	3
5	Fourth St.	NB	Chestnut St.	1,446	9
6	Fourth St.	NB	Walnut St.	1,641	7
7	Fourth St.	NB	Jefferson St.	1,274	11
8	Fourth St.	SB	Walnut St.	1,239	12
9	Fourth St.	SB	Broadway	1,387	10
10	Market St.	WB	Fourth St.	1,512	8
11	Walnut St.	WB	Fourth St.	2,223	2
12	Market St.	EB	Fourth St.	2,394	1
All				20,423	

[a] 6:00 AM to 6:00 PM, inclusive, for April-May 1963 (Louisville Transit Co.).

Field Procedure

To permit total examination of the existing passenger service operation, the number of passengers and the time required for rear-door alighting, front-door alighting, and front-door boarding were individually measured and recorded. To ensure compatibility for comparison of separately observed passenger service operations, each event was defined as follows:

1. *Alighting time* is the interval of time measured from the moment the bus stops and the door has opened to the moment when the last alighting passenger has stepped from the bus, regardless of whether the bus door(s) remains open or is closed.

2. *Boarding time,* except as given in the following, is the interval of time measured from the moment the bus stops and the door has opened to the moment when the last passenger waiting to board has stepped onto the bus, regardless of whether the bus door remains open or is closed, and whether the last passenger boarding is able to move directly to the fare collection point. When passengers alight from the door being used for boarding prior to the beginning of the boarding operation, the end of the "alighting time" interval, as defined previously, is considered to be coincidental with the beginning of the "boarding time" interval.

3. *Passenger service time* is defined as the interval of time measured from the beginning of the "alighting time" and/or "boarding time," whichever occurs first, to the end of the "alighting time" and/or "boarding time," whichever occurs last.

The points selected for beginning and ending each event were chosen because they were subject to the least distortion due to operators' habits. Some operators were observed to begin movement as soon as the last boarding passenger stepped onto the bus; others were observed to complete the collection of all fares prior to beginning movement. Doors also were observed to have been left open after the completion of the boarding operation, which would have made their closing a poor choice for defining the end of the "alighting" or "boarding time" intervals. This was observed to be especially true if the weather was hot or if the bus was delayed by traffic signals in a near-side stop location. "Stragglers" boarding under this latter condition were ignored as far as these measurements were concerned. Their inclusion would have introduced an unpredictable variable in the survey operations.

Each observation of passenger service operations required two observers—one for timing intervals and one for volume measurements and general observations.

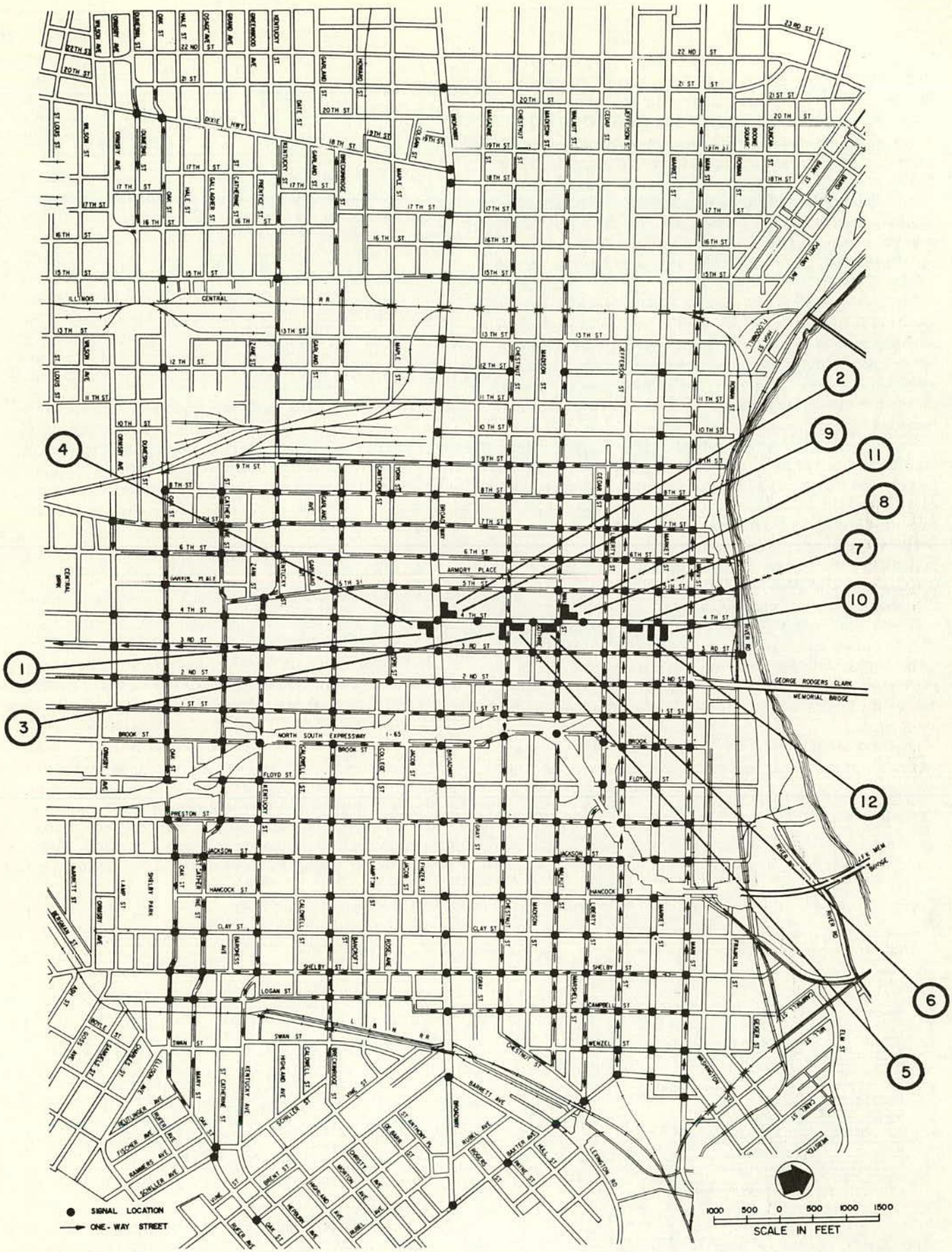


Figure G-185. Location map, Experiment F63.

Measurements

Definition of types of passengers by assumed trip purpose for the three survey periods is given in Table G-172.

During the period from May 16 through May 29, 1968, inclusive, four separate crews of two men each surveyed existing passenger service operations, as given in Table G-173.

To separate the influences that various combinations of alighting and/or boarding passengers may have had on the over-all passenger service time, each individually observed and measured passenger service operation was coded (Table G-174).

The range in time of observed and recorded passenger service was from 1 to 106 sec, with from 1 to 43 passengers alighting and/or boarding. Figure G-186 shows the number of observations for each passenger group size (alighting and/or boarding). Figure G-187 shows the minimum, mean, and maximum passenger service times observed and recorded for each passenger group size.

Summaries of the recorded field data by survey period, classified as to type of passenger service, are given in Tables G-175, G-176, and G-177. A summary of all survey periods is given in Table G-178. The observed mean alighting times of 1.9 and 2.2 sec for the rear and front doors, respectively, and the mean boarding time of 2.6 sec, for all survey periods, compare favorably with the ranges of 1.5 to 6 sec for alighting and 2 to 8 sec for boarding, as given in the *Highway Capacity Manual* (*1*, p. 346).

In the AM period (Table G-175) the mean observed alighting time for both the front and rear doors appears to be significantly less than the mean times observed during the midday and PM periods. This appears to be logical due to the high percentage of inbound commuters in the AM period.

Of the total observations, 81, or 17 percent, were made while rain was falling. A comparison of these totals and their means with those for observations made when it was not raining are given in Table G-179. There does not appear to be any significant difference in the observed times, which, while seeming to be illogical, may be partially explained by the fact that the "rain" occurring was light and intermittent.

Figure G-188 shows the over-all passenger direction, inbound and outbound, as well as the dominant passenger service activity, either alighting or boarding, individually by station location. Stations 1, 2, and 3 indicate a positive reversal of activity by survey period; Stations 4, 5, 6, and 7 are predominantly alighting locations. Conversely, Stations 8, 10, 11, and 12 are predominantly boarding locations. Station 9 is opposite to the normally expected trend of passenger service activity.

Analysis

An analysis of the survey data was made to determine over-all passenger service time for alighting and boarding, separately and in combination. Because an accurate prediction of door use could not be made under existing conditions of operation, the seven surveyed classifications of passenger service were combined for analysis into three categories, as follows:

1. Alighting only (Codes 1, 2, and 3).
2. Boarding only (Code 4).
3. Boarding with alighting (Codes 5, 6, and 7).

Statistical analyses were performed, using the least squares method of multiple linear stepwise regression. Separate equations were developed for each category for each survey period (Tables G-180, G-181, G-182). The number of alighting passengers (X_1) and the number of boarding passengers (X_2) are the independent variables, and the passenger service time (Y) is the dependent variable. In general, the correlation coefficients indicate strong correlation, and the standard errors of estimate are sufficiently small to permit the predictive equations to be meaningful. The acceptable limits of the independent variables are listed in the last column of each table. Passenger service times predicted, using independent variables outside these

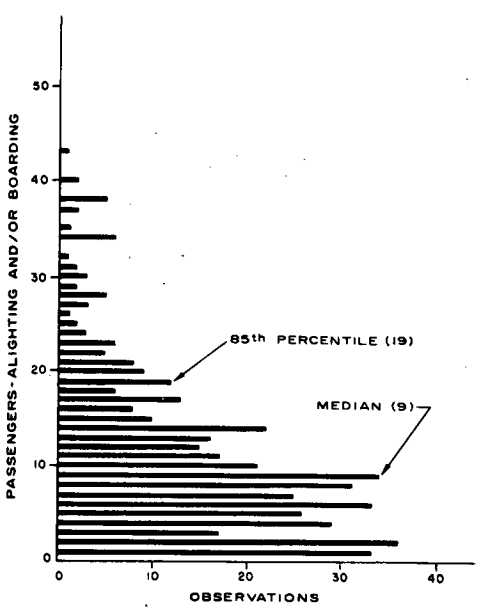


Figure G-186. Range of observed passenger group size.

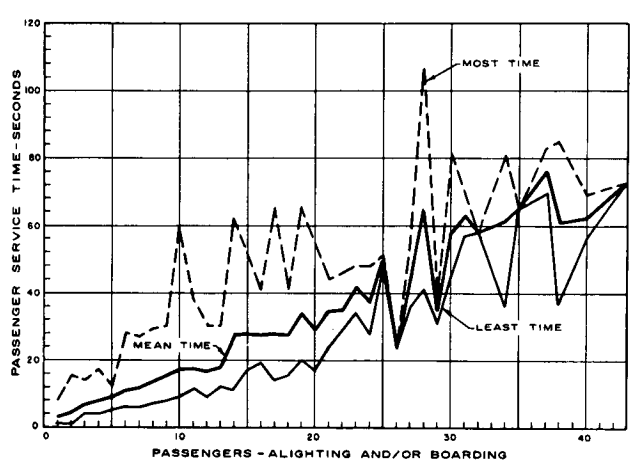


Figure G-187. Range of observed passenger service times.

limits, cannot be considered to be consistent with the basic data.

The equations for the combined times of each category are shown graphically in Figures G-189, G-190, and G-191. Confidence limits at the 95-percent level for categories of alighting only and boarding only are shown in Figures G-189 and G-190, respectively. The outer limits are for the minimum and maximum expected values for individual observations. For example, a single sample at a location indicates that 10 passengers alighted from a bus at 7:30 AM. Using Figure G-189, the maximum expected passenger service time would be 19.4 sec, the minimum expected passenger service time would be 6.4 sec, and the average expected passenger service time would be 12.9 sec. Suppose that the number of passengers alighting from the same bus was observed for three days and the mean number of alighting passengers was 10 persons. Then, using Figure G-189, the maximum expected passenger service time would be 13.9 sec, the minimum expected passenger service time would be 12.2 sec, and the average expected passenger service time would be 12.9 sec.

Another use of the inner limits of Figures G-189 and G-190 would be to determine expected passenger service time at a particular stop for a predetermined time period (i.e., morning peak hours, midday period, or evening peak hours). For example, passenger use of a particular stop surveyed between the hours of 7:00 and 9:00 AM indicated an average use per bus of 12 persons boarding and none alighting. Using Figure G-190, the maximum, average, and minimum expected passenger service time would be 32.5 sec, 30.8 sec, and 29.0 sec, respectively.

Confidence intervals were not computed for the curves shown in Figure G-191, because they could not be effectively shown in this illustration. In the use of Figure G-191, for example, assume that 15 persons were observed alighting and 25 persons were observed boarding a bus at a particular location. The expected passenger service time from Figure G-191 would be 64.3 sec.

The regression analysis indicated that:

1. The passenger service time for local bus transit service is strongly related to the specific combinations of alighting and boarding passengers.

2. Equations having high correlation coefficients and relatively low standard errors of estimate can be derived to meaningfully predict the time required for passenger service based on local operating conditions. (See Tables G-180, G-181, and G-182, and Figs. G-189, G-190, and G-191.)

Bus service using different fare collection methods and operating conditions from those observed and analyzed in this report may result in passenger service time that varies significantly from that predicted by using Figures G-189, G-190, and G-191. Such results are not unexpected and merely affirm the stated limitations of the application of the predictive equations. However, the method of analysis used in this experiment is applicable to data obtained under varied conditions.

TABLE G-172

TRIP PURPOSE

FROM	TO	ASSUMED TRIP PURPOSE
7:00 AM	9:00 AM	Commuter-inbound
9:00 AM	4:00 PM	Shopper
4:00 PM	6:00 PM	Commuter and shopper—outbound

TABLE G-173

FIELD SURVEY SUMMARY

SURVEY PERIOD	HOURS OF OBS.	% OF TOTAL	BUSES TOTAL	NO STOP	MISSED[a]	SURVEYED	% OF TOTAL SURVEYED
AM	5	12.4	88	10	4	74	15.8
Midday	27¾	69.0	327	34	2	291	62.4
PM	7½	18.6	102	0	0	102	21.8
Total	40¼	100.0	517	44	6	467	100.0

[a] Arrived simultaneously with bus being surveyed.

TABLE G-174

PASSENGER SERVICE CLASSIFICATION

CODE	FRONT DOOR	REAR DOOR
1	(No service)	Alighting
2	Alighting	(No service)
3	Alighting	Alighting
4	Boarding	(No service)
5	Boarding	Alighting
6	Alighting and boarding	(No service)
7	Alighting and boarding	Alighting

TABLE G-175

SUMMARY OF OBSERVED PASSENGER SERVICE OPERATIONS, AM PERIOD—7:00 TO 8:59 AM, INCLUSIVE

		REAR DOOR		FRONT DOOR						BOTH DOORS		
		ALIGHTING		ALIGHTING		BOARDING		ALIGHTING & BOARDING		ALIGHTING	ALIGHTING & BOARDING	
PASS. SERVICE CLASS.	NO. OF OBS.	NO.	TIME (SEC)	NO.	TIME (SEC)	NO.	TIME (SEC)	NO.	TIME (SEC)	NO.	NO.	SERVICE TIME (SEC)
1	1	4	5	—	—	—	—	—	—	4	4	5
2	3	—	—	5	15	—	—	5	15	5	5	15
3	23	130	204	72	135	—	—	72	135	202	202	223
4	3	—	—	—	—	9	16	9	16	—	9	16
5	6	30	47	—	—	16	40	16	40	30	46	51
6	2	—	—	3	5	7	15	10	20	3	10	20
7	36	190	305	118	201	190	491	308	692	308	498	712
All	74	354	561	198	356	222	562	420	918	552	774	1042
Avg./ obs.		4.8	7.6	2.7	4.8	3.0	7.6	5.7	12.4	7.5	10.5	14.1
Avg./ pass.			1.6		1.8		2.5		2.2			1.3

	DOOR USE BY PASSENGERS (%)	
DOOR	ALIGHTING	BOARDING
Front	36	100
Rear	64	—
All	100	100

TABLE G-176

SUMMARY OF OBSERVED PASSENGER SERVICE OPERATIONS, MIDDAY PERIOD—9:00 AM TO 3:59 PM, INCLUSIVE

		REAR DOOR		FRONT DOOR						BOTH DOORS		
		ALIGHTING		ALIGHTING		BOARDING		ALIGHTING & BOARDING		ALIGHTING	ALIGHTING & BOARDING	
PASS. SERVICE CLASS.	NO. OF OBS.	NO.	TIME (SEC)	NO.	TIME (SEC)	NO.	TIME (SEC)	NO.	TIME (SEC)	NO.	NO.	SERVICE TIME (SEC)
1	10	31	64	—	—	—	—	—	—	31	31	64
2	12	—	—	15	47	—	—	15	47	15	15	47
3	49	221	416	181	436	—	—	181	436	402	402	547
4	26	—	—	—	—	96	249	96	249	—	96	249
5	45	179	381	—	—	377	1029	377	1029	179	556	1063
6	30	—	—	54	126	86	289	140	415	54	140	415
7	119	557	1127	362	812	885	2297	1247	3109	919	1804	3146
All	291	988	1988	612	1421	1444	3864	2056	5285	1600	3044	5531
Avg./ obs.		3.4	6.8	2.1	4.9	5.0	13.3	7.1	18.2	5.5	10.5	19.0
Avg./ pass.			2.0		2.3		2.7		2.6			1.8

	DOOR USE BY PASSENGERS (%)	
DOOR	ALIGHTING	BOARDING
Front	38	100
Rear	62	—
All	100	100

TABLE G-177

SUMMARY OF OBSERVED PASSENGER SERVICE OPERATIONS, PM PERIOD—4:00 TO 6:00 PM, INCLUSIVE

PASS. SERVICE CLASS.	NO. OF OBS.	REAR DOOR ALIGHTING NO.	REAR DOOR ALIGHTING TIME (SEC)	FRONT DOOR ALIGHTING NO.	FRONT DOOR ALIGHTING TIME (SEC)	FRONT DOOR BOARDING NO.	FRONT DOOR BOARDING TIME (SEC)	FRONT DOOR ALIGHTING & BOARDING NO.	FRONT DOOR ALIGHTING & BOARDING TIME (SEC)	BOTH DOORS ALIGHTING NO.	BOTH DOORS ALIGHTING & BOARDING NO.	BOTH DOORS ALIGHTING & BOARDING SERVICE TIME (SEC)
1	5	15	33	—	—	—	—	—	—	15	15	33
2	3	—	—	4	8	—	—	4	8	4	4	8
3	15	60	123	36	102	—	—	36	102	96	96	142
4	13	—	—	—	—	99	280	99	280	—	99	280
5	20	85	178	—	—	239	557	239	557	85	324	557
6	8	—	—	12	29	50	143	62	172	12	62	172
7	38	225	434	119	236	292	762	411	998	344	636	1047
All	102	385	768	171	375	680	1742	851	2117	556	1236	2239
Avg./ obs.		3.8	7.5	1.7	3.7	6.7	17.1	8.3	20.8	5.5	12.1	22.0
Avg./ pass.			2.0		2.2		2.6		2.5			1.8

DOOR	DOOR USE BY PASSENGERS (%) ALIGHTING	DOOR USE BY PASSENGERS (%) BOARDING
Front	31	100
Rear	69	—
All	100	100

TABLE G-178

SUMMARY OF OBSERVED PASSENGER SERVICE OPERATIONS, ALL PERIODS—7:00 AM TO 6:00 PM, INCLUSIVE

PASS. SERVICE CLASS.	NO. OF OBS.	REAR DOOR ALIGHTING NO.	REAR DOOR ALIGHTING TIME (SEC)	FRONT DOOR ALIGHTING NO.	FRONT DOOR ALIGHTING TIME (SEC)	FRONT DOOR BOARDING NO.	FRONT DOOR BOARDING TIME (SEC)	FRONT DOOR ALIGHTING & BOARDING NO.	FRONT DOOR ALIGHTING & BOARDING TIME (SEC)	BOTH DOORS ALIGHTING NO.	BOTH DOORS ALIGHTING & BOARDING NO.	BOTH DOORS ALIGHTING & BOARDING SERVICE TIME (SEC)
1	16	50	102	—	—	—	—	—	—	50	50	102
2	18	—	—	24	70	—	—	24	70	24	24	70
3	87	411	743	289	673	—	—	289	673	700	700	912
4	42	—	—	—	—	204	545	204	545	—	204	545
5	71	294	606	—	—	632	1626	632	1626	294	926	1671
6	40	—	—	69	160	143	447	212	607	69	212	607
7	193	972	1866	599	1249	1367	3550	1966	4799	1571	2938	4905
All	467	1727	3317	981	2152	2346	6168	3327	8320	2708	5054	8812
Avg./ obs.		3.7	7.1	2.1	4.6	5.0	13.2	7.1	17.8	5.8	10.8	18.9
Avg./ pass.			1.9		2.2		2.6		2.5			1.7

DOOR	DOOR USE BY PASSENGERS (%) ALIGHTING	DOOR USE BY PASSENGERS (%) BOARDING
Front	36	100
Rear	64	—
All	100	100

TABLE G-179

SUMMARY OF OBSERVED PASSENGER SERVICE OPERATIONS, ALL PERIODS—7:00 AM TO 6:00 PM, INCLUSIVE

PASS. SERVICE CLASS.	NO. OF OBS.	REAR DOOR ALIGHTING NO.	REAR DOOR ALIGHTING TIME (SEC)	FRONT DOOR ALIGHTING NO.	FRONT DOOR ALIGHTING TIME (SEC)	FRONT DOOR BOARDING NO.	FRONT DOOR BOARDING TIME (SEC)	FRONT DOOR ALIGHTING & BOARDING NO.	FRONT DOOR ALIGHTING & BOARDING TIME (SEC)	BOTH DOORS ALIGHTING NO.	BOTH DOORS ALIGHTING & BOARDING NO.	BOTH DOORS ALIGHTING & BOARDING SERVICE TIME (SEC)
(*a*) Weather: rain												
All	81	253	515	122	276	512	1323	634	1599	375	887	1683
Avg./obs.		3.1	6.4	1.5	3.4	6.3	16.3	7.8	19.7	4.6	11.0	20.8
Avg./pass.			2.0		2.3		2.6		2.5			1.9
(*b*) Weather: no rain												
All	386	1474	2792	859	1876	1834	4776	2693	6652	2333	4167	7034
Avg./obs.		3.8	7.2	2.2	4.9	4.8	12.4	7.0	17.2	6.0	10.8	18.2
Avg./pass.			1.9		2.2		2.6		2.5			1.7

	DOOR USE BY PASSENGERS (%)			
	ALIGHTING		BOARDING	
DOOR	RAIN	NO RAIN	RAIN	NO RAIN
Front	33	37	100	100
Rear	69	63	—	—
All	100	100	100	100

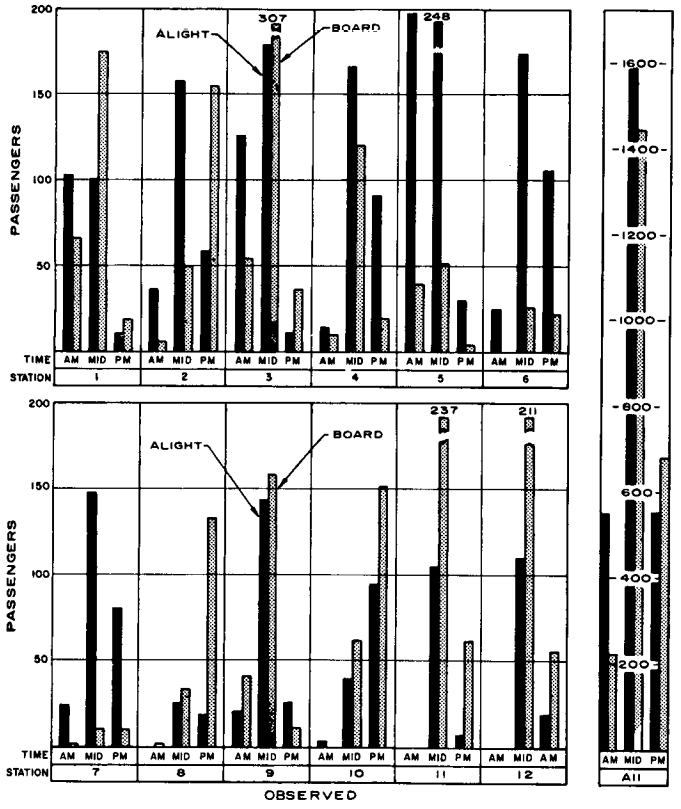


Figure G-188. Passengers observed alighting and boarding.

The predictive information obtainable by using Figures G-189, G-190, and G-191 can be used to determine the efficiency of existing passenger service operations and to evaluate changes such as the potential benefits resulting from combining bus stop locations.

Simulated Rear-Door Alighting Only

Referring to Table G-178, the average time observed per passenger service operation for passengers alighting from the front door was 4.6 sec. If this time is added to the 7.1 sec average time observed for passengers alighting from the rear door, the apparent total rear-door alighting time per observation would be 11.7 sec. The average time required for passengers boarding at the front door per observation was 13.2 sec.

From this computation it would appear that the mean passenger service time for all observations would have been 13.2 sec, instead of the 18.9 sec observed, if all alighting had been from the rear door, a time saving per observation of 4.6 sec, or about 25 percent.

An examination of the individually recorded operations by survey period revealed that in one-half of all operations observed, passengers alighted from the front door, while other passengers were waiting to board (Table G-183).

To determine if a rear-door-alighting-only policy would actually result in reduced passenger service time for the operations observed, a simulation study was made.

TABLE G-180

REGRESSION EQUATIONS, PASSENGER SERVICE TIME, ALIGHTING ONLY, "CASH AND CHANGE" SYSTEM

PERIOD TIME	NO. OF OBS.	CORR. COEF.	STANDARD ERROR OF ESTIMATE	PREDICTIVE EQUATIONS FOR PASSENGER SERVICE TIME	ACCEPTABLE RANGE OF X_1
AM	27	0.90	2.245	$\hat{Y}=1.8203+0.9187X_1$	$1\leq X_1\leq 20$
Midday	71	0.89	3.096	$\hat{Y}=1.6067+1.2141X_1$	$1\leq X_1\leq 19$
PM	23	0.81	3.151	$\hat{Y}=2.0938+1.1725X_1$	$1\leq X_1\leq 17$
All	121	0.87	3.069	$\hat{Y}=1.8437+1.1122X_1$	$1\leq X_1\leq 20$

X_1 = number of passengers alighting; and
$\hat{Y}$ = expected time required for passenger service (sec).

TABLE G-181

REGRESSION EQUATIONS, PASSENGER SERVICE TIME, BOARDING ONLY,[a] "CASH AND CHANGE" SYSTEM

TIME PERIOD	NO. OF OBS.	CORR. COEF.	STANDARD ERROR OF ESTIMATE	PREDICTIVE EQUATIONS FOR PASSENGER SERVICE TIME	ACCEPTABLE RANGE OF X_2
Midday	26	0.97	2.727	$\hat{Y}=0.2396+2.5288X_2$	$1\leq X_2\leq 14$
PM	12	0.97	3.183	$\hat{Y}=-0.6494+2.7169X_2$	$2\leq X_2\leq 16$
Both	41	0.97	2.907	$\hat{Y}=-0.0855+2.5855X_2$	$1\leq X_2\leq 16$

[a] Sufficient data for the AM period were not available for analysis.
X_2 = number of passengers boarding; and
$\hat{Y}$ = expected time required for passenger service (sec).

TABLE G-182

REGRESSION EQUATIONS FOR BUS PASSENGER SERVICE TIME, "CASH AND CHANGE" SYSTEM, BOARDING WITH ALIGHTING

TIME PERIOD	NO. OF OBS.	MULTIPLE CORR. COEF.	STANDARD ERROR OF ESTIMATE	PREDICTIVE EQUATIONS FOR PASSENGER SERVICE TIME	ACCEPTABLE RANGE FOR X_1 AND X_2
AM	43	0.89	3.850	$\hat{Y}=3.5985+1.0089X_1-0.0913X_1X_2+0.4653X_2^2-0.0215X_2^3$	$1\leq X_1\leq 22$ $1\leq X_2\leq 17$
Midday	191	0.94	6.383	$\hat{Y}=1.1762+1.3822X_1-2.3041X_2-0.0828X_1X_2+0.0013X_2^3$	$1\leq X_1\leq 21$ $1\leq X_2\leq 30$
PM	63	0.98	4.164	$\hat{Y}=0.4757+1.1987X_1+2.2614X_2-0.0423X_1X_2$	$1\leq X_1\leq 24$ $1\leq X_2\leq 36$
All	297	0.94	5.787	$\hat{Y}=1.7701+0.9727X_1+2.2756X_2-0.0234X_1X_2$	$1\leq X_1\leq 24$ $1\leq X_2\leq 36$

X_1 = number of passengers alighting;
X_2 = number of passengers boarding; and
$\hat{Y}$ = expected time required for passenger service (sec).

Each operation involving a front-door conflict between alighting and boarding passengers, Code 6 and Code 7, was revised by simulating a rear-door-alighting-only operation. This was done by reassigning the actually observed total time for front-door alighting to the rear door. The results of this simulation are given in Table G-184.

The apparent indicated time saving per passenger service operation for Codes 6 and 7 is 2.0 sec, which corresponds to a saving of 8.5 percent in over-all passenger service time for Code 6 or Code 7 operations if alighting is restricted to the rear door only. (Mean passenger service time for Code 6 and Code 7 combined for all observations as calculated from Table G-178 is 23.7 sec.)

Whereas Code 6 and Code 7 operations would save time by a rear-door-alighting-only policy, Code 3, alighting from both doors without any boarding, would obviously require

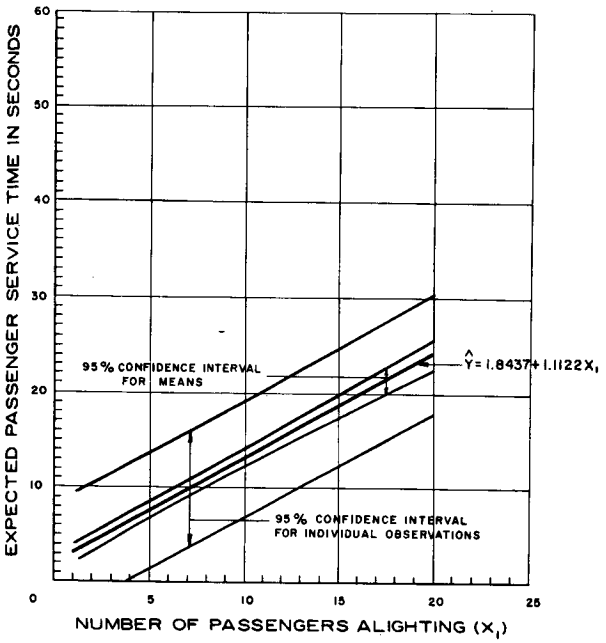


Figure G-189. Expected passenger service time, alighting only, "cash and change" system.

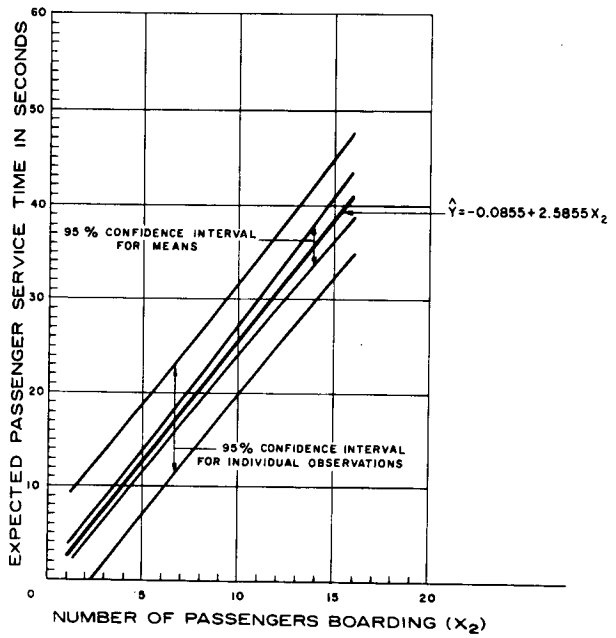


Figure G-190. Expected passenger service time, boarding only, "cash and change" system.

additional time. The amount of added time for the operations observed is given in Table G-185.

A comparison of Tables G-184 and G-185 indicates a rear-door-alighting-only policy would result in an insignificant time saving for the operations observed. Only Codes 3, 6, and 7 would be affected by such a policy.

The fact that no Code 3 operations were observed at Stations 1, 8, 11, and 12, and almost 90 percent of such operations observed occurred at Stations 5, 6, and 7 supports the conclusion that a rear-door-alighting-only policy might prove beneficial for particular bus stop locations or segments of routes where more passengers repeatedly board than alight. This could be particularly applicable on routes leaving the CBD area in the PM period.

To achieve the possible benefits of reduced passenger service time, compliance on the part of alighting passengers is required. A measure of voluntary compliance might be achieved by means of a driver-operated, blank-out type, illuminated sign located over the aisle at the front door. Such a sign would permit the application of a flexible policy requiring passengers to alight only from the rear door at certain stops and permitting alighting from both doors when experience indicates favorable conditions for such operation.

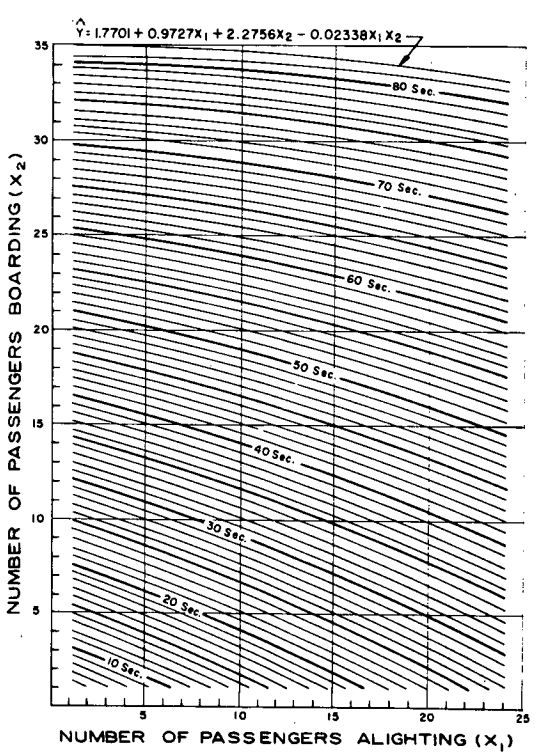


Figure G-191. Expected passenger service time, boarding with alighting, "cash and change" system.

TABLE G-183

FRONT-DOOR OPERATIONS

TIME PERIOD	NO. OF OBS.	BOARDING ONLY NO. OF OBS.	BOARDING ONLY % OF TOTAL	ALIGHTING AND BOARDING NO. OF OBS.	ALIGHTING AND BOARDING % OF TOTAL
AM	74	9	12	38	51
Midday	291	71	24	149	51
PM	102	33	32	46	45
All	467	113	24	233	50

TABLE G-184

CODES 6 AND 7, SIMULATED REAR-DOOR ALIGHTING ONLY

TIME PERIOD	NO. OF OBS.	NO CHANGE	REASSIGN USING RECORDED TIMES: IN-CREASE	DE-CREASE	NET CHANGE (SEC)
AM	38	1	12	25	−70.0
Midday	149	3	34	112	−373.0
PM	46	2	17	27	−17.0
All	233	6	63	164	−460.0
Mean/obs.					−2.0

TABLE G-185

CODE 3, SIMULATED REAR-DOOR ALIGHTING ONLY

TIME PERIOD	NO. OF OBS.	REASSIGN USING RECORDED TIMES: INCREASE	DECREASE	NET CHANGE (SEC)
AM	23	23	0	+16.0
Midday	49	49	0	+305.0
PM	15	15	0	+83.0
All	87	87	0	+404.0
Mean/obs.				+4.6

Addendum to Bus Transit—Passenger Service Operations—Experiment F63

On November 10, 1968, the Louisville Transit Company changed its method of fare collection from a pay-enter system where the driver would collect the fare and give change ("cash and change" system) to a pay-enter system where the driver would not touch the fare ("exact fare" system). In this latter system, the passenger is required to deposit the exact fare in a sealed box as he enters. If he does not have the correct fare, he can deposit a greater amount and receive script from the driver for the overpayment. The passenger can redeem the script at one of three locations. Presently, less than 0.1 percent of the total number of passengers per week receive script, according to Louisville Transit Company records.

The purpose of this addendum is to determine if passenger service time for the "exact fare" system is related to specific combinations of alighting and boarding. The experimental area is the same as shown in Figure G-185, and surveys were performed at the same 12 locations given in Table G-171. The surveys were performed as described under "Field Procedure," except that no distinction was made as to the door(s) from which the passengers alighted and/or boarded. These movements were recorded between June 23, 1969, and June 27, 1969, during the following time periods: AM—7:30 to 9:00 AM; midday—9:30 to 11:00 AM; and PM—4:00 to 5:30 PM.

Data were collected for a total of 537 observations, of which 147 were from stops with alighting passengers only, 31 were from stops with boarding passengers only, and 359 were from stops with simultaneously boarding and alighting passengers. These were analyzed in the following categories: Category 1, alighting only, Category II, boarding only, and Category III, boarding with alighting.

Analysis.—Because time did not appear to drastically influence the results of the least squares procedures for the "cash and change" system, linear regression equations were determined for each category of the "exact fare" system, combining the time periods within these categories. Table G-186 gives the resultant equations for predicting passenger service time. The number of passengers alighting (X_1) and the number of passengers boarding (X_2) are the independent variables; the passenger service time ($\hat{Y}$) is the dependent variable. The acceptable limits of the independent variables are listed in Table G-186. Generally, it is not considered reasonable to predict outside of these limits.

The equations in Table G-186 are shown graphically in Figures G-192, G-193, and G-194. Confidence limits at the 95-percent level for categories of alighting only and board-

TABLE G-186

REGRESSION EQUATIONS FOR BUS PASSENGER SERVICE TIME, "EXACT FARE" SYSTEM

CATEGORY	MULTIPLE CORR. COEF.	STANDARD ERROR OF ESTIMATE	NO. OF OBS.	EQUATION FOR PREDICTING PASSENGER SERVICE TIME	ACCEPTABLE RANGE OF INDEPENDENT VARIABLES
I—alighting only	0.88	2.738	147	$\hat{Y}=2.2345+1.0792X_1$	$1\leq X_1\leq 26$
II—boarding only	0.97	2.702	31	$\hat{Y}=0.5863+1.9957X_2$	$1\leq X_2\leq 25$
III—boarding with alighting	0.91	6.556	359	$\hat{Y}=1.6043+0.9588X_1+2.154X_2-0.0202X_1X_2$	$1\leq X_1\leq 30$ $1\leq X_2\leq 33$

X_1 = number of passengers alighting;
X_2 = number of passengers boarding; and
$\hat{Y}$ = predicted time required for operation (sec).

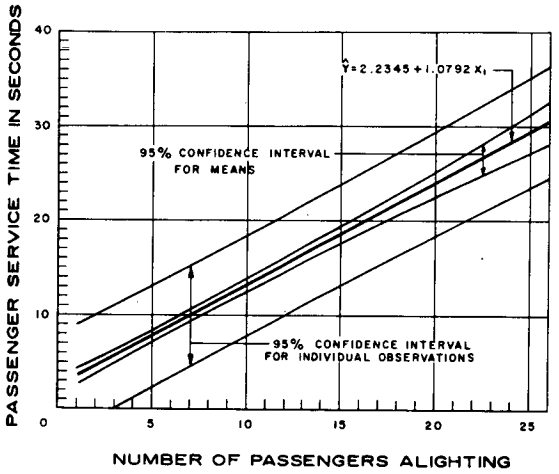


Figure G-192. Expected passenger service time, alighting only, "exact fare" system.

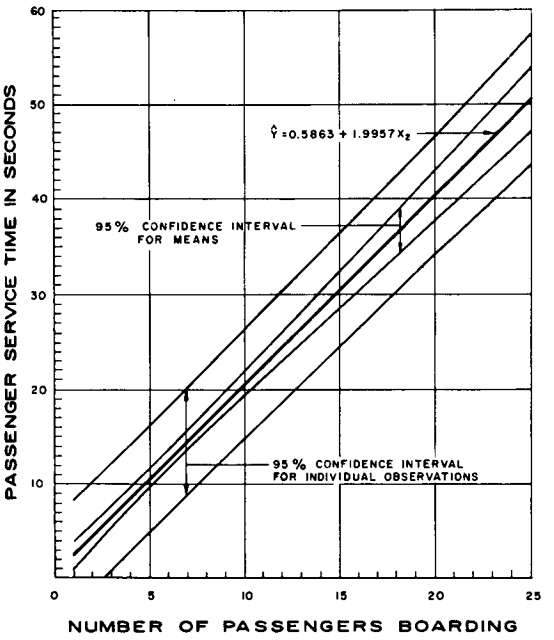


Figure G-193. Expected passenger service time, boarding only, "exact fare" system.

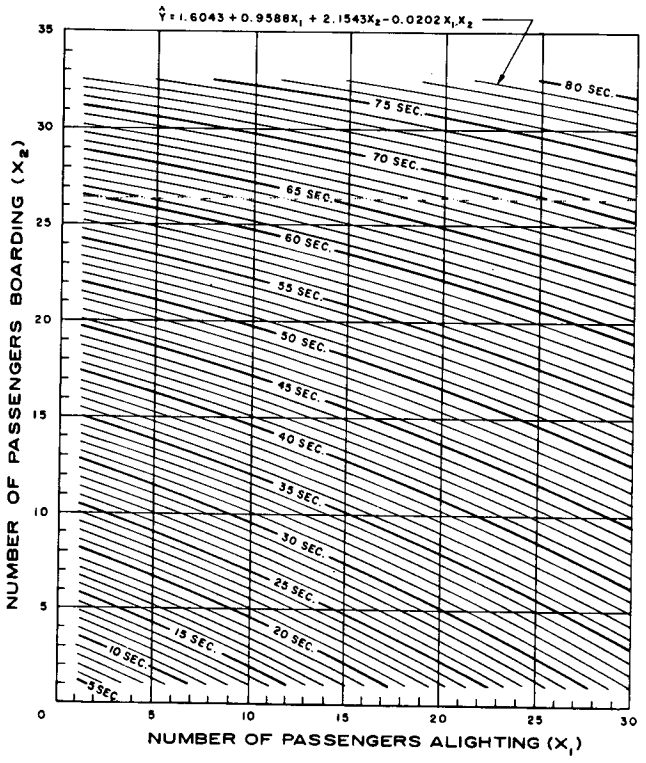


Figure G-194. Expected passenger service time, boarding with alighting, "exact fare" system.

ing only are shown in Figures G-192 and G-193, respectively. The uses of these limits are the same as those described previously for the "cash and change" system under "Analysis."

Conclusions.—Based on the foregoing analysis, it is concluded that:

1. Passenger service time for local bus transit service using an "exact fare" system is strongly related to the specific combinations of alighting and boarding passengers.

2. Equations having high correlation coefficients and relatively low standard errors of estimate can be derived to meaningfully predict the time required for passenger service based on local operating conditions and an "exact fare" system.

Optimum Location of Bus Stops—Experiment F64

Local transit bus operations over any given route can be separated into the categories of stop operation, passenger service, and route time between bus stops. Each of these items influences the over-all efficiency of operations to degrees that vary with local conditions.

Experiment F64 is concerned with the influence that stop operations have on bus route operations and involves segmenting stop operations into defined phases susceptible to incremental time measurement. The classification and time measurement of local factors affecting stop operations are a necessary part of the experiment.

Experimental Area

Eight bus stops in the Louisville study area were selected for observation and measurement of stop operations. These locations were chosen because they had similar traffic volumes, physical features, and traffic signal timing. They were also selected because of their proximity to the center of the downtown area. The locations of these bus stops, with respect to the study area, are shown in Figures G-195 and G-196.

To test the local factors believed to exert the greatest influence on the efficiency of stop operations, these bus stops included near-side and far-side locations; parking

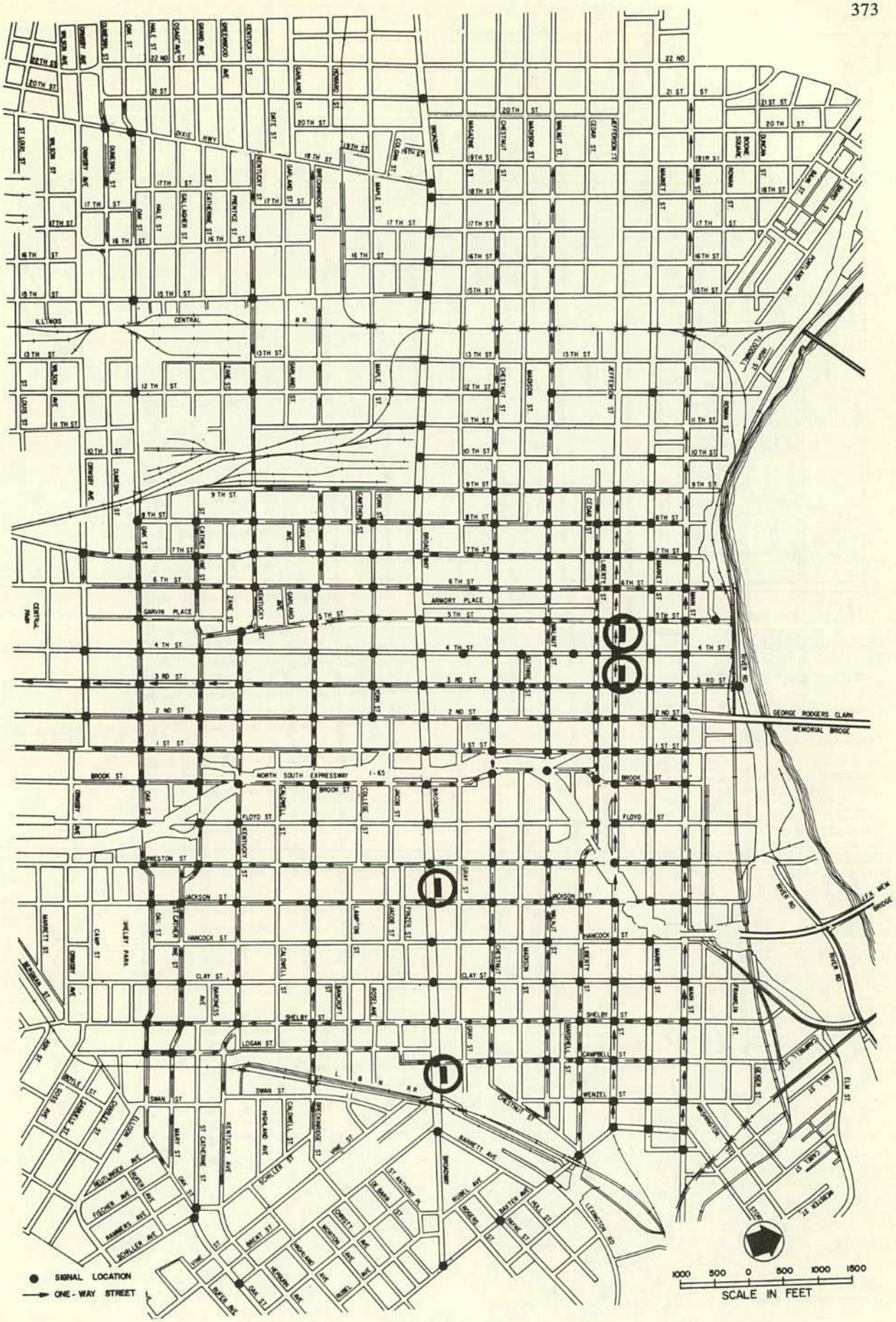


Figure G-195. Location map (parking permitted), Experiment F64.

Figure G-196. Location map (parking prohibited), Experiment F64.

permitted and prohibited on the bus approach; and cross-street traffic flow either from the left or the right. One stop location of each type was selected (Fig. G-197).

Purpose and Scope

The purpose of Experiment F64 is to determine whether a near-side or far-side stop location is better for local transit bus operations at traffic-signal-controlled intersections with respect to the frequency and magnitude of delays encountered.

This determination will be made on the basis of a comparison of operations at bus stop locations characterized by the varying conditions of parking, direction of cross-street traffic flow, and volumes of traffic during different time periods of an average weekday.

Design of Experiment

The street network of a city must function for a variety of interests, not all of which are in harmony. It must serve through and local trips by passenger vehicles, buses, and trucks, and must also provide access to abutting properties for the pickup and delivery of persons and goods. Therefore, in determining the effects of modifications to traffic operations for optimizing flow on such streets, it becomes necessary to give consideration to the functional purpose of the street, the elements of the traffic stream, the physical street pattern, and the use of available curb space.

In almost every city there exist arterial streets where the volumes of bus traffic are a significant element of the total traffic flow. Modifications to bus operations on such streets may have a disproportionate effect on the flow of other elements in the traffic stream, due to the size and operating characteristics of buses.

Conversely, improvement projects on these same streets, intended primarily to improve through traffic movement or eliminate local traffic congestion, may affect bus operations significantly.

The cities of Newark and Louisville are well suited for study of bus operations due to their basic differences in street networks, traffic conditions, and bus volumes. Figure G-198 shows the differences in bus operations in the two study areas.

Conferences were held with the city and transit officials of Newark and Louisville to determine the range and location of problems affecting bus flow. These locations and potentials were examined. In addition, all bus routes in each study area were surveyed to determine other locations where experiments could be conducted.

Study Plan.—Historically, bus stops have been located on the near side of intersections. Occasionally, they have

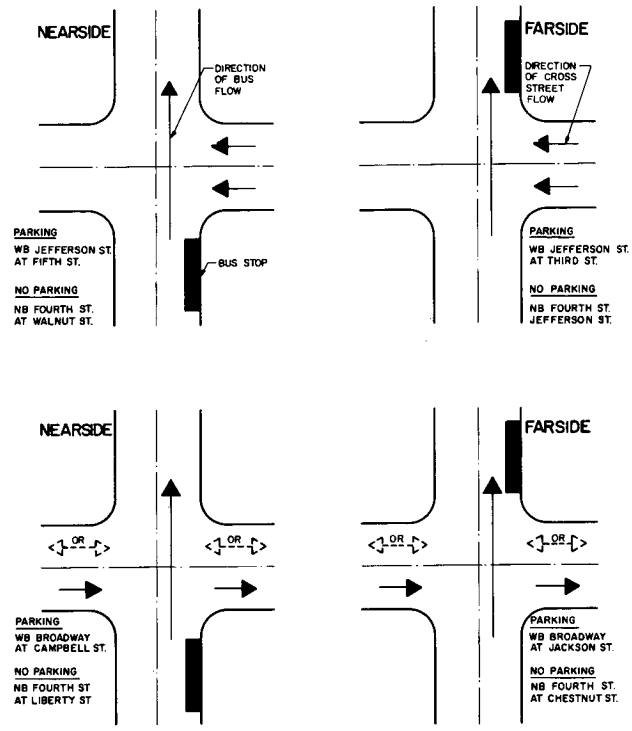


Figure G-197. Types of bus stops.

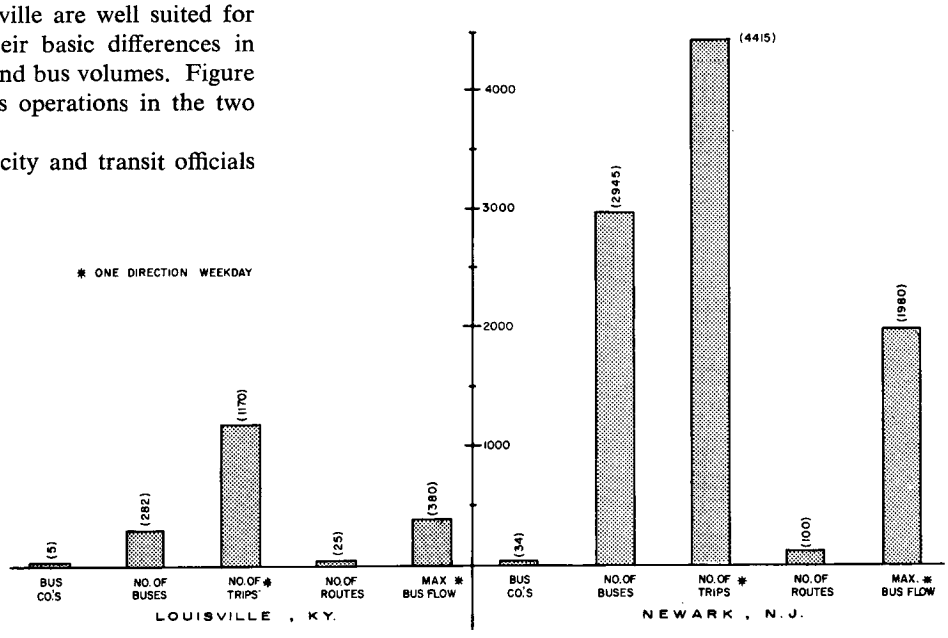


Figure G-198. Comparison of bus transit operation, 1968.

been located or relocated to the far side of the intersection when serious conflicts have occurred due to right-turning vehicles or for other traffic or physical conditions.

The study plan is designed to determine what the significant factors should be in designating either a near- or far-side stop location.

To set the limits of this study, it is necessary that several terms be defined:

Bus stop operations are defined as all actions or events occurring between the beginning of deceleration into the bus stop and the end of acceleration out of the bus stop, excluding the providing of passenger service.

1. The point of beginning deceleration is arbitrarily established as a point 155 ft before the stop line at the traffic signal.
2. The coincident end of deceleration and the beginning of the passenger service operation is defined as the point in time when the bus has stopped and the door or doors have been opened to permit passengers to alight and/or board.
3. The coincident end of the passenger service operation and the point of beginning acceleration is defined as the point in time when the last passenger has boarded or alighted.
4. The point of ending acceleration is arbitrarily established 120 ft beyond the far-side crosswalk.

These points are identical for near-side and far-side stop locations to permit comparison (Fig. G-199).

Passenger service operations are defined as all actions or events occurring while passengers are permitted to alight or board the bus, beginning with the end of the deceleration movement and ending with the beginning of the acceleration movement as defined by bus stop operations. Passenger service operations are dependent in time only on the number and type of passengers requiring service and are independent from all other bus operations.

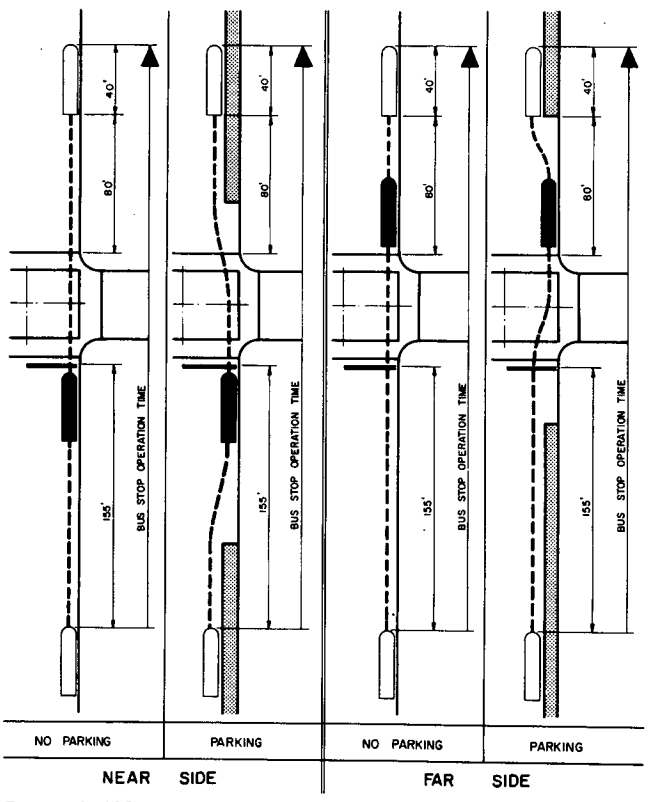


Figure G-199. Bus stop operation time limits.

Bus route operations are defined as all actions or events occurring between bus stop operations at succeeding stops. They are considered to exert no effect on the bus stop operations.

The principal delays to bus stop operations are generally conceded to be:

Near-Side Stops.—

1. Traffic signal delay—a red signal indication when approaching the stop in a queue, or when the passenger service operation has been completed (entry and exit).
2. Traffic in the bus stop area waiting to make a right turn (entry and exit).
3. Through or right-turning traffic from the adjacent lane (exit).

Far-Side Stops.—

1. Traffic signal delay at the approach to the intersection (entry).
2. Moving traffic in the adjacent lane (exit).

Surveillance System Design.—A measurement system was devised using two observers at each site to measure and classify these delays as well as to measure the times required for deceleration, the providing of passenger service, and acceleration.

Surveys.—The general physical characteristics, as well as the related traffic control features for each of the eight selected stop locations, are shown in Figures G-200 through G-207. The approximate approach volumes are also noted on these figures. The symbol DT adjacent to the approach volume number indicates the daily traffic volume as determined from short-term automatic traffic recorder counts. These figures have not been adjusted for multi-axled vehicles or seasonal factors. The symbol AADT similarly used indicates the annual average daily traffic for 1964 as recorded in *Traffic Volumes and Flow Characteristics* (September 1966) Vogt-Ivers and Associates, a report for the Louisville Metropolitan Comprehensive Transportation and Development Program.

Progressive signal timing, as well as the local intersection traffic signal timing, was determined; and time-space diagrams have been prepared to illustrate the relationship of adjacent intersections in this context (Figs. G-208, G-209, and G-210).

Periods of observations were restricted to fair weather and "normal" traffic conditions to avoid introducing additional variables into the evaluation and analysis. A total of 168 local transit bus stop operations were observed. Measurements were taken at the study sites between 7 AM and 6 PM in the period from May 22, 1968, to May 27, 1968.

Analysis and Conclusions.—To determine the practicality of attempting to predict the effects of relocating bus stops, the differences between near- and far-side bus stops located at signal-controlled intersections were statistically analyzed.

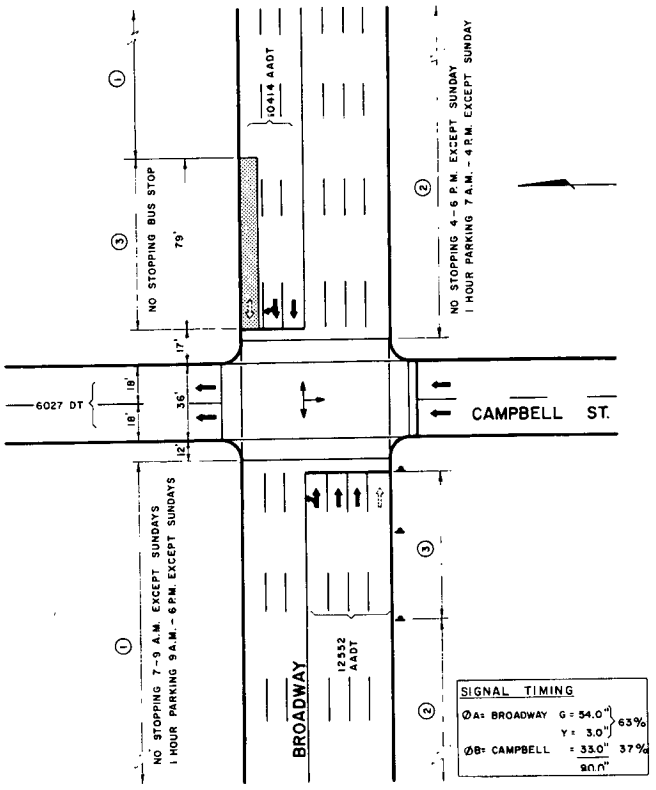


Figure G-200. Vicinity map, westbound Broadway at Campbell Street.

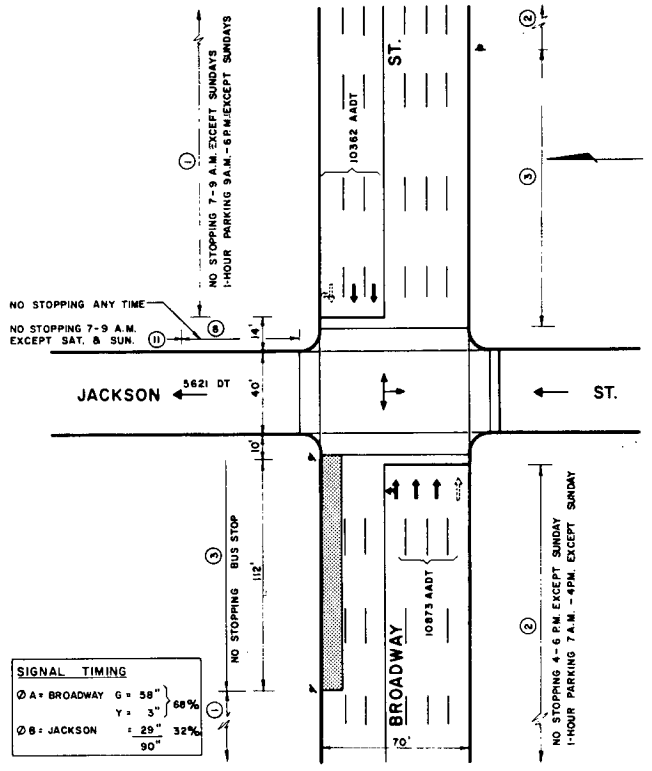


Figure G-201. Vicinity map, westbound Broadway at Jackson Street.

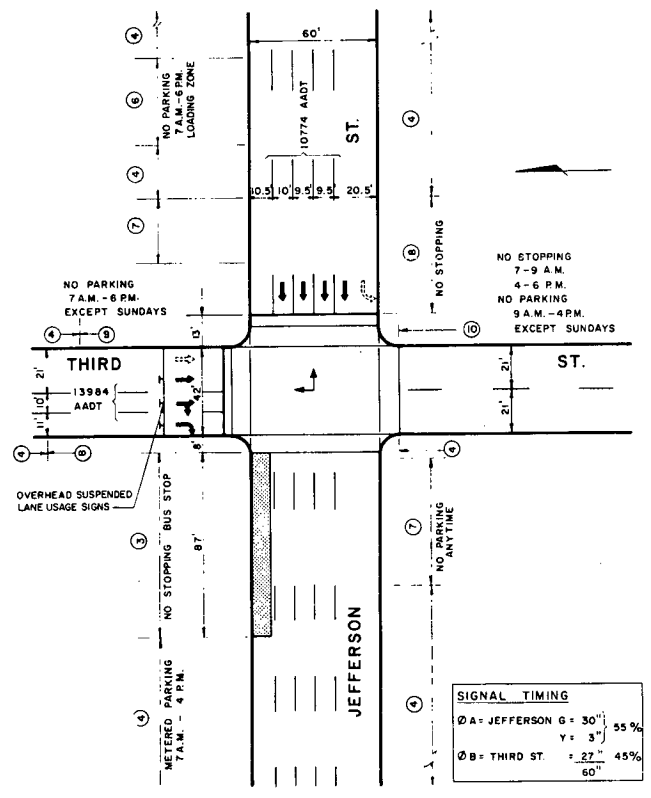


Figure G-202. Vicinity map, westbound Jefferson Street at Third Street.

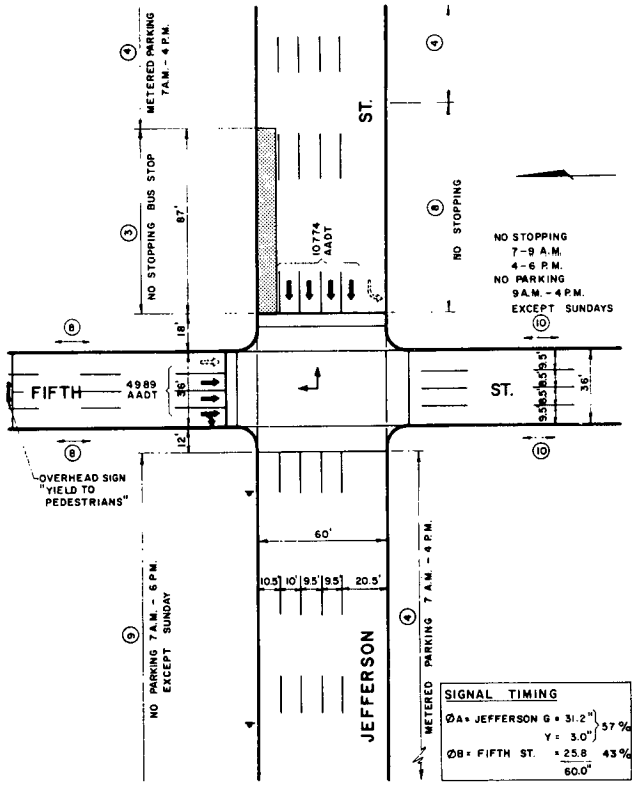


Figure G-203. Vicinity map, westbound Jefferson Street at Fifth Street.

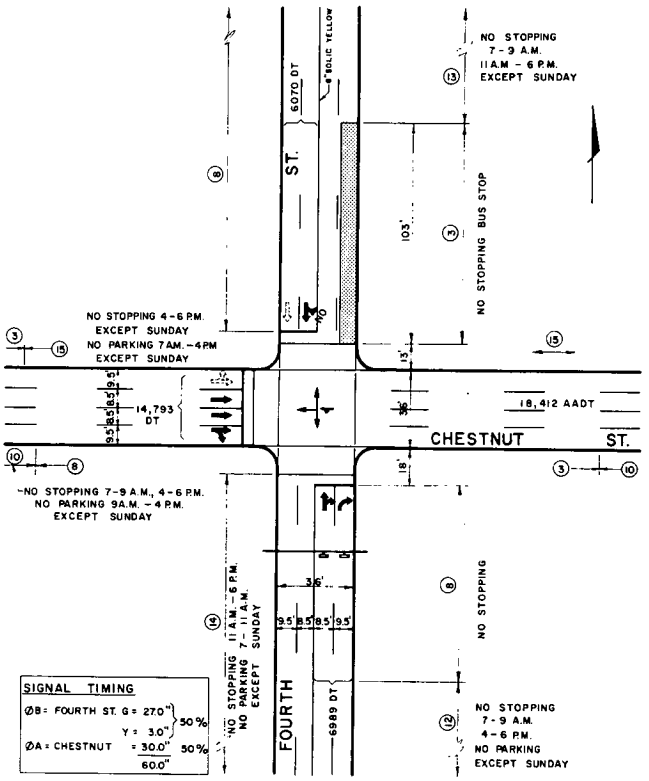


Figure G-204. Vicinity map, northbound Fourth Street at Chestnut Street.

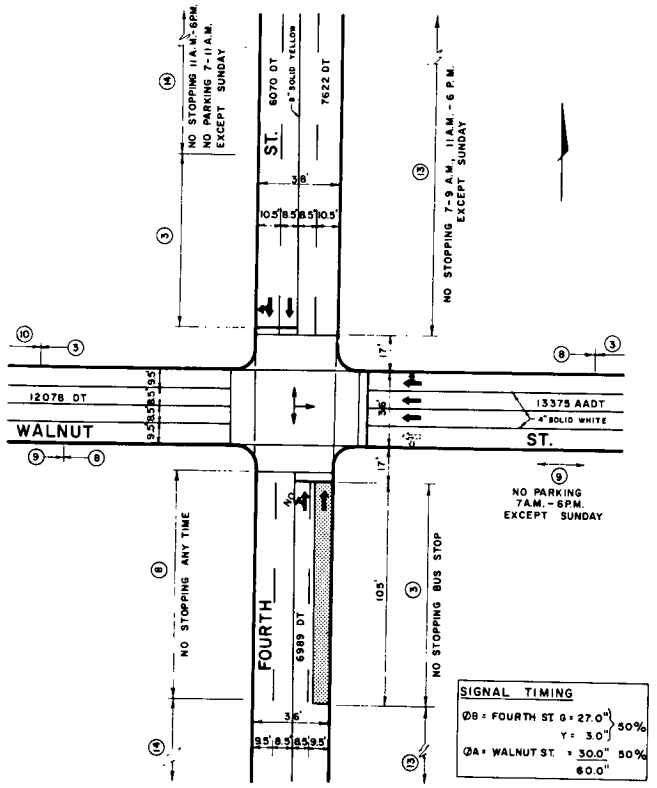


Figure G-205. Vicinity map, northbound Fourth Street at Walnut Street.

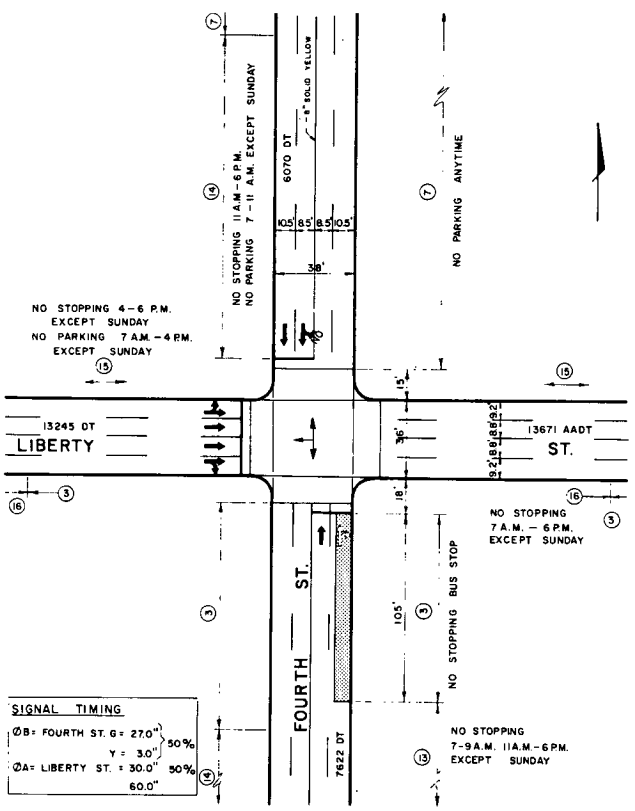


Figure G-206. Vicinity map, northbound Fourth Street at Liberty Street.

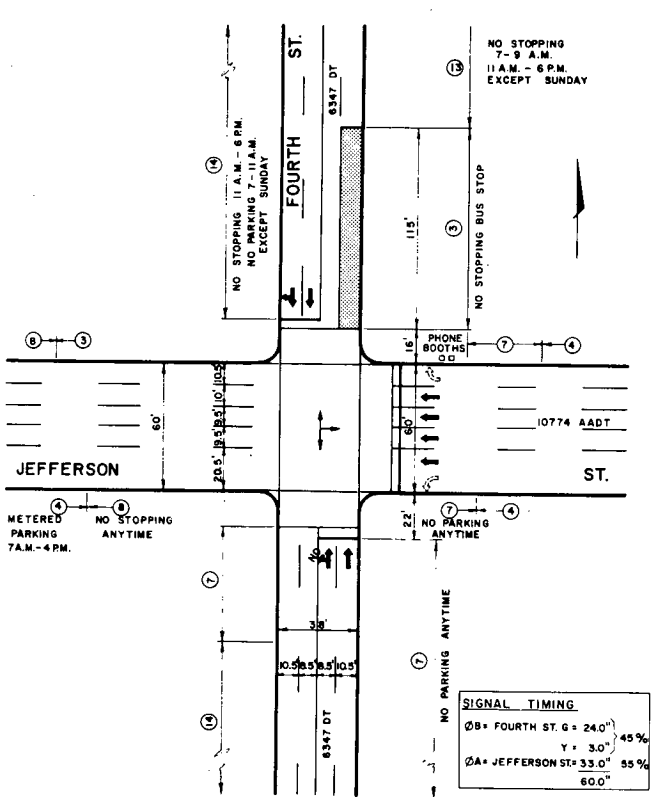


Figure G-207. Vicinity map, northbound Fourth Street at Jefferson Street.

This analysis was performed to determine the effects of the presence or absence of parking on the bus approach, the direction of cross-street traffic flow, and the time of day on the frequency of signal delay, bus stop operation time, and bus stop operation time less the time measured as traffic signal delay.

Between the limits established as shown in Figure G-199, bus operations require time for movement, including deceleration into and acceleration out of the stop location, and passenger service operations. Additional time may be required for traffic signal delays, delays due to other traffic movements (including pedestrians), and "stalling" delays.

Bus operations within the area shown in Figure G-199, less the time required for passenger service operations, have been previously defined as bus stop operations.

During the timing of the bus stop operation, a bus stopping or remaining stopped, after the original reason for stopping was no longer evident, was considered to be "stalling," and the time the bus remained immobile was measured. This was done in recognition of the fact that a bus operator ahead of his published schedule may "stall" intentionally to arrive at his checkpoints on time.

In the absence of an "on board" observer to monitor the amount of true "stalling" delay, 3 sec of the measured

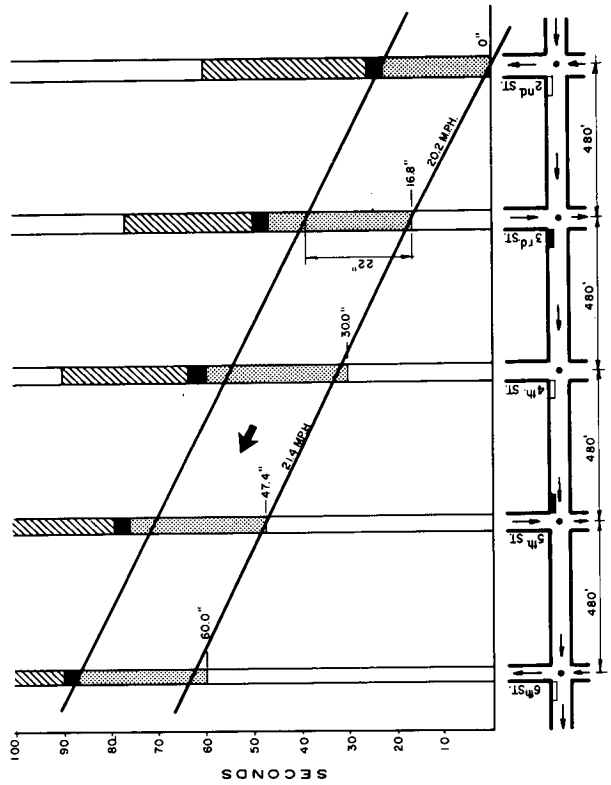


Figure G-209. Signal progression, Jefferson Street.

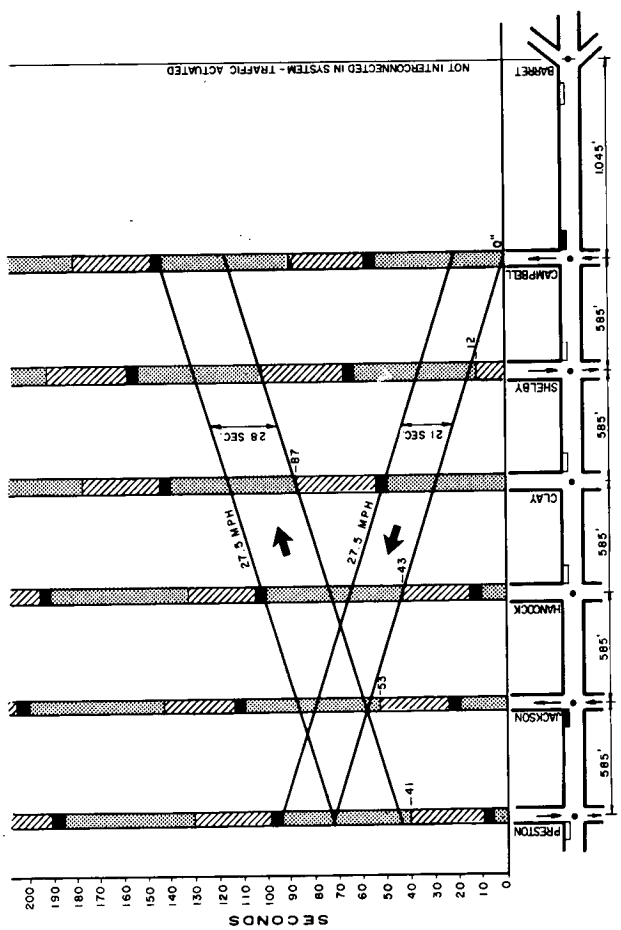


Figure G-208. Signal progression, Broadway.

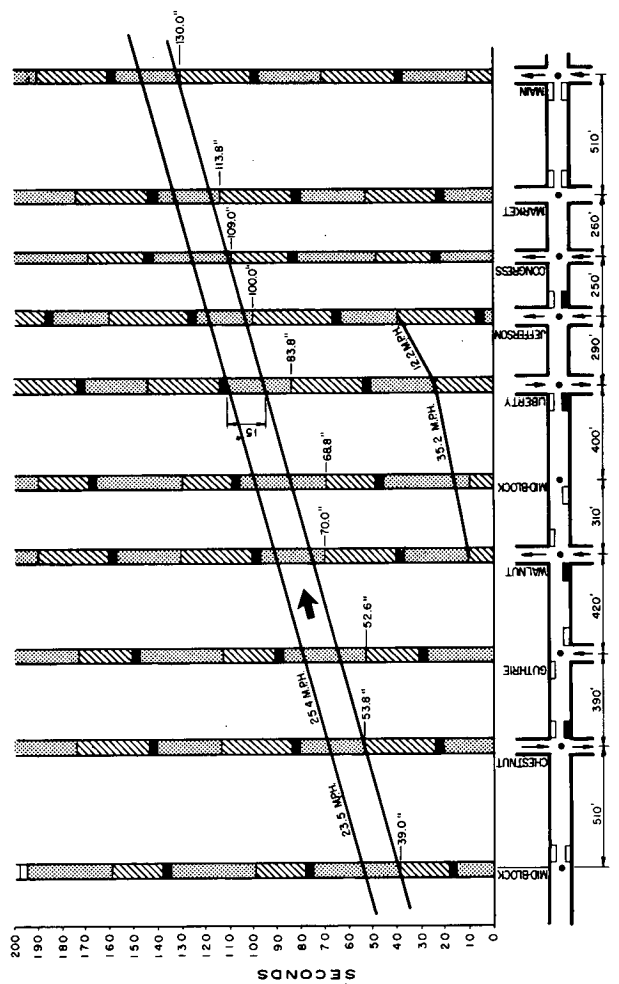


Figure G-210. Signal progression, Fourth Street.

TABLE G-187

OBSERVATIONS OF BUS STOP OPERATIONS, TYPE AND FREQUENCY OF DELAYS OBSERVED

TIME PERIOD	PARKING: PERMITTED — CROSS TRAFFIC: LEFT TO RIGHT — NEAR SIDE (O S T ST)	PARKING: PERMITTED — CROSS TRAFFIC: LEFT TO RIGHT — FAR SIDE (O S T ST)	PARKING: PERMITTED — CROSS TRAFFIC: RIGHT TO LEFT — NEAR SIDE (O S T ST)	PARKING: PERMITTED — CROSS TRAFFIC: RIGHT TO LEFT — FAR SIDE (O S T ST)	PARKING: PROHIBITED — CROSS TRAFFIC: LEFT TO RIGHT — NEAR SIDE (O S T ST)	PARKING: PROHIBITED — CROSS TRAFFIC: LEFT TO RIGHT — FAR SIDE (O S T ST)	PARKING: PROHIBITED — CROSS TRAFFIC: RIGHT TO LEFT — NEAR SIDE (O S T ST)	PARKING: PROHIBITED — CROSS TRAFFIC: RIGHT TO LEFT — FAR SIDE (O S T ST)
	WB BROADWAY AT CAMPBELL	WB BROADWAY AT JACKSON	WB JEFFERSON AT FIFTH	WB JEFFERSON AT THIRD	NB FOURTH AT LIBERTY	NB FOURTH AT CHESTNUT	NB FOURTH AT WALNUT	NB FOURTH AT JEFFERSON
AM			9 9 1 0	6 0 1 0				
Midday	12 5 0 1	9 3 0 0	8 7 0 0	10 1 0 0	12 12 2 0	7 5 1 0	10 8 0 1	21 1 1 7
PM	18 6 0 5	13 2 0 0	5 3 0 2	4 1 0 0				
All	30 11 0 6	22 5 0 0	22 19 1 2	20 2 1 0				
					WB BROADWAY AT CAMPBELL	WB BROADWAY AT JACKSON		
AM					10 5 0 1	14 1 0 1		

O = observations, total; *S* = signal delay, number of occurrences; *T* = traffic delay, number of occurrences; and *ST* = "stalling" delay, number of occurrences.

delay, classified as "stalling," were considered to be necessary to the bus operator's duties. The influence that excessive stalling may have had on the analysis was eliminated by deducting all "stalling" time in excess of 3 sec from the over-all bus stop operation time. Of the 168 observations used in the analysis, 18 included such "stalling" delay, with 12 of these occurrences exceeding 3 sec.

Time periods were selected for their characteristics of differing traffic volumes and bus operations. The morning period included the hours from 7 to 9 AM; the midday period, from 9 AM to 4 PM; and the evening period, from 4 to 6 PM. The observations of bus stop operations are given in Table G-187, which also gives the type and frequency of delays encountered during these observations.

Parking, Permitted; Cross Traffic, Left to Right.—At the bus stops located on Broadway (Fig. G-195), the mean bus stop operation time observed (Table G-188) was greater for the near-side location in both time periods.

The number of bus stop operation times observed with and without signal delay for the midday and PM time periods, as summarized in Table G-187, were analyzed by the chi square method. This analysis indicated that signal delay is not related to stop location.

As the observed frequency of signal delay was 14.0 percent greater for the near-side location, additional observations may well have resulted in a conclusion that location was significant to the frequency of signal delay.

Table G-189 gives the cell means and variances for the stop locations on Broadway for the midday and PM time periods grouped as to stop location, the presence or absence of signal delay, and time period. The fact that stopping

TABLE G-188

MEAN BUS STOP OPERATION TIMES: PARKING, PERMITTED; CROSS TRAFFIC, LEFT TO RIGHT

TIME PERIOD	FAR SIDE (JACKSON ST.) OBS.	FAR SIDE MEAN (SEC)	NEAR SIDE (CAMPBELL ST.) OBS.	NEAR SIDE MEAN (SEC)	DIFFERENCE (SEC)	DIFFERENCE (%)
Midday	9	30.6	12	32.1	+1.5	+4.9
PM	13	21.5	18	32.3	+10.8	+50.2
Both	22	25.2	30	32.2	+7.0	+27.8

TABLE G-189

CELL MEANS AND VARIANCES, BUS STOP OPERATION TIME: PARKING PERMITTED; CROSS TRAFFIC, LEFT TO RIGHT (BUS: WESTBOUND ON BROADWAY)

INTERSECTION LOCATION	STOP LOCATION	SIGNAL DELAY	TIME PERIOD	NO. OF OBS.	CELL MEAN	CELL VARIANCE
Campbell St.	Near	No	Midday	7	24.1	17.14
		No	PM	12	27.6	24.27
Jackson St.	Far	No	Midday	6	21.0	17.20
		No	PM	11	16.9	10.89
Campbell St.	Near	Yes	Midday	5	43.2	88.70
		Yes	PM	6	41.8	55.77
Jackson St.	Far	Yes	Midday	3	49.7	58.33
		Yes	PM	2	46.5	0.50

was prohibited in the north curb lane during the AM time period (Figs. G-200 and G-201) required the values for that period to be analyzed separately under the condition of "parking prohibited."

By inspection, it is evident that two populations are present for each stop location (i.e., those with and those without signal delay). The values in Table G-189 were regrouped as to stop location and presence or absence of signal delay and a two-factor ANOVA was performed. This analysis indicated the following factors to be significant with respect to bus stop operation times:

1. The presence of signal delay as compared to the absence of such delay.
2. The interaction effect of the presence or absence of signal delay with near-side or far-side stop locations.

The effect of this interaction is shown in a plot of the marginal means with Tukey's limits for multiple comparison drawn for the upper 5 percentage points (Fig. G-211). For these two locations, analysis indicated:

1. Bus stop operation time is greater with the presence of signal delay than in the absence of such delay.
2. Bus stop operation time is not significantly different with the presence of signal delay with regard to stop location.
3. Bus stop operation time is 42.9 percent greater for the near-side location than for the far-side location if signal delay is not present.

The presence of signal delay will increase bus stop operation time. Even though it is not statistically significant, it should be noted that the mean bus stop operation time observed in the presence of signal delay was 13.9 percent greater for the far-side location. The presence of signal delay in the over-all bus stop operation may cause more delay at the far-side location, as all of such delay is wasted time, whereas at the near-side location a portion of the delay resulting from signals is frequently used for passenger service operations. This situation can result in less over-all signal delay time for the near-side location for an identical amount of red signal time.

In the absence of signal delay or any other type of delay, as indicated in Table G-187, the greater bus stop operation time for the near-side stop location may reflect greater time losses due to deceleration and acceleration occurring at this particular location.

The observations of bus stop operation times at the Broadway stop locations that included signal delay in the midday and PM time periods, as indicated in Table G-187, were revised by removing the measured signal delays. These revised values were combined with the values of bus stop operation time without signal delay, producing the cell means and variances given in Table G-190. The effects of signal delay time have not been totally eliminated from these values, because the time losses due to acceleration and deceleration from a signal delay could not be measured.

After combining the midday and PM time period data, a t test was performed. This analysis indicated that bus stop operation time less signal delay is significantly less for the far-side stop location than for the near-side stop location.

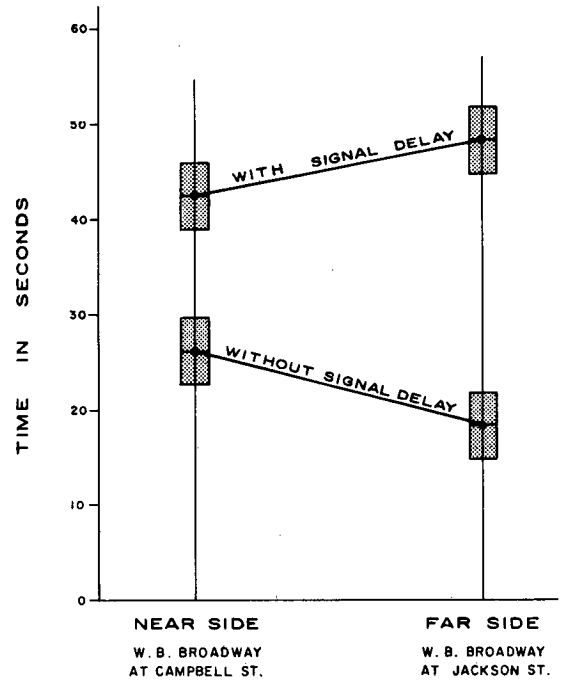


Figure G-211. Bus stop operation time, parking permitted, cross traffic left to right.

TABLE G-190

CELL MEANS AND VARIANCES, BUS STOP OPERATION TIME LESS TRAFFIC-SIGNAL DELAY TIME: PARKING, PERMITTED; CROSS TRAFFIC, LEFT TO RIGHT (BUS: WESTBOUND ON BROADWAY)

INTERSECTION LOCATION	STOP LOCATION	TIME PERIOD	NO. OF OBS.	CELL MEAN	CELL VARIANCE
Campbell St.	Near	Midday	12	25.6	15.54
		PM	18	27.2	31.44
Jackson St.	Far	Midday	9	22.3	57.50
		PM	13	18.5	28.27

In quantitative terms, the mean time of 26.5 sec for the near-side stop location is 24.1 percent greater than the mean time of 20.1 sec for the far-side stop location. In the absence of any measured delays due to other traffic, this difference may again be interpreted to reflect greater time losses at the near-side location due to deceleration and acceleration movements.

Parking, Permitted; Cross Traffic, Right to Left.—At the bus stops located on Jefferson Street (Fig. G-195) the mean bus stop operation time observed (Table G-191) was greater for the near-side location in all time periods.

The number of bus stop operation times observed without signal delay for all time periods (Table G-187) was analyzed by the chi square method. This analysis indicated:

TABLE G-191

MEAN BUS STOP OPERATION TIMES: PARKING, PERMITTED; CROSS TRAFFIC, RIGHT TO LEFT (BUS: WESTBOUND ON JEFFERSON STREET)

TIME PERIOD	FAR SIDE (THIRD ST.) OBS.	FAR SIDE (THIRD ST.) MEAN (SEC)	NEAR SIDE (FIFTH ST.) OBS.	NEAR SIDE (FIFTH ST.) MEAN (SEC)	DIFFERENCE (SEC)	DIFFERENCE (%)
AM	6	24.5	9	38.6	+14.1	+57.6
Midday	10	25.1	8	39.3	+14.2	+56.6
PM	4	30.8	5	33.4	+2.6	+8.4
All	20	26.1	22	37.6	+11.5	+44.1

TABLE G-192

CELL MEANS AND VARIANCES, BUS STOP OPERATION TIME LESS TRAFFIC SIGNAL DELAY TIME: PARKING, PERMITTED; CROSS TRAFFIC, RIGHT TO LEFT (BUS: WESTBOUND ON JEFFERSON STREET)

INTERSECTION LOCATION	STOP LOCATION	TIME PERIOD	NO. OF OBS.	CELL MEAN	CELL VARIANCE
Fifth St.	Near	AM	9	26.1	61.61
		Midday	8	25.6	12.55
		PM	5	24.0	12.50
Third St.	Far	AM	6	24.5	26.70
		Midday	10	22.6	18.04
		PM	4	21.8	34.92

1. Signal delay is related to stop location.
2. The frequency of signal delay is greater for the near-side location than for the far-side location.

One reason why the frequency of signal delay may be greater at the near-side location is that performance of passenger service operations restricts the freedom of departure through the intersection for such buses. Buses using the far-side location are restricted in their passage through the intersection only by the influence of events occurring previous to their arrival at the intersection. Such events may include time of departure from the previous stop location as related to the subsequent arrival at the intersection, progressive signal system operation design, and conflicts with other vehicles encountered en route from the previous stop location.

As only three observations without signal delay were recorded for the near-side stop and two with signal delay for the far-side stop, all individual values of bus stop operation time that included signal delay time were revised by removing the measured signal delays. These revised values were then combined with the values of bus stop operation time without signal delay and grouped as to stop location and time period (Table G-192).

Using the cell means and variances given in Table G-192, a two-factor ANOVA was performed. This analysis indicated that bus stop operation time less signal delay is not significantly different for the far-side stop location than for the near-side stop location.

As only one delay due to other traffic movement was observed at each location, this conclusion may be interpreted to indicate time losses due to deceleration and acceleration movements were not significantly different for either location.

Parking, Prohibited; Cross Traffic, Left to Right.—At the bus stops located on Fourth Street at Liberty and Chestnut Streets, and on Broadway (Fig. G-195) the mean bus stop operation times observed, as given in Table G-193, were greater for the near-side location in both the AM and midday time periods.

The number of bus stop operation times observed on Fourth Street at the foregoing locations with and without signal delay for the midday time period, as summarized in Table G-187, were analyzed by the chi square method. This analysis indicated that signal delay is not related to stop location.

The number of bus stop operation times observed on Broadway with and without signal delay for the AM time period, as summarized in Table G-187, were also analyzed by the chi square method. As previously noted, parking was prohibited at the Broadway locations during the AM time period only. This latter analysis indicated:

1. Signal delay is related to stop location.
2. The frequency of signal delay is greater for the near-side location than for the far-side location.

These two analyses of identical factors are completely contradictory. The fact that the near-side stop location on Broadway was located at the first signalized intersection of an interconnected system for the approach direction may have introduced an additional variable into the data. As no measurements were made regarding this situation, its influence on the data cannot be evaluated.

With regard to the two far-side locations analyzed previously, the influence the preceding stop location may have on a succeeding far-side stop location with regard to approach traffic volumes and the traffic signal progression is of particular interest. Examination of the time-space diagrams showing the signal progressions on Broadway and Fourth Street (Figs. G-208 and G-210, respectively) appears to eliminate signal progression as being totally responsible for the incidence of signal delay observed at the Chestnut Street far-side stop location as compared to the Jackson Street far-side stop location.

At the former location, the preceding bus stop is located on the far side of a mid-block pedestrian traffic signal, 510 ft to the south. Random bus departures from this preceding stop may cause a high percentage of such buses to drop behind the through green signal band, thereby reaching the Chestnut Street intersection after the signal indications have changed to red for northbound movement.

At the latter location, the preceding bus stop is located at the near side of Hancock Street, which is signalized. Buses leaving this stop will probably include a high per-

TABLE G-193

MEAN BUS STOP OPERATION TIMES: PARKING, PROHIBITED; CROSS TRAFFIC, LEFT TO RIGHT

TIME PERIOD	FAR SIDE OBS.	FAR SIDE MEAN (SEC)	NEAR SIDE OBS.	NEAR SIDE MEAN (SEC)	DIFFERENCE (SEC)	DIFFERENCE (%)
	Jackson St.		Campbell St.			
AM	14	22.9	10	32.5	+9.6	+41.9
	Chestnut St.		Liberty St.			
Midday	7	36.3	12	52.3	+16.0	+44.1

TABLE G-194

CELL MEANS AND VARIANCES, BUS STOP OPERATION TIME: PARKING, PROHIBITED; CROSS TRAFFIC, LEFT TO RIGHT

INTERSECTION LOCATION	STOP LOCATION	SIGNAL DELAY	TIME PERIOD	NO. OF OBS.	CELL MEAN	CELL VARIANCE
(*a*) Bus: NB on Fourth St.						
Liberty St.	Near	—	Midday	12	52.3	45.70
Chestnut St.	Far	—	Midday	7	36.3	200.57
(*b*) Bus: WB on Broadway						
Campbell St.	Near	No	AM	5	23.2	7.20
	Near	Yes	AM	5	42.0	87.50
Jackson St.	Far	No	AM	13	20.5	7.44
	Far	Yes	AM	1	53.0	0.00

centage that leave at the beginning of the green phase for Broadway, normally ensuring such buses against a signal delay at Jackson Street.

The cell values for the bus stop operation times observed for the Fourth Street stop locations given in Table G-194 could not be transformed to a state of equal variance. F ratio and median tests were performed on these data without significant conclusions (Table G-195).

The single observation of bus stop operation time with signal delay recorded at the Jackson Street stop location on Broadway resulting in a zero variance for this cell precluded analysis of bus stop operation time alone for the Broadway stop locations.

The individual values of bus stop operation time for the Fourth Street stop locations for the midday time period, less any signal delays, were combined and grouped by location of stop and direction of cross-street traffic flow (Table G-196). For the Liberty Street and Chestnut Street locations, analysis indicated that bus stop operation time less signal delay is not significantly different with regard to stop location.

The cell means and variances for the individual values of bus stop operation time less signal delay for the Broadway stop locations in the AM time period (Table G-196) were subjected to a t test. The results of this analysis indicated that bus stop operation time less signal delay is significantly less at the $\alpha = 0.10$ level for the far stop location than for the near-side stop location.

Again, the two analyses contradict each other. The results from the similar analyses of the data for the locations where parking was permitted were what could be expected. That is, where cross-traffic flow is from left to right, the near-side location would normally be expected to experience more delays due to other traffic movements, such as right turns, compared to the far-side location. Conversely, where cross-traffic flow is from right to left, no significant differences would be expected between stop locations due to movements of other traffic.

As the analysis of the Liberty and Chestnut Street stop locations on Fourth Street contradicts the similar analysis for the Campbell and Jackson Street stop locations on Broadway, it is concluded that individual differences be-

TABLE G-195

F RATIO AND MEDIAN TESTS, BUS STOP OPERATION TIME (BUS: NORTHBOUND ON FOURTH STREET)

ANALYSIS GROUP	INTERSECTION LOCATION	STOP LOCATION	CROSS-STREET FLOW DIRECTION	NO. OF OBS.	MEDIAN	F RATIO	EQUAL VARIANCE	MEDIANS SIGNIFICANTLY DIFFERENT
I	Walnut St.	Near	Right to left	10	52.0	4.945	No	No
	Liberty St.	Near	Left to right	12	50.5			
II	Jefferson St.	Far	Right to left	21	33.0	3.795	No	No
	Chestnut St.	Far	Left to right	7	37.0			
III	Chestnut St.	Far	Left to right	7	37.0	4.389	No	No
	Liberty St.	Near	Left to right	12	50.5			
IV	Walnut St.	Near	Right to left	10	52.0	4.276	No	Yes
	Jefferson St.	Far	Right to left	21	33.0			

TABLE G-196

CELL MEANS AND VARIANCES, BUS STOP OPERATION TIME LESS TRAFFIC SIGNAL DELAY TIME: PARKING, PROHIBITED; CROSS TRAFFIC, LEFT TO RIGHT

INTERSECTION LOCATION	STOP LOCATION	TIME PERIOD	NO. OF OBS.	CELL MEAN	CELL VARIANCE
	(*a*) Bus: NB on Fourth St.				
Liberty St.	Near	Midday	12	25.1	50.08
Chestnut St.	Far	Midday	7	23.9	33.14
	(*b*) Bus: WB on Broadway				
Campbell St.	Near	AM	10	23.8	21.29
Jackson St.	Far	AM	14	21.4	16.25

TABLE G-197

MEAN BUS STOP LOCATION TIMES: PARKING, PROHIBITED; CROSS TRAFFIC, RIGHT TO LEFT

	FAR SIDE (JEFFERSON ST.)		NEAR SIDE (WALNUT ST.)		DIFFERENCE	
TIME PERIOD	OBS.	MEAN (SEC)	OBS.	MEAN (SEC)	(SEC)	(%)
Midday	21	33.0	10	47.8	+14.8	+44.8

tween stop locations and/or factors not measured are responsible for the differing conclusions.

Parking, Prohibited; Cross Traffic, Right to Left.—At the bus stops located on Fourth Street at Walnut and Jefferson Streets (Fig. G-196) the mean bus stop operation times observed (Table G-197) were greater for the near-side location.

The number of bus stop operation times observed with and without signal delay, as summarized in Table G-187, were analyzed by the chi square method. This analysis indicated:

1. Signal delay is related to stop location.
2. The frequency of signal delay is greater for the near-side location than for the far-side location.

This conclusion substantiates that reached for the identical conditions with parking permitted and appears to be logical for the same reasons.

The cell values for the bus stop operation times observed for the Fourth Street stop locations given in Table G-198 could not be transformed to a state of equal variance. *F* ratio tests performed on the variance given in Table G-198, together with median tests for the Walnut and Jefferson Street stop locations (Table G-195) indicated that bus stop operation time is significantly less for the far-side stop location than for the near-side stop location. This conclusion is logical and the result of the greater signal delay experienced at the near-side stop location.

The individual values of bus stop operation time for these locations for the midday time period less any signal delay were combined and grouped by location of stop and direction of cross-street traffic flow (Table G-199).

A two-factor ANOVA was performed on the data for the Fourth Street location in Tables G-196 and G-199. This analysis indicated that the interaction effect of near-side or far-side stop location with cross-street traffic flow from left to right or right to left is significant with respect to bus stop operation time less signal delay.

The effect of this interaction is shown in a plot of the marginal means with Tukey's limits for multiple comparison drawn for the upper 5 percentage points (Fig. G-212). For the Walnut and Jefferson Street locations, analysis indicated that bus stop operation time less signal delay is significantly greater for the far-side stop location than for the near-side stop location.

As the delays due to other traffic, as given in Table G-187, result in a total difference of only 1 sec, this conclusion may be interpreted to indicate time losses due to deceleration and acceleration movements are greater for the far-side location. This conclusion is the exact opposite from that drawn from the analysis for the Broadway stop locations; and, as no parked vehicles were present to hinder the exiting of buses from the far-side stop location on Fourth Street, it appears to indicate individual differences in operat-

TABLE G-198

CELL MEANS AND VARIANCES, BUS STOP OPERATION TIME: PARKING, PROHIBITED; CROSS TRAFFIC, RIGHT TO LEFT

INTERSECTION LOCATION	STOP LOCATION	TIME PERIOD	NO. OF OBS.	CELL MEAN	CELL VARIANCE
Walnut St.	Near	Midday	10	47.8	225.96
Jefferson St.	Far	Midday	21	33.0	52.85

TABLE G-199

CELL MEANS AND VARIANCES, BUS STOP OPERATION TIME LESS TRAFFIC SIGNAL DELAY TIME: PARKING, PROHIBITED; CROSS TRAFFIC, RIGHT TO LEFT

INTERSECTION LOCATION	STOP LOCATION	TIME PERIOD	NO. OF OBS.	CELL MEAN	CELL VARIANCE
Walnut St.	Near	Midday	10	24.0	30.00
Jefferson St.	Far	Midday	21	32.9	53.59

ing conditions at the locations observed were responsible for these differing conclusions.

A further comparison between near-side and far-side stop locations, as shown in Figure G-212, indicated:

1. Bus stop operation time less signal delay for near-side stop locations is not significantly different with regard to the direction of cross-street traffic flow.
2. Bus stop operation time less signal delay for far-side stop locations is greater where cross-street traffic flow is from right to left than where cross-street traffic flow is from left to right.

As no great differences in delays due to other traffic movements were observed, these conclusions are again interpreted as illustrating individual operating differences between bus stops, rather than trends resulting from the direction of cross-street traffic flow.

Conclusions.—The summary of conclusions resulting from these statistical analyses is given in Table G-200.

Although eight bus stops in Louisville were observed for 168 individual bus stop operations under selected operating conditions, delays due to the operation of other traffic were observed in only six instances, precluding a determination of the effects of other traffic movements on bus stop relocation. That these delays do occur in the study area and undoubtedly influence bus stop operations is evidenced by the summary of survey data given in Table G-201. These data were collected in the same area at a different time from the data used in the analyses using somewhat similar methods.

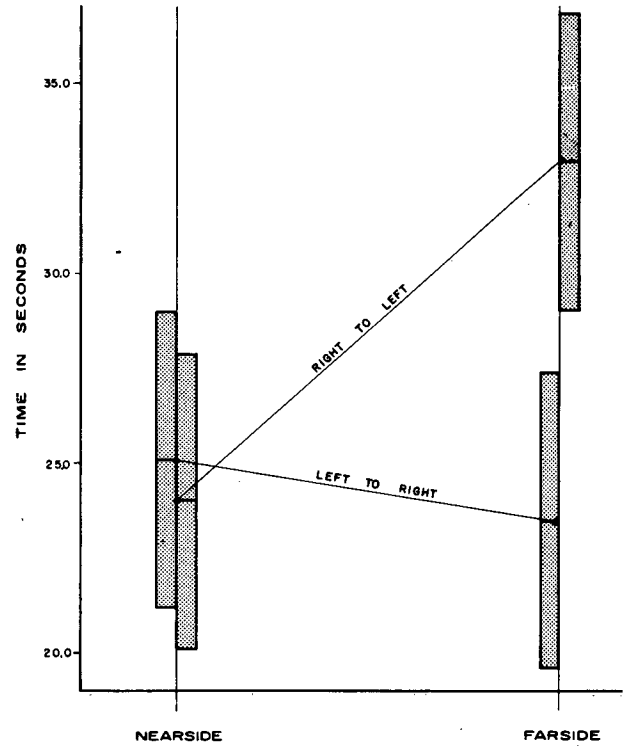


Figure G-212. Bus stop operation time (less signal delay time), parking prohibited.

TABLE G-200

SUMMARY OF ANALYSIS, NEAR-SIDE VERSUS FAR-SIDE BUS STOP LOCATION

FACTOR ANALYZED	PARKING: PERMITTED		PARKING: PROHIBITED		
	CROSS TRAFFIC: LEFT TO RIGHT	CROSS TRAFFIC: RIGHT TO LEFT	CROSS TRAFFIC: LEFT TO RIGHT		CROSS TRAFFIC: RIGHT TO LEFT
Frequency of signal delay (%):			[a]	[b]	
Near side	36.7	86.4	50.0	100.0	80.0
Far side	22.7	10.0	7.7	71.4	5.0
Related to location	No	Yes	Yes	No	Yes
Bus stop operation time	For all observations:				
	Near side > far side	Near side > far side [c]	Near side > far side [c]	Near side > far side [d]	Near side > far side
	When signal delay is present: far side 13.9% > near side				
	When signal delay is not present: near side 42.9% > far side				
Bus stop operation time less signal delay	Near side 24.1% > far side	No significant difference	Near side 10.1% > far side	Near side: No significant difference Far side: R-L > L-R	

[a] Broadway stop locations—AM time period only.
[b] Fourth street stop locations—Midday and PM time periods combined.
[c] Insufficient data for statistical analysis.
[d] Difference not significant.

TABLE G-201

ADDITIONAL OBSERVATIONS OF BUS STOP OPERATIONS, TYPE AND FREQUENCY OF DELAYS OBSERVED

TIME PERIOD	PARKING: PERMITTED				PARKING: PROHIBITED															
	CROSS TRAFFIC: LEFT TO RIGHT, FAR SIDE				CROSS TRAFFIC: LEFT TO RIGHT								CROSS TRAFFIC: RIGHT TO LEFT							
					NEAR SIDE				FAR SIDE				NEAR SIDE				FAR SIDE			
	O	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*	*O*	*S*	*T*	*ST*
AM	18	[a]	1	1	52	35	7	2	64	[a]	0	1	32	15	0	4	16	[a]	0	0
Midday	15	[a]	0	0	78	45	14	6	113	[a]	5	18	38	21	1	0	23	[a]	0	3
PM	11	[a]	0	0	22	14	8	2	45	[a]	1	7	14	7	1	0	12	[a]	0	2
All	44	[a]	1	1	152	94	29	10	222	[a]	6	26	84	43	2	4	51	[a]	0	5
	(1 location)				(6 locations)				(5 locations)				(2 locations)				(1 location)			

O = observations, total;
S = signal delay, number of occurrences;
T = traffic delay, number of occurrences; and
ST = "stalling" delay, number of occurrences.
[a] = Data not collected.

Variations in the time required to decelerate into and accelerate out of bus stops were discovered. This fact, together with the influences that successive stop locations on a street having progressive signal operation may have on bus stop operations, suggests the necessity for additional research that can be best accomplished by evaluating the effects of relocating actual bus stops operating under a variety of conditions.

Specific conclusions reached by statistical analyses of the observed data are:

1. Bus stop operation time was observed to be greater at near-side locations.
2. Signal delay was observed to occur more frequently at near-side stop locations.[1]
3. When signal delay occurred, total delay experienced was greater at far-side locations.[2]
4. When no signal delay occurred, total delay experienced was greater at near-side stop locations.[2]

Safety and Convenience.—The major causes of delay to bus stop operations, other than those due to traffic signals, were observed to be conflict with right-turning vehicles at near-side locations and moving traffic in the adjacent lane for far-side stop locations.

A significant safety advantage of far-side stop locations was observed when alighting passengers desired to cross the street on which the bus stop was located. At near-side stop locations, these passengers walked to the crosswalk at the front of the bus and their passage across the street conflicted with further movement of the bus. At far-side stop locations, the alighting passengers walked to the crosswalk at the rear of the bus where their ability to see approaching traffic was not impaired and where their crossing movements did not interfere with further movement of the bus.

[1]Although this conclusion was not substantiated statistically in all instances examined, the trend was constant.
[2]These conditions were statistically significant only where parking was permitted and cross-street traffic flow was from left to right.

The direction of movement of passengers boarding and alighting from the buses was not observed during this study. This factor, together with that of bus transfers, must be considered in the physical relocation of any bus stop.

Relocated Bus Stop at Broadway and Campbell Street—Experiment F51

Experiment F51 evaluates the effects of relocating a bus stop from a near-side location to a far-side location at the intersection of Broadway and Campbell Street in Louisville (Fig. G-213). This experiment is the extension of the findings of Experiment F64.

Experimental Area

Broadway is a two-way major arterial street bisecting the downtown area of Louisville (Fig. G-214). On the north side of the street, stopping is prohibited between the hours of 7:00 and 9:00 AM and parking is permitted at all other times. Stopping is restricted on the south side of the street between the hours of 4:00 and 6:00 PM; parking is permitted at all other times. Campbell Street is a one-way street permitting travel in a northbound direction. It services local trips in the fringe of the downtown area.

The traffic signal at this intersection is part of the PR system on Broadway and provides for progressive movement on Broadway. Traffic volumes for the morning and afternoon peak hours of an average day in 1968 are shown in Figure G-215.

Design of Experiment

Bus operations at this intersection include a high volume of transfer activity between intersecting routes; the volume of right-turning vehicles from Broadway into Campbell Street is moderate.

Surveillance system for this experiment consisted of:

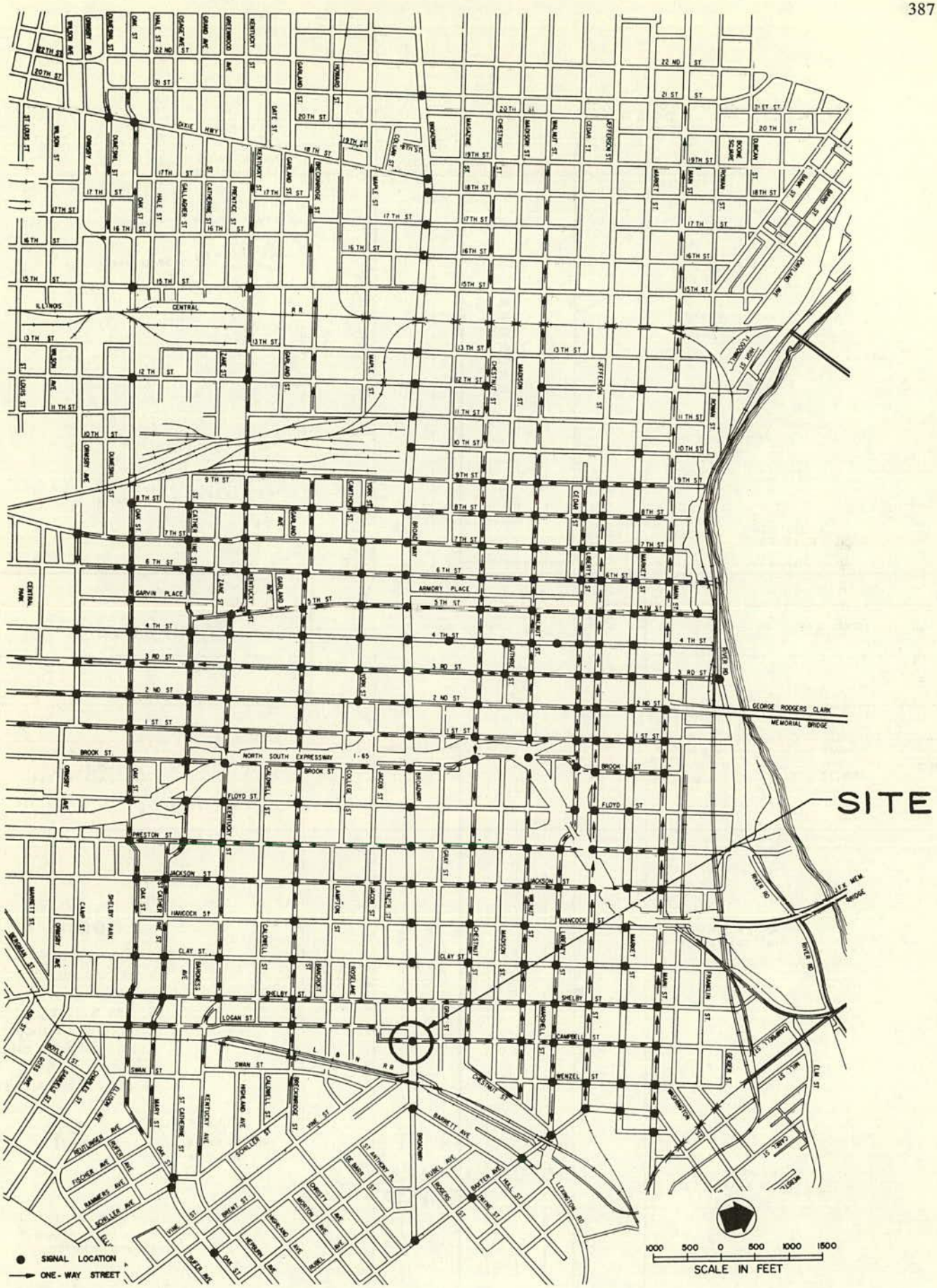


Figure G-213. Location map, Experiment F51.

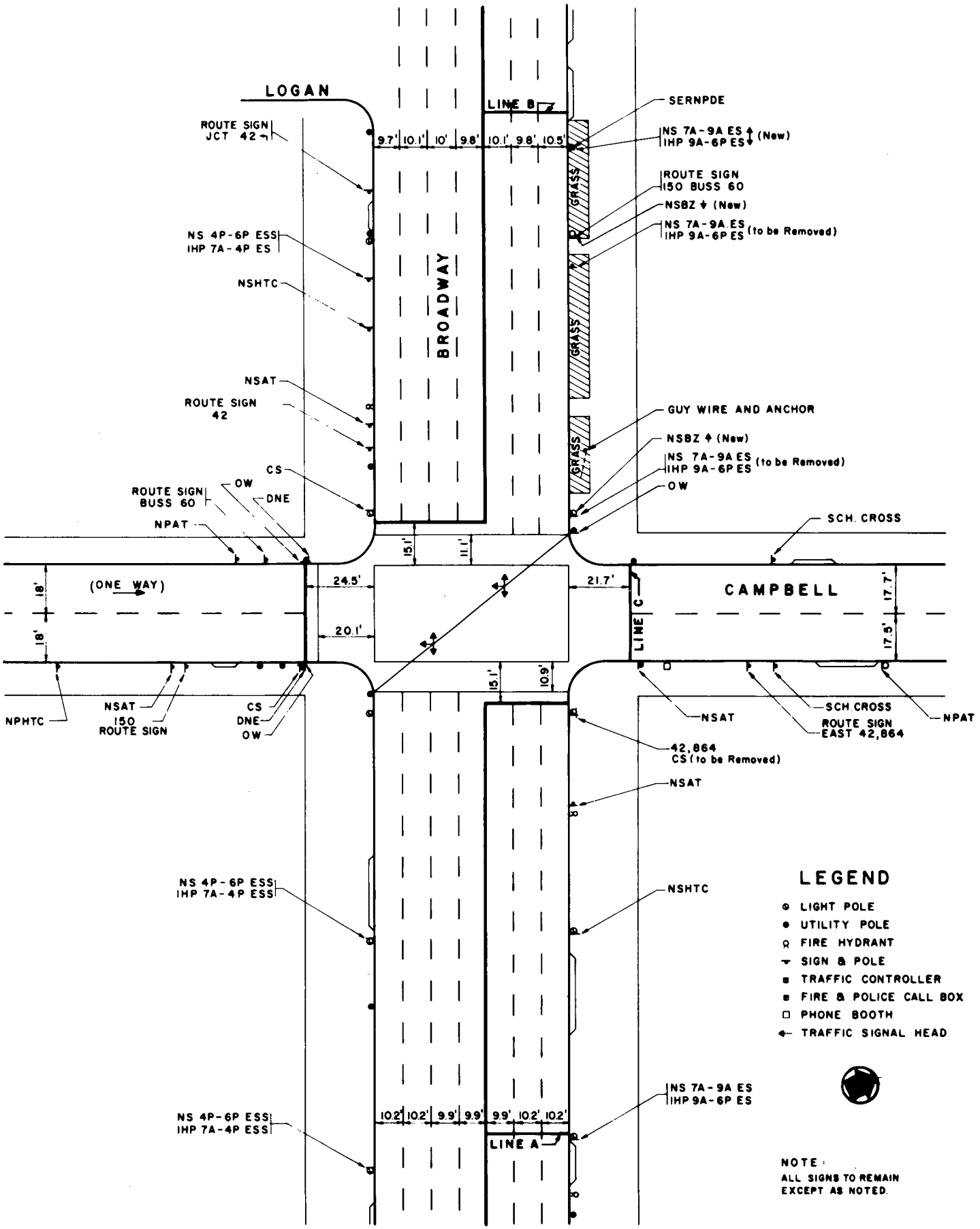


Figure G-214. Vicinity map.

1. Speed and delay runs—runs were made on Broadway between Barrett Avenue and Jackson Street.

2. Travel time measurements—the time required by autos, light trucks, heavy trucks, and buses in the right lane to travel from line A to line C (Fig. G-214) was recorded by stopwatch measurements.

3. Bus stop operation time measurements—the procedures stated in Experiment F64 were followed for this study.

4. Vehicle counts—the number of vehicles stopped per cycle and the number of vehicles through per cycle were recorded by lane for the westbound Broadway approach to Campbell Street. Vehicles were classified as to autos, trucks, and buses.

5. Automatic Traffic Recorder (ATR) equipment counts —ATR counts were taken for 24-hr periods concurrently with other surveys. The two counters used for this experiment were located at lines A and B (Fig. G-214).

The surveillance system was in operation during the following time periods: AM peak—7:30 to 9:30 AM; AM midday—9:30 to 11:30 AM; PM midday—2:00 to 3:30 PM; and PM peak—4:00 to 5:30 PM.

Dates of implementation and measurements are as follows:

PERIOD	DATES
"Before"	3/31/69, 4/1/69, 4/2/69
Implementation	4/4/69
"After"	4/14/69, 4/15/69, 4/16/69

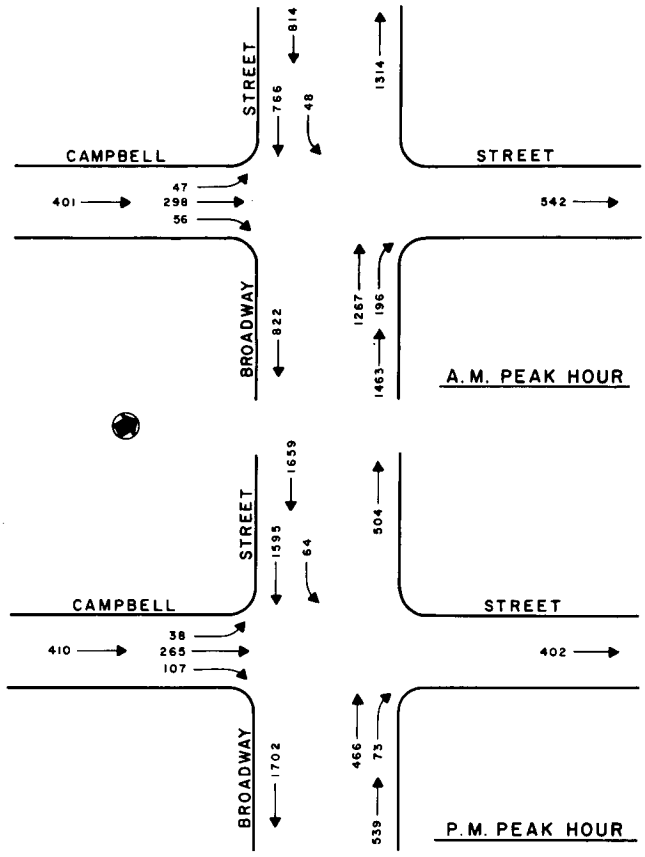


Figure G-215. 1968 AWDT.

Analysis

Travel time information from the speed and delay run data was analyzed to detect differences between the "before" and "after" conditions. Mean values for these data are given in Table G-202. As indicated, there was no significant difference observed for any time period.

Analysis of travel time and delay time information of right-turning vehicles indicated no significant difference between the "before" and "after" measurements.

Analysis of the bus stop operation time indicated no differences between the "before" and "after" measurements except during the PM time period, when signal delay was encountered (Tables G-203 and G-204).

During the AM time period the "after" variance was significantly less than the "before" variance, when signal delay was not present; the converse was true during the PM midday time period, when signal delay was present.

The vehicle count data summarized in Tables G-205 through G-207 are composed of information for all time periods taken on three days. Table G-205 gives the turning

TABLE G-202

SPEED AND DELAY ANALYSIS, TRAVEL TIME, MEAN VALUES

DIRECTION OF TRAVEL	TIME PERIOD	MEAN VALUE (SEC) "BEFORE"	"AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$
EB	AM	130.7	129.8	−0.9(−0.7%)	No
	Midday	125.6	133.7	8.1 (6.4%)	No
	PM	134.9	141.5	6.6 (4.9%)	No
WB	AM	159.1	174.9	15.8 (9.9%)	No
	Midday	156.1	176.0	19.9 (12.7%)	No
	PM	157.1	167.0	9.9 (6.3%)	No

TABLE G-203

BUS STOP OPERATION TIME, MEAN VALUES

TIME PERIOD	OCCURRENCE OF SIGNAL DELAY	MEAN VALUE (SEC) "BEFORE"	"AFTER"	DIFFERENCE	SIG. @ $\alpha=0.05$
AM	Yes	43.5	46.8	3.3 (7.6%)	No
	No	29.3	24.0	−5.3(−18.1%)	No
AM midday	Yes	50.3	44.0	−6.3(−12.5%)	No
	No	30.8	30.2	−0.6 (−1.9%)	No
PM midday	Yes	39.5	40.5	1.0 (2.5%)	No
	No	28.2	29.1	0.9 (3.2%)	No
PM	Yes	42.2	52.7	10.5 (24.9%)	Yes
	No	29.7	31.3	1.6 (5.4%)	No

TABLE G-204

BUS STOP OPERATION TIME, VARIANCES

TIME PERIOD	OCCURRENCE OF SIGNAL DELAY	VARIANCE (SEC) "BEFORE"	"AFTER"	NO. OF OBS. "BEFORE"	"AFTER"	F RATIO	LEVEL OF SIG.
AM	Yes	135.1	94.6	18	13	1.43	NS
	No	64.2	27.8	23	20	2.31	0.05
	Yes	69.6	88.3	7	8	1.27	NS
	No	94.6	52.3	11	15	1.81	NS
PM	Yes	25.1	90.3	6	13	3.60	0.10
	No	63.2	96.5	18	16	1.53	NS
	Yes	51.0	47.8	14	9	1.07	NS
	No	61.3	94.8	27	27	1.55	NS

NS = not significant.

TABLE G-205

VEHICLE COUNTS, BY MOVEMENT

CONDITION	VEHICLES, BY MOVEMENT STRAIGHT	RIGHT TURN	ALL
"Before"	12,232	2,212	14,444
"After"	12,600	2,363	14,963

TABLE G-206

VEHICLE COUNTS, LANE DISTRIBUTION

CONDITION	VEHICLES, BY LANE LEFT	MIDDLE	CURB	ALL
"Before"	5,269	6,889	2,286	14,444
"After"	5,638	7,107	2,218	14,963

movements at the westbound Broadway approach to Campbell Street. A significant increase is indicated in the "after" volume for both the straight and right-turning movements. Analysis of the lane distribution data given in Table G-206 indicates that the increased volumes, given in Table G-205, during the "after" conditions occurred in the left and middle lanes. Table G-207 gives the number of buses in each lane during the "before" and "after" conditions. As indicated, the number of buses in the curb lane was reduced, whereas the number of buses in the middle lane increased.

Conclusions

Based on the analysis of this experiment, it is concluded that:

1. Travel time from speed and delay runs did not change between the "before" and "after" conditions.
2. Travel time and delay time for right-turning vehicles did not change.
3. Mean values of bus stop operation time did not change except for an increase of 10.5 sec (24.9 percent) during the PM time period, with the occurrence of signal delay.
4. Traffic volumes increased in the middle and left lanes

on the westbound Broadway approach to Campbell Street.

5. The number of buses in the curb lane decreased, whereas the number of buses in the middle lane increased.

TABLE G-207

BUS COUNTS, LANE DISTRIBUTION

CONDITION	BUSES, BY LANE			
	LEFT	MIDDLE	CURB	ALL
"Before"	5	64	167	236
"After"	5	113	118	236

Relocated Bus Stop—Fourth Street and Liberty Street—Experiment F53

Experiment F53 evaluates the effect of relocating a bus stop from a near-side location to a far-side location at the intersection of Fourth Street and Liberty Street, in Louisville (Fig. G-216). This experiment is an extension of the findings of Experiment F64.

Experimental Area

Fourth Street is a two-way street serving the major retail establishments in downtown Louisville. On the west side of the street, parking is not permitted between the hours of 7:00 and 11:00 AM; stopping is not permitted between the hours of 11:00 AM and 6:00 PM (Fig. G-217). On the east side of the street, stopping is not permitted between 7:00 and 9:00 AM; parking is prohibited between the hours of 11:00 AM and 6:00 PM. Trucks are permitted to load and unload when parking is prohibited. Liberty Street is a one-way street permitting travel in the eastbound direction. Parking is prohibited on the north side of the street between the hours of 7:00 AM and 4:00 PM; stopping is prohibited between 4:00 and 6:00 PM.

The traffic signal at the Fourth Street and Liberty Street intersection is part of the progressive system on both streets. The signal is set so that 50 percent of the cycle time is allocated to Fourth Street and 50 percent is allocated to Liberty Street. Traffic volumes for the morning and afternoon peak hours of an average weekday in 1968 are shown in Figure G-218.

Design of Experiment

Observations of traffic movements at the intersection of Fourth Street and Liberty Street indicated significant delays due to buses loading and unloading at the near-side stop on Fourth Street. Additional delays were caused by the high volume of vehicles turning right from Fourth Street onto Liberty Street. This movement was frequently interrupted by the high volume of pedestrians using the east crosswalk across Liberty Street.

Traffic flow improvements were made in two phases. Phase 1 consisted of relocating the existing bus stop from its near-side position to a far-side position. Phase 2 consisted of the installation of overhead and curb-mounted signs and pavement arrows that indicated that curb-lane traffic must turn right. After the installation of Phase 1, by order of the Louisville Transit Company buses were not permitted to use the left lane to bypass right-turning vehicles. Transit officials, concerned about safety, wanted to avoid the condition where a bus would be in the left lane and a vehicle would proceed through the intersection in the curb lane, thereby preventing the bus from entering the far-side position. Buses traveled in the left lane after the installation of the Phase 2 improvements.

The surveillance system for this experiment included:

1. Speed and delay runs—runs were made on Fourth Street between Broadway and Main Street.

2. Travel time measurements—the time required by autos, light trucks, heavy trucks, and buses in the right lane to travel from line A to line C (Fig. G-217) was recorded by stopwatch measurements.

3. Bus stop operation time measurements—the procedure stated in Experiment F64 was followed for this study.

4. Vehicle counts—number of vehicles stopped per cycle and the number of vehicles through per cycle were recorded by lane for the northbound Fourth Street approach to Liberty Street. Vehicles were classified as to autos, trucks, and buses.

5. Automatic Traffic Recorder (ATR) equipment counts —ATR counts were taken for 24-hr periods concurrently with other surveys. Two counters were used for this experiment and were located at lines A and B (Fig. G-217).

The surveillance system was in operation during the following time periods: AM—7:30 to 9:00 AM; AM midday—9:30 to 11:00 AM; PM midday—2:00 to 3:30 PM; and PM—4:00 to 5:30 PM.

Dates of implementation and measurements are as follows:

CONDITION	DATES
Data group 1	3/26/69, 3/27/69, 3/28/69
Phase 1—implementation	4/1/69
Data group 2	4/6/69, 4/10/69, 4/11/69
Phase 2—implementation	4/16/69
Data group 3	4/23/69, 4/24/69, 4/25/69

Analysis

Travel time information from the speed and delay run data was analyzed for the northbound and southbound directions of travel on Fourth Street. Mean values and variances are given in Tables G-208 and G-209. As indicated after the implementation of Phase 1, travel time for the northbound direction was significantly increased during the midday period and significantly reduced during the PM period. For the southbound direction, travel time was significantly increased during the AM period and significantly decreased during the PM time period. With the implementation of Phase 2, PM travel times were further reduced for an over-

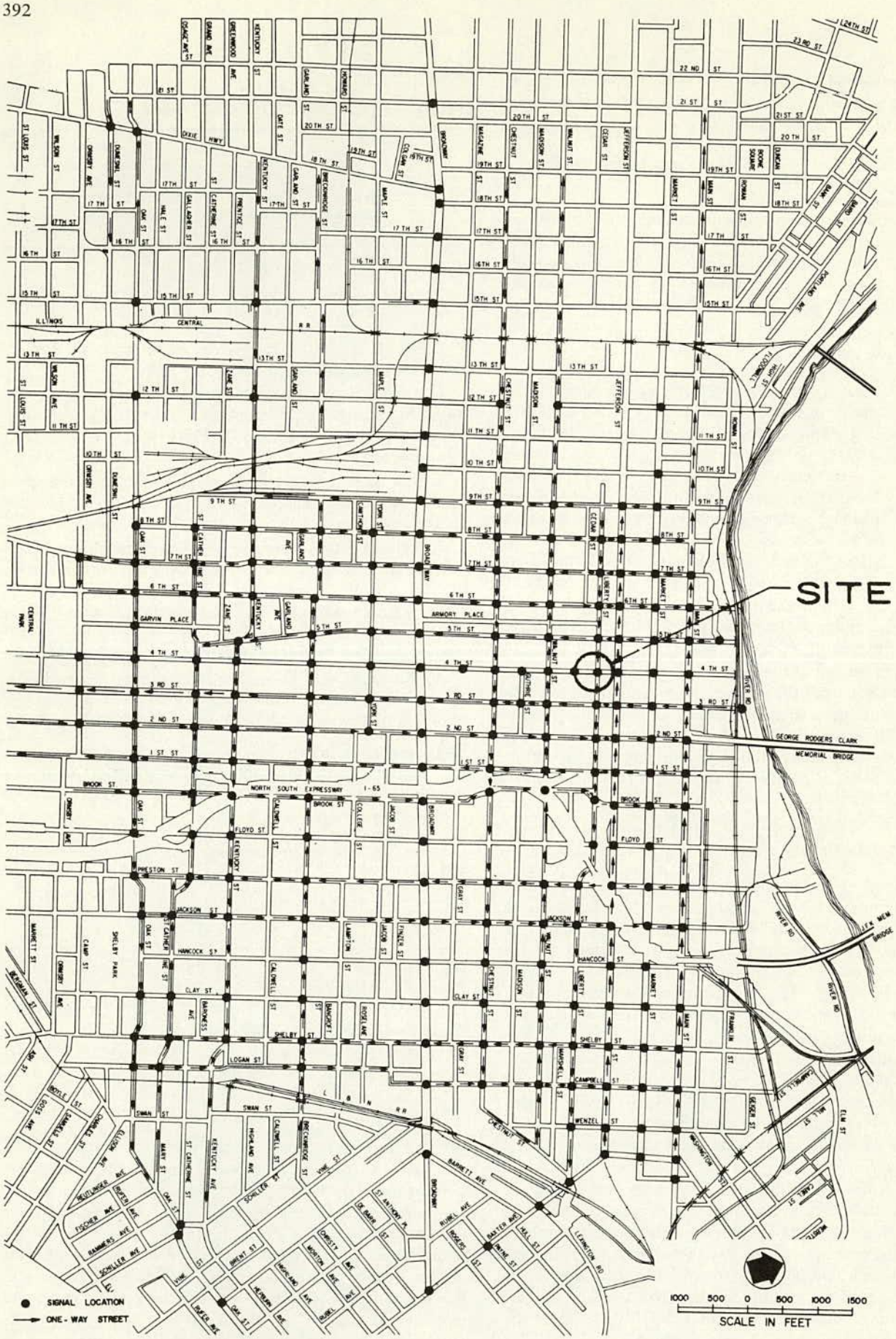


Figure G-216. Location map, Experiment F53.

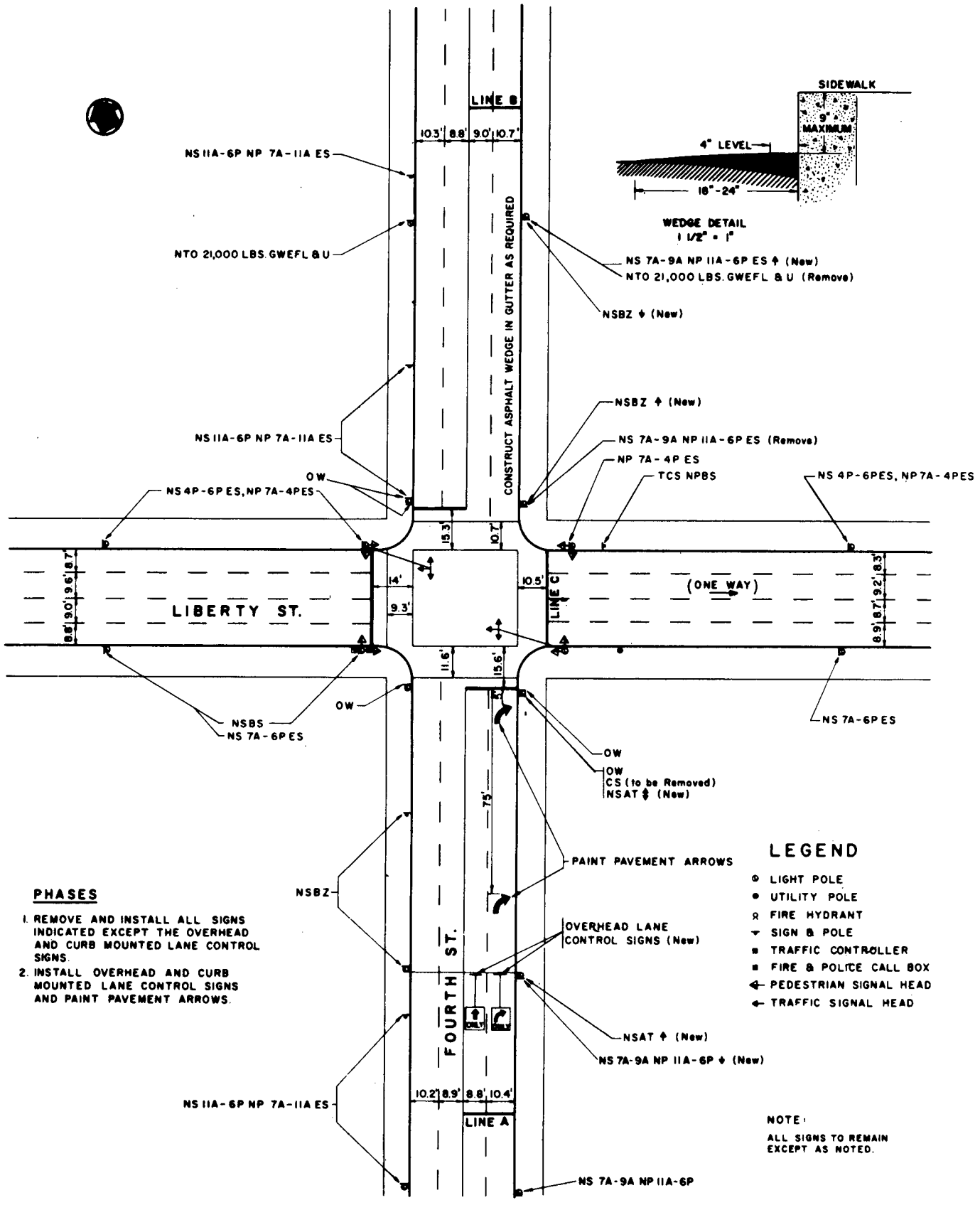


Figure G-217. Vicinity map.

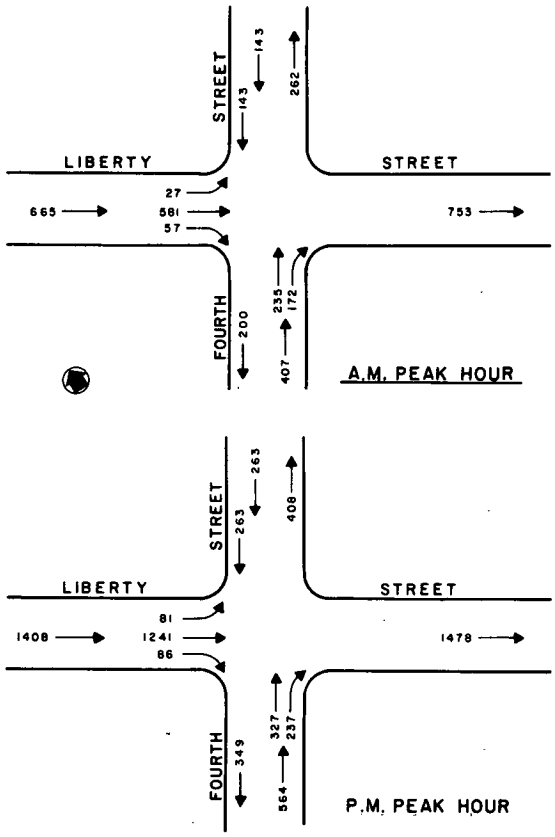


Figure G-218. 1968 AWDT.

all time saving of 76.2 sec for the northbound direction and 48.2 sec for the southbound direction. The results of this analysis were expected, because relocation of the bus stop in Phase 1 without the use of the left lane by buses would tend to increase over-all travel times. Because the greatest traffic hour occurred during the PM peak period, it was expected that the greatest change would be observed during that time period. A significant reduction in the variances between data groups 1 and 3 was observed, as given in Table G-209.

The absence or presence of signal delay was considered in the travel time analysis for right-turning vehicles. Tables G-210 and G-211 list the means and variance values for these data, respectively. There was no change in the mean values between data groups for turning vehicles when they encountered no signal delay. During the midday time periods travel time was significantly increased between data groups 1 and 2 and data groups 2 and 3 for vehicles that encountered signal delay. A significant reduction was noted between data groups 1 and 3 for the PM midday period and the PM peak period. This trend is the same as indicated for the travel time data (i.e., an increase in travel time between data groups 1 and 2 and a reduction in time between data groups 2 and 3). Comparison of the variance values in Table G-211 indicated a significant increase during the AM midday period between data groups 1 and 2; significant decreases for the AM midday and both PM periods between data groups 2 and 3; and, again, a decrease for all time periods between data groups 1 and 3.

Delay times for right-turning vehicles (Tables G-212 and G-213) indicate similar results, as described previously.

Bus stop operation time was recorded for two conditions: (1) when signal delay was present, and (2) when signal delay was not present. Mean values and variance values for this analysis are given in Tables G-214 and G-215. As indicated, the implementation of Phase 1 significantly reduced the bus stop operation time for three of the four time periods when signal delay was not present. When signal delay was present, bus stop operation time was significantly increased in all four time periods. The changes in variability were mixed, as given in Table G-215.

After the implementation of Phase 2, bus stop operations without the occurrence of signal delay indicated no significant difference between data groups. However, three of the four time periods were significantly different when signal delay was present. The over-all effect between data groups 1 and 3 indicated no significant difference when signal delay occurred and significant reductions in the AM midday, PM midday, and PM peak time periods when sig-

TABLE G-208

SPEED AND DELAY ANALYSIS, MEAN VALUES

TIME PERIOD	DIRECTION OF TRAVEL	MEAN VALUE (SEC), BY DATA GROUP 1	2	3	DIFFERENCE BETWEEN DATA GROUPS: 1 AND 2	SIG. @ $\alpha=0.05$	2 AND 3	SIG. @ $\alpha=0.05$	1 AND 3	SIG. @ $\alpha=0.05$
AM	NB	161.5	174.1	176.8	12.6 (7.8%)	No	2.7 (1.6%)	No	15.3 (9.5%)	No
Midday		301.6	364.5	279.5	62.9 (20.9%)	Yes	−85.0 (−23.3%)	Yes	−22.1 (−7.3%)	No
PM		314.5	284.9	238.3	−29.6 (−9.4%)	Yes	−46.6 (−16.4%)	Yes	−76.2 (−24.2%)	Yes
AM	SB	285.2	319.1	288.1	33.9 (11.9%)	Yes	−31.0 (−9.7%)	Yes	2.9 (1.0%)	No
Midday		356.8	371.8	342.0	15.0 (4.2%)	No	−29.8 (−8.0%)	Yes	−14.8 (−4.1%)	No
PM		377.8	349.8	329.6	−28.0 (−7.4%)	Yes	−20.2 (−5.8%)	No	−48.2 (−12.8%)	Yes

TABLE G-209

SPEED AND DELAY ANALYSIS, VARIANCES

DIRECTION OF TRAVEL	TIME PERIOD	VARIANCE (SEC), BY DATA GROUP 1	2	3	NO. OF OBS., BY DATA GROUP 1	2	3	COMPARISON OF VARIANCES BETWEEN DATA GROUPS: 1 AND 2 F RATIO	2 AND 3 F RATIO	1 AND 3 F RATIO
NB	AM	1036.6	1179.4	475.8	17	15	18	1.14	2.48[b]	2.18[a]
	Midday	3765.8	12559.9	2160.6	25	21	30	3.34[b]	5.81[b]	1.74[a]
	PM	5410.5	3848.6	1947.9	12	15	15	1.41	1.98	2.78[b]
SB	AM	568.6	1495.8	943.5	18	17	18	2.63[b]	1.59	1.66
	Midday	6044.5	3543.6	2261.1	27	27	32	1.71[a]	1.57	2.67[b]
	PM	3960.0	1540.4	617.5	14	14	15	2.57[b]	2.49[a]	6.41[b]

[a] Significant at $0.10 \geq \alpha > 0.05$.
[b] Significant at $\alpha = 0.05$.

TABLE G-210

TRAVEL TIME FOR RIGHT-TURNING VEHICLES, MEAN VALUES

TIME PERIOD	OCCURRENCE OF DELAY	MEAN VALUE (SEC), BY DATA GROUP 1	2	3	DIFFERENCE BETWEEN DATA GROUPS: 1 AND 2	SIG. @ $\alpha=0.05$	2 AND 3	SIG. @ $\alpha=0.05$	1 AND 3	SIG. @ $\alpha=0.05$
AM peak	Yes	31.1	34.0	28.7	2.9 (9.3%)	No	−5.3 (−15.6%)	No	−2.4 (−7.7%)	No
	No	10.8	10.8	10.7	0.0 (0.0%)	No	−0.1 (−0.9%)	No	−0.1 (−0.9%)	No
AM midday	Yes	37.3	46.4	38.7	9.1 (24.4%)	Yes	−7.7 (−16.6%)	Yes	1.4 (3.8%)	No
	No	12.9	12.3	11.6	−0.6 (−4.7%)	No	−0.7 (−5.7%)	No	−1.3 (−10.1%)	No
PM midday	Yes	41.5	50.4	34.6	8.9 (21.4%)	Yes	−15.8 (−31.3%)	Yes	−6.9 (−16.6%)	Yes
	No	12.6	12.9	12.7	0.3 (2.4%)	No	−0.2 (−1.6%)	No	0.1 (0.8%)	No
PM peak	Yes	54.4	55.0	41.4	0.6 (1.1%)	No	−13.6 (−24.7%)	Yes	−13.0 (−23.9%)	Yes
	No	13.5	15.5	14.2	2.0 (14.8%)	No	−1.3 (−8.4%)	No	0.7 (5.2%)	No

TABLE G-211

TRAVEL TIME FOR RIGHT-TURNING VEHICLES, VARIANCES

TIME PERIOD	OCCURRENCE OF DELAY	VARIANCE (SEC), BY DATA GROUP 1	2	3	NO. OF OBS., BY DATA GROUP 1	2	3	COMPARISON OF VARIANCES BETWEEN DATA GROUPS: 1 AND 2 F RATIO	2 AND 3 F RATIO	1 AND 3 F RATIO
AM peak	Yes	260.8	212.8	155.5	89	78	71	1.23	1.37	1.68[a]
	No	13.2	11.3	13.6	138	157	158	1.17	1.20	1.03
AM midday	Yes	256.2	458.9	250.7	103	104	92	1.79[a]	1.83[a]	1.02
	No	15.1	11.3	9.8	76	60	96	1.34	1.15	1.54[a]
PM midday	Yes	565.4	584.1	311.1	138	153	129	1.03	1.88[a]	1.82[a]
	No	15.9	21.6	16.0	58	23	100	1.36	1.35	1.01
PM peak	Yes	563.7	609.4	407.9	142	141	155	1.08	1.49[a]	1.38[a]
	No	20.1	27.3	27.7	36	30	42	1.36	1.01	1.38

[a] Significant at $\alpha = 0.05$.

TABLE G-212

DELAY TIME FOR RIGHT-TURNING VEHICLES, MEAN VALUES

TIME PERIOD	MEAN VALUE (SEC), BY DATA GROUP 1	2	3	DIFFERENCE BETWEEN DATA GROUPS (SEC) 1 AND 2	SIG. @ $\alpha=0.05$	2 AND 3	SIG. @ $\alpha=0.05$	1 AND 3	SIG. @ $\alpha=0.05$
AM peak	16.8	18.2	13.5	1.4 (8.3%)	No	−4.7(−25.8%)	No	−3.3(−19.6%)	No
AM midday	19.6	24.9	19.5	5.3(27.0%)	Yes	−5.4(−21.7%)	Yes	−0.1 (0.5%)	No
PM midday	22.6	30.8	18.7	8.2(36.3%)	Yes	−12.1(−39.3%)	Yes	−3.9(−17.3%)	Yes
PM peak	32.1	35.6	23.2	3.5(10.9%)	No	−12.4(−34.8%)	Yes	−8.9(−27.7%)	Yes

TABLE G-213

DELAY TIME FOR RIGHT-TURNING VEHICLES, VARIANCES

TIME PERIOD	VARIANCE (SEC), BY DATA GROUP 1	2	3	NO. OF OBS., BY DATA GROUP 1	2	3	COMPARISON BETWEEN DATA GROUPS 1 AND 2 *F* RATIO	2 AND 3 *F* RATIO	1 AND 3 *F* RATIO
AM peak	227.0	175.4	127.7	89	78	71	1.29	1.37	1.78[b]
AM midday	200.9	292.5	174.2	103	104	92	1.46[a]	1.68[b]	1.15
PM midday	388.9	434.1	237.1	138	153	129	1.12	1.83[b]	1.64[b]
PM peak	400.3	473.8	309.0	142	141	155	1.18	1.53[b]	1.30[a]

[a] Significant at $0.10 \geqq \alpha > 0.05$.
[b] Significant at $\alpha = 0.05$.

nal delay did not occur. The variances, given in Table G-215, indicate substantial reductions between data groups 1 and 3 for many of these time periods.

Analysis of the vehicle counts is summarized in Tables G-216 through G-219. These represent data for three days of counts taken during all time periods. Table G-216 gives the turning movements of all vehicles. Analysis of this table indicated that the straight-through movement for data group 3 was significantly less than the straight movements for the other two data groups. Analysis of the lane distribution (Table G-217) indicated that curb lane use for data group 3 was significantly lower than similar use for data groups 1 and 2. Although the number of right turns from each lane remained relatively constant for each data group, the number of vehicles proceeding straight through the intersection showed a significant shift in lane use. Table G-218 gives the number of turning vehicles proceeding through the intersection in each lane. It is evident that the

TABLE G-214

BUS STOP OPERATION TIME, MEAN VALUES

TIME PERIOD	OCCURRENCE OF SIGNAL DELAY	MEAN VALUE (SEC), BY DATA GROUP 1	2	3	DIFFERENCE BETWEEN DATA GROUPS 1 AND 2	SIG. @ $\alpha=0.05$	2 AND 3	SIG. @ $\alpha=0.05$	1 AND 3	SIG. @ $\alpha=0.05$
AM peak	Yes	51.9	60.0	55.8	8.1 (15.6%)	Yes	−4.2 (−7.0%)	No	3.9 (7.5%)	No
	No	24.6	25.7	25.5	1.1 (4.5%)	No	−0.2 (−0.8%)	No	0.9 (3.7%)	No
AM midday	Yes	55.9	73.7	55.7	17.8 (31.8%)	Yes	−18.0 (−24.4%)	Yes	0.2 (0.4%)	No
	No	41.7	28.8	30.5	−12.9 (−30.9%)	Yes	1.7 (5.9%)	No	−11.2 (−26.9%)	Yes
PM midday	Yes	57.0	77.3	56.2	20.3 (35.6%)	Yes	−21.1 (−27.3%)	Yes	−0.8 (−1.4%)	No
	No	39.8	24.6	25.6	−15.2 (−38.2%)	Yes	1.0 (4.1%)	No	−14.2 (−35.7%)	Yes
PM peak	Yes	53.0	84.1	57.8	31.1 (58.7%)	Yes	−26.3 (−31.3%)	Yes	4.8 (9.1%)	No
	No	40.9	28.2	27.0	−12.7 (−31.1%)	Yes	−1.2 (−4.3%)	No	−13.9 (−34.0%)	Yes

TABLE G-215

BUS STOP OPERATION TIME, VARIANCES

TIME PERIOD	OCCURRENCE OF SIGNAL DELAY	VARIANCE (SEC), BY DATA GROUP 1	2	3	NO. OF OBS., BY DATA GROUP 1	2	3	COMPARISON BETWEEN DATA GROUPS: 1 AND 2 F RATIO	2 AND 3 F RATIO	1 AND 3 F RATIO
AM peak	Yes	84.4	38.5	7.8	48	27	21	2.19[b]	4.94[b]	10.82[b]
	No	13.2	12.2	14.5	16	44	51	1.08	1.19	1.10
AM midday	Yes	239.4	476.0	91.5	27	16	6	1.99[a]	5.20[b]	2.62
	No	257.1	50.2	38.1	6	12	24	5.12[b]	1.32	6.75[b]
PM midday	Yes	626.6	310.7	59.6	24	24	16	2.02[b]	5.21[b]	10.51[b]
	No	338.9	22.8	15.9	4	5	28	14.86[b]	1.43	21.31[b]
PM peak	Yes	317.7	444.3	63.7	24	32	16	1.40	6.97[b]	4.99[b]
	No	323.9	39.4	42.6	9	6	26	8.22[b]	1.08	7.60[b]

[a] Significant at $0.10 \geq \alpha > 0.05$.
[b] Significant at $\alpha \leq 0.05$.

TABLE G-216

VEHICLE COUNTS, BY MOVEMENT

DATA GROUP	VEHICLES, BY MOVEMENT: STRAIGHT	RIGHT TURN	ALL
1	5,331	3,739	9,070
2	5,271	3,945	9,216
3	4,836	3,631	8,467

TABLE G-217

VEHICLE COUNTS, LANE DISTRIBUTION

DATA GROUP	VEHICLES, BY LANE: LEFT	CURB	ALL
1	4,855	4,215	9,070
2	4,867	4,349	9,216
3	4,710	3,757	8,467

TABLE G-218

VEHICLES, BY MOVEMENT

DATA GROUP	LEFT LANE: STRAIGHT	LEFT LANE: RIGHT TURN	CURB LANE: STRAIGHT	CURB LANE: RIGHT TURN	ALL
1	4,796	59	535	3,680	9,070
2	4,805	62	466	3,883	9,216
3	4,647	63	189	3,568	8,467

TABLE G-219

BUS COUNTS, LANE DISTRIBUTION

DATA GROUP	BUSES, BY LANE: LEFT	CURB	ALL
1	80	285	365
2	92	259	351
3	361	2	363

implementation of Phase 1 had no effect on the number of vehicles in each lane. However, the implementation of Phase 2, which made the curb lane a mandatory right-turn lane, reduced the number of vehicles proceeding straight through from the curb lane. This decrease was not reflected in an increase in the vehicles in the left lane, indicating a reduction in demand. This reduction of the demand was verified by the ATR counts. The number of buses in each lane is given in Table G-219. As expected, buses did not shift from the curb lane with the implementation of Phase 1. After the implementation of Phase 2, all but two buses shifted to the left lane. As previously described, a significant reduction in travel time for right-turning vehicles was observed with the implementation of Phase 2, when signal delay was present during the AM time periods.

Conclusions

Based on the results of this analysis, it is concluded that:

1. From the speed and delay run data, travel time during the PM period was significantly reduced with the implementation of Phase 1 and further reduced with the implementation of Phase 2.
2. When signal delay was present, Phase 1 improvements resulted in: (1) increased travel time and delay time for right-turning vehicles during the midday time periods, and (2) increased bus stop operation time for all time periods.
3. When signal delay was not present, Phase 1 improvements resulted in reduced bus stop operation time for all time periods except the AM peak period.
4. When signal delay was present, Phase 2 improvements resulted in reduced travel time and delay time for right-turning vehicles during the PM midday, and PM peak periods.
5. When signal delay was not present Phase 2 improvements resulted in reduced bus stop operation time for the AM midday, PM midday, and PM peak time periods.
6. The number of vehicles traveling straight through the intersection, the number of vehicles in the curb lane, and the number of buses in the curb lane were reduced after the implementation of Phase 2.

Market Street Bus Operations—Experiment C110

Experiment C110 evaluates changes in bus operations on Market Street between Mulberry and Washington Streets in Newark. Specifically, it was designed to:

1. Reduce delays to buses and other vehicles.
2. Improve the quality of bus service.
3. Increase capacity.
4. Improve safety for bus passengers.

The work included revisions to bus transit operations, the manual placement of traffic cones and signs, erection of permanent signs, and modifications to traffic signal operations.

Experimental Area

Market Street is a major east-west arterial street located in Newark's CBD (Fig. G-219). In the experimental area it is 59 ft wide, with pavement marking delineating five traffic lanes. The experimental area is the origin and destination of a considerable portion of the daily bus transit passengers in the CBD.

Traffic signal controls at the intersection of Mulberry, Beaver, Broad, Halsey, and Washington Streets provided a basically simultaneous offset operation that was not changed during this experiment. The signal operation at Washington Street, however, was revised, as described in Experiment A33, and resulted in a 4.1-percent decrease in green time per cycle for westbound Market Street traffic.

Design of Experiment

The high concentration of westbound bus transit operations in the PM peak period caused an historical record of delays and congestion. On at least two occasions, transit company and city officials had revised transit and traffic operations to alleviate this problem. The original revision had resulted in the north curb lane being designated for the exclusive use of buses between 4:00 and 6:00 PM. This regulation had been in continual effect for many years and was not revised for this experiment. In its original application, all westbound buses were required to operate in this lane. The resulting operations were described by one transit company official as permitting ". . . a person to walk on the tops of stopped buses from Mulberry Street to Washington Street without stopping or touching the pavement." The delays to transit operations due to all buses moving at a rate controlled by the slowest bus were so excessive that a different plan was tried.

With this second plan, the westbound bus routes were divided into groups with equal numbers of boarding passengers. One group of routes performed passenger service operations in the second lane; the remaining group continued operations in the curb lane. The effect of this revision was to halve the delays experienced under the prior plan. Buses assigned to a particular lane continued to move at a rate controlled by the slowest bus in that lane. A new element of hazard was introduced with this plan due to the necessity for bus passengers to board and alight from the street for the second-lane operation. This hazard was partially overcome by placing the lane line separating the curb and second lane 17 ft from the curb face and using the excess lane width to delineate an "in the street" bus loading area. The remainder of the street was divided into three additional lanes by pavement markings, resulting in two lanes for eastbound traffic and three lanes for westbound traffic. Parking, stopping, and standing were prohibited in both curb lanes. This arrangement was necessary to reduce delays to other traffic movements. As parking was permitted in the south curb lane at all times except the AM and PM peak traffic periods, the normal direction of flow in the center lane was eastbound. Reversal of flow direction in this lane was accomplished without the use of signs, signals, or other devices, providing an element of continual potential hazard. This "dual lane" transit operation was placed into effect on December 19, 1956, and was an immediate success as far as transit operations were concerned. Some details of the "in the street" bus loading area and the operation at the time of its inauguration are shown in Figures G-220 and G-221.

By 1969 the closely spaced traffic cones (Fig. G-220) were no longer in use; the buses in the curb lane were observed to maneuver in and out of that lane in an effort to expedite their travel through the area. In addition, protection for the "in the street" bus loading area had been reduced to a few portable signs connected by chains. These were placed only in the immediate approach area to the intersections of Broad, Halsey, and Washington Streets. Figure G-222, taken in May 1969, shows bus passengers waiting in the street with minimal protection against buses and other vehicles. These were the "before" conditions for this experiment. Details of the pavement marking and the curb- and second-lane bus route grouping are shown in Figure G-223.

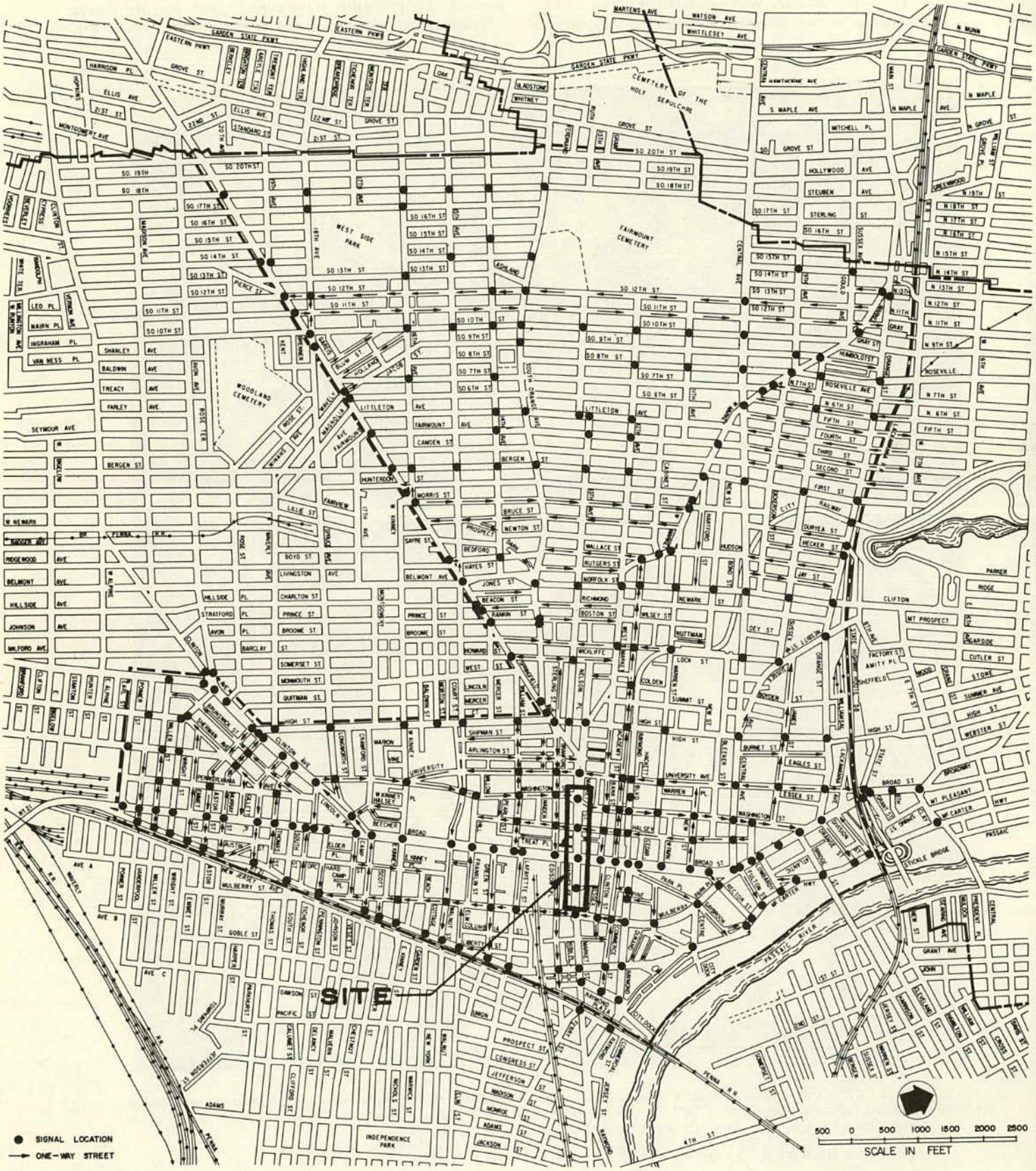


Figure G-219. Location map, Experiment C110.

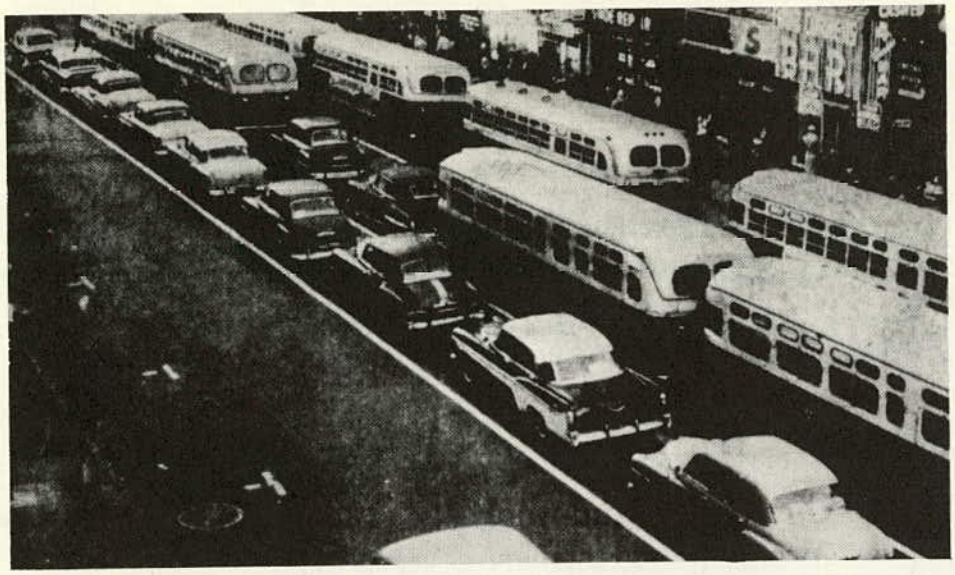
Early in this project, delays of considerable magnitude to buses and other vehicles were observed, as well as the potential hazard to bus passengers waiting in the street. Two surveys were made to measure the magnitude of the problem and to qualify the elements producing the delays.

On Tuesday, December 19, 1967, a survey was con-

MARKET ST. LANES—New traffic lanes on Market St. ease congestion during rush hours. "A" marks lane exclusively reserved for cars and trucks; "B" is outside bus lane; "C" marks pedestrian "safety zone" to use buses in "B" lane; "D" is second bus lane, for vehicles to be boarded from sidewalk.

NEWARK STAR-LEDGER Thursday, Dec. 20, 1956

Figure G-220. "Dual-lane" transit operations, December 20, 1956.



TRAFFIC FUNNEL—Westbound traffic in Market St. waits for change of light under new evening rush-hour set up.

Figure G-221. "Dual-lane" transit operation (Newark Sunday News, *January 6, 1956*).

Figure G-222. "Dual-lane" transit operations, May 1969.

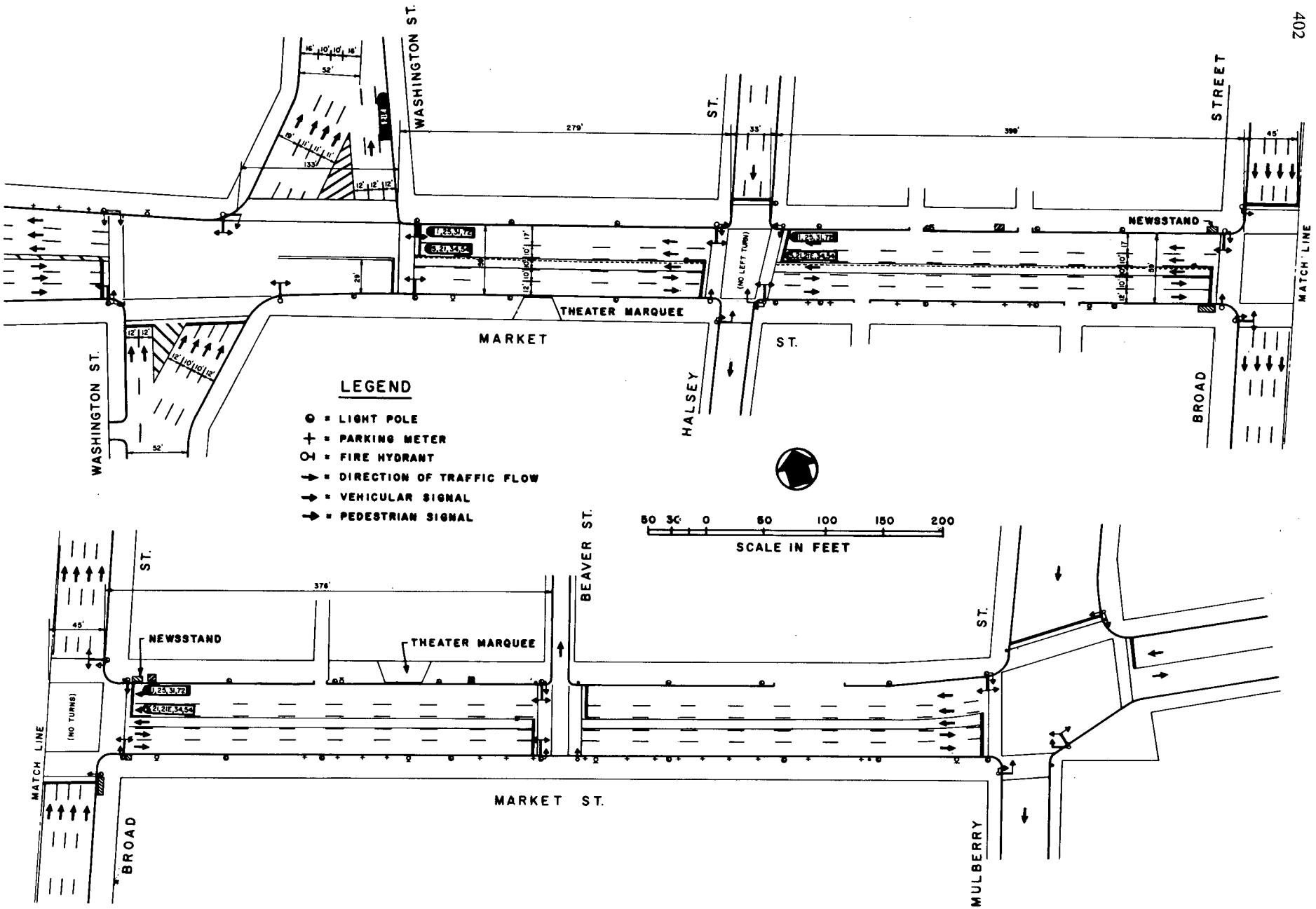


Figure G-223. Vicinity map, "before."

ducted at the intersections of Market Street with Broad, Halsey, and Washington Streets. Westbound buses that were stopped on the red signal and those that proceeded through were counted at each location. Beginning at 4:00 PM, 80 consecutive signal cycles were counted in approximately 2 hr. Table G-220 gives the results of this survey. These data indicate that more than 100 buses left each intersection between 4:30 and 5:30 PM, whereas more than 200 buses left the area during the 4:00 to 6:00 PM period. The Broad Street intersection was the major source of delay to transit operations, with twice as many buses stopped by a red signal indication as were cleared during the following green signal interval.

On Monday, December 9, 1968, an additional survey was conducted on Market Street between Beaver and Broad Streets from 4:00 until 6:00 PM. During this survey the arrival time of each bus at Beaver Street and the corresponding departure time across Broad Street was recorded to the nearest second, using stopwatches. Passenger service operations for each route were also observed, and the number of boarding passengers and the time required for such passengers to board were recorded. These data are given in Table G-221.

As indicated, the total time of 3 min and 27 sec (3:27) for each bus to traverse these defined limits and perform passenger service operations was similar on an over-all basis for each lane of operation. Similarly, passenger service operations averaged 17.3 passengers boarding each bus, requiring 1 min and 5 sec (1:05), with little variation between lanes of operation. Specific routes, however, did experience differences in these values, particularly in the second-lane operations. The average net travel and delay time of 2 min and 22 sec (2:22) was identical for each lane. With an assumed average speed when moving of 7.5 mph, about 34 sec were required to travel the 375 ft between checkpoints. Almost 1 min of delay per bus was due to the operating conditions.

A minute-by-minute listing of the data surveyed at Beaver Street is given in Table G-222. It indicated that no buses arrived during 25 percent of the minutes observed, two or more buses arrived during 46.7 percent of the minutes observed, and four or more buses arrived during 12.5 percent of the minutes observed. These varying bus headways undoubtedly created many problems and can be attributed to downstream traffic congestion. Control of arrival times at Beaver Street was beyond the scope of this experiment.

With the average boarding time of approximately 1 min used to define an occurrence of conflict between buses having the same route designation, the varying headways described previously resulted in 41 conflicts. These involved 70 of the 185 buses observed, as indicated in Table G-221. The Route 31-South Orange bus line experienced conflicts involving 60 percent of its arriving buses by this definition. Field observation confirmed this was true.

Although a total of nine separately numbered bus routes use Market Street in the experimental area, the Route 2-Ampere bus line was not considered in either the design or surveillance phases of the experiment. This route enters Market Street at Halsey Street and exits at Washington Street without performing passenger service operations on Market Street. The Route 21-Orange express bus service was given separate consideration in the surveillance phase of this experiment, because these buses did not perform passenger service operations on Market Street between Halsey Street and Washington Street. Twenty-four of the Route 31-South Orange Avenue buses are independently owned and operated; however, they operated on a coordinated schedule. Four of the Route 25-Springfield buses are also independently owned but do not operate on a coordinated schedule with other Route 25 buses. No special surveillance was performed on these buses due to difficulty in properly identifying them. All other buses operating in the experimental area were owned and operated by the Public Service Coordinated Transport Company. With the exceptions previously noted, all buses of each numbered route performed passenger service operations in the lanes designated and at the locations shown in Figure G-223.

The improvement for this experiment was designed using as basic concepts:

1. The elimination of the "dual-lane" passenger service operations in favor of curbside passenger service operations only.
2. The establishment of defined bus stop locations for specific routes or groups of routes.

The objective was to create an "on-street" bus terminal facility and to increase headways between buses using the identical stop locations. Restrictions to this design included:

1. Maintaining common bus stop locations for bus routes having a "community of interest" (i.e., buses having a common route over a considerable distance and/or serving the same general bus passenger generator).
2. Providing the same number of stop locations in the same general area as existed in the "before" condition.

Exceptions to this latter restriction were the Route 5-Kinney, Route 54-Devine Street, and Route 72-Madison-Newark Express, which were provided with two stop locations in the "after" condition.

The groupings of routes for specific stop locations were determined by the bus companies. The "community of interest" restriction did not result in an ideal distribution of scheduled bus volumes among the stop locations within the experimental area.

Bus stop dimensions and spacings were determined from:

1. Bus operating characteristics as described by the report of Technical Committee 6-C of the Institute of Traffic Engineers (*9*).
2. Tests at the site, using buses identical to those in regular use.
3. The limiting physical characteristics of the curb length available.

The design plan is shown in Figure G-224.

The assignment of buses to each stop location for the period from 4:00 to 6:00 PM, based on the survey data given in Table G-221, was as follows:

1. Routes 1 and 25—62 buses (33.6 percent of total).

TABLE G-220

4:00 TO 6:00 PM WESTBOUND BUSES ON MARKET STREET [a]

TIME PERIOD, PM	NO. OF BUSES																	
	BROAD ST.						HALSEY ST.						WASHINGTON ST.					
	STOP			THROUGH			STOP			THROUGH			STOP			THROUGH		
	CURB LANE	2ND LANE	ALL	CURB LANE	2ND LANE	ALL	CURB LANE	2ND LANE	ALL	CURB LANE	2ND LANE	ALL	CURB LANE	2ND LANE	ALL	CURB LANE	2ND LANE	ALL
4:00–4:15	38	21	59	17	9	26	18	16	34	20	13	33	12	7	19	17	11	28
4:15–4:30	28	14	42	11	11	22	12	9	21	12	9	21	11	5	16	14	7	21
4:30–4:45	34	17	51	16	4	20	17	4	21	17	4	21	5	3	8	16	9	25
4:45–5:00	30	32	62	13	16	29	22	13	35	16	13	29	7	6	13	16	8	24
5:00–5:15	58	25	83	13	15	28	13	11	24	11	11	22	5	12	17	12	13	25
5:15–5:30	33	14	47	19	8	27	20	14	34	20	12	32	11	7	18	20	15	35
4:30–5:30	155	88	243	61	43	104	72	42	114	64	40	104	28	28	56	64	45	109
5:30–5:45	28	17	45	11	12	23	11	11	22	9	11	20	7	7	14	11	9	20
5:45–6:00 [b]	28	0	28	16	2	18	27	0	27	19	6	25	14	7	21	18	9	27
4:00–6:00	277	140	417	116	77	193	140	78	218	124	79	203	72	54	126	124	81	205

[a] Tuesday, December 19, 1967.
[b] Reverted to single-lane passenger service in this time period.

TABLE G-221

4:00 TO 6:00 PM WESTBOUND BUS OPERATIONS ON MARKET STREET,[a] BEAVER STREET TO BROAD STREET

LANE	BUS ROUTE	TRIP TIME		PASSENGER SERVICE			NET TRAVEL AND DELAY TIME (MIN:SEC)	CONFLICT FREQUENCY						
		BUSES OBS.	AVERAGE PER BUS (MIN:SEC)	BUSES OBS.	AVERAGE PASS. BOARDING	AVERAGE TIME (MIN:SEC)		BUSES OBS.	SIMULTANEOUS ARRIVAL 3 BUSES	SIMULTANEOUS ARRIVAL 2 BUSES	OTHER HEADWAYS ≤1:00	ALL OCCUR.	ALL BUSES (CONFLICTS)	% OF TOTAL
Curb	1	20	3:28	15	16.7	0:54	2:34	24	0	2	2	4	7	29
	25	30	3:43	27	19.4	1:19	2:24	38	0	5	3	8	15	39
	31	27	3:08	28	14.9	0:51	2:17	45	1	8	9	18	27	60
	72	3	2:21	1	13.0	0:41	1:40	4	0	0	0	0	0	0
	All	80	3:24	71	17.0	1:02	2:22	111	1	15	14	30	49	44.1
Second	5	9	3:20	9	31.0	1:43	1:37	13	0	0	0	0	0	0
	21	21	3:58	13	15.6	1:05	2:53	25	0	1	3	4	8	32
	34	7	2:54	13	14.1	0:56	1:58	24	0	3	3	6	11	46
	54	7	2:58	2	9.0	0:28	2:30	12	0	1	0	1	2	17
	All	44	3:31	37	18.5	1:09	2:22	74	0	5	6	11	21	28.4
Both	All	124	3:27	108	17.5	1:05	2:22	185	1	20	20	41	70	37.8

[a] Monday, December 9, 1968.

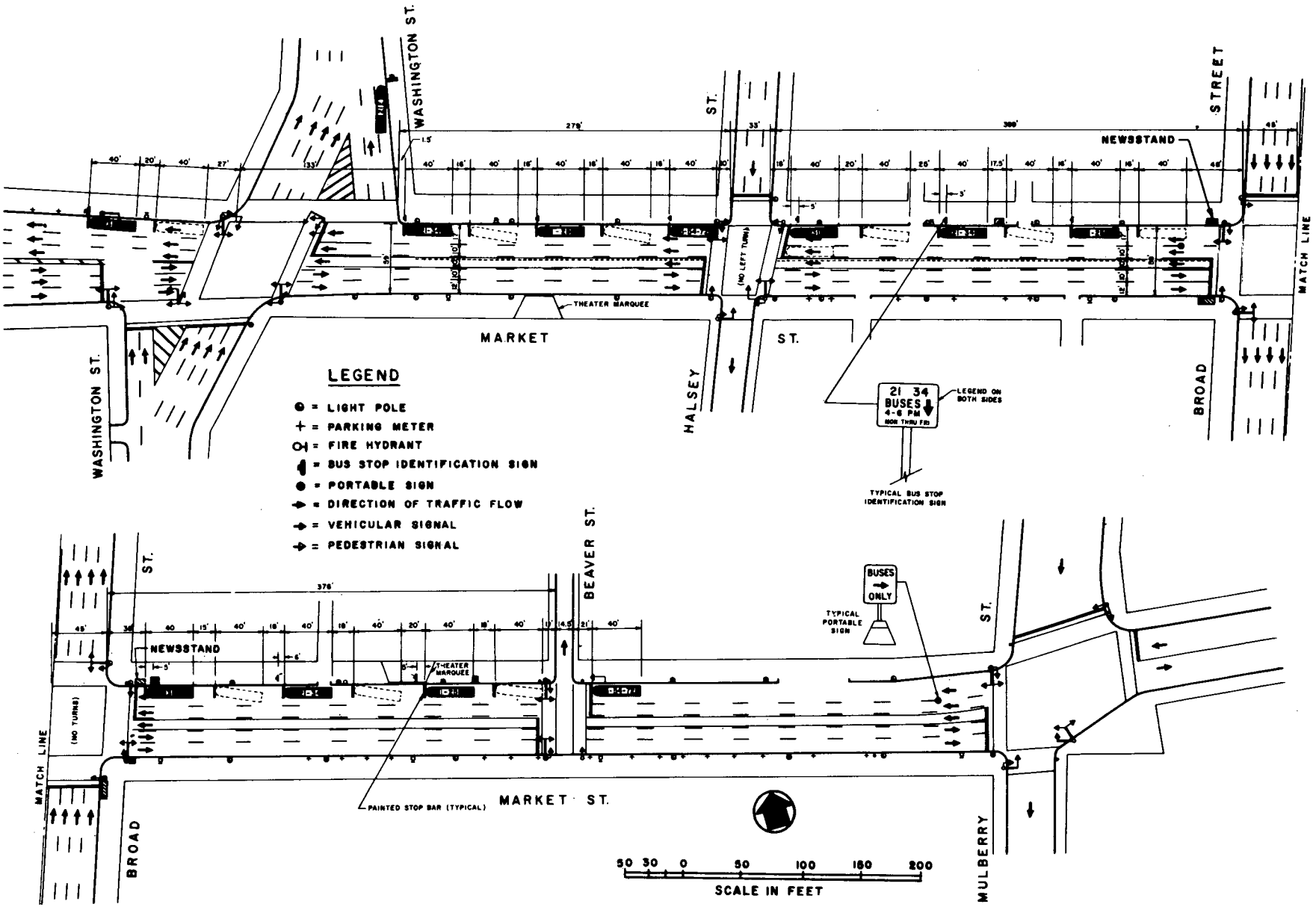


Figure G-224. Vicinity map, "after."

TABLE G-222

4:00 TO 6:00 P.M. WESTBOUND BUS ARRIVAL FREQUENCY ON MARKET STREET AT BEAVER STREET

BUS ARRIVALS PER MINUTE	MINUTES OBS.	ALL BUSES
0	30	0
1	34	34
2	30	60
3	11	33
4	11	44
5	2	10
6	1	6
7	1	7
All	120	194

2. Routes 21 and 34—49 buses (26.4 percent of total).
3. Routes 5, 54, and 72—29 buses (15.7 percent of total).
4. Route 31—45 buses (24.3 percent of total).

As shown in Figure G-224, the stop locations for Routes 21 and 34, Routes 1 and 25, and Route 31 provided sufficient space for two buses to simultaneously provide passenger service. Only the stop location for Routes 5, 54, and 72 was limited to single-bus occupancy. A distance of 16 to 25 ft between individual bus spaces, together with the 17-ft curb lane width, permitted a following bus to leave its stop location at the completion of passenger service operations, even though the preceding bus stop location was still occupied. This spacing also permitted an arriving bus to enter an empty space ahead of a bus remaining at the curb. Stop bars were painted in the curb lane to assist bus operators in properly positioning their buses at the curb. Route identification signs to assist bus operators and passengers alike were also erected at the first stop position in each stop location (Fig. G-225).

The east crosswalk at the Washington Street intersection (Fig. G-223) was relocated in conjunction with Experiment A33 (Fig. G-224). This change was made to permit maximum use of the curb space between Halsey and Washington Streets, to reduce bus-pedestrian conflicts, and to permit the Route 21 Express buses to turn right in front of buses stopped in the first position at this corner. The limited curb length in this block also required that the third stop location for Route 31 buses be positioned on the far side of this intersection, as shown in Figure G-224.

Originally the curb and second lanes were specified for exclusive bus use, primarily at the insistence of the bus companies, to ensure sufficient opportunity for buses to leave their designated curbside stop locations without hindrance from passenger cars and trucks. After the initial day of operation, this regulation was found to be both unenforceable and unnecessary, and only the curb lane was designated for exclusive bus use at the times of the "after" surveys.

Bus company supervisory personnel assisted in the implementation of these revisions and metered the dispatch of buses on Routes 5, 54, and 72 from the first stop location at the near side of Beaver Street. This metering was vital to prevent a second bus from arriving at the second stop location on the far side of Halsey Street when that stop was occupied. Such an occurrence would have blocked the Halsey Street approach to Market Street. Supervision was necessary in the initial stages of implementation to ensure that bus operators provided passenger service only after reaching the designated stop locations. Supervision was also required to prevent bus operators from "stalling" after completion of passenger service operations. This type of operation had occurred during the "before" condition and

Figure G-225. "After" bus stop identification sign.

NEWARK SUNDAY NEWS May 18, 1969

City Changes Four Corners Bus System

A new bus stop system for the Broad and Market Streets area goes into effect at 4 p.m. tomorrow, according to Police Director Dominick A. Spina.

The program, an experimental one, is aimed at eliminating the present two-lane loading operation in Market Street. The new loading points for each bus line will be clearly marked by signs at the curb, Spina said.

The program, called "Oi the Street Bus Terminals," will operate weekdays between 4 and 6 p.m. for one month to determine if it will improve safety and reduce traffic delays. If it does, Spina said, he will seek to make it permanent.

SUNDAY STAR LEDGER May 18, 1969

Newark trying out a bus loading plan

The Newark Police Department will initiate a mouth-long experimental bus loading program in downtown Newark tomorrow in an effort to improve safety and reduce traffic delays during peak periods.

The program provides three specific bus loading positions on north curbs in blocks along Market Street in the vicinity of Broad Street to replace the present two-lane loading operations.

Each of the pickup points will be clearly marked by signs. The program will operate on weekdays between 4 to 6 p.m.

Police Director Dominick A. Spina said the experimental program is being instituted after a careful study of downtown Newark traffic patterns, adding the program will become permanent if it proves successful during the tryout period.

Spina called the program another step toward improving downtown traffic conditions. "We are constantly striving" he said "to correct peak traffic problems."

TEMPORARY CHANGE IN BUS STOPS MARKET STREET, NEWARK

IN COOPERATION WITH THE TRAFFIC DEPARTMENT OF THE CITY OF NEWARK, FOLLOWING A STUDY BY EDWARDS & KELCEY, CONSULTING ENGINEERS, THE PRESENT SYSTEM OF DUAL LOADING LANES, EFFECTIVE ON WEEKDAYS BETWEEN 4.00 AND 6.00 P.M., WILL BE DISCONTINUED BEGINNING MONDAY, MAY 19, FOR A TRIAL PERIOD.

NEWLY ASSIGNED CURB BUS STOPS, IDENTIFIED BY DESCRIPTIVE SIGNS ALONG MARKET STREET FROM BEAVER STREET TO UNIVERSITY AVENUE, WILL BE USED DURING THE TRIAL PERIOD BY ALL OUTBOUND BUS LINES OPERATING ON MARKET STREET DURING THE 4.00 TO 6.00 P.M. RUSH HOUR PERIOD.

PUBLIC SERVICE COORDINATED TRANSPORT

Figure G-226. Public notice regarding experiment.

could be attributed to the competition between different companies for bus passengers.

To ensure effective, safe, and positive reversal of the flow direction in the center lane during the 4:00 to 6:00 PM period in the "after" condition, rubber traffic cones were placed manually along the line dividing opposing directions of traffic flow at 4:00 PM. Portable signs were also placed along the lane line between the curb and second lanes, designating the curb lane for exclusive bus use. These signs supplemented existing post-mounted signs along the north curb. All portable devices were removed at 6:00 PM.

The experiment was implemented on May 19, 1969, with the cooperation of the various bus companies and responsible city officials. Prior to actual implementation, notices were posted in the buses on the affected bus routes, informing passengers of the impending changes. Additional notice was provided by newspaper articles. These notices are shown in Figure G-226. The signs on the street (Fig. G-225) also served to draw attention to the revised operations. Only minor problems were encountered during the initial implementation relative to bus passengers searching for the appropriate bus stop locations. Public acceptance

of the revised operations was quickly gained, and favorable comments on the change were overheard. The confused milling about by waiting bus passengers that frequently occurred during the "before" conditions on a bus arrival was almost totally eliminated.

The surveillance system used to detect changes in operations included time-lapse movie photography and cordon observations of bus operations. Additional observations of westbound traffic at Washington Street were made as a part of the surveillance system for Experiment A33.

The time-lapse movies were intended to verify the bus company's survey data and to provide supplemental data relative to the operation of other traffic. A high-rise building to the west of the experimental area was selected for the position of the movie camera. Using a telescopic lens and an intervalometer, pictures were taken at the rate of one frame per second for the period from 4:30 to 5:30 PM on one day of the "before" and one day of the "after" conditions. Projection of the developed film at a controlled rate permitted direct time measurements of vehicles within the field of view with an accuracy of ±3 percent. The observed field of view of these movies is shown graphically in Figure G-227.

Analysis of these films revealed that individual bus route numbers could not be ascertained. However, the degree of accuracy in timing vehicular movements made their use highly valuable.

The cordon observations of bus operations were made by bus company supervisory personnel at Beaver Street and at Washington Street, using ordinary wristwatches for timing. Bus passenger loads in each individual bus were visually estimated by these experienced persons. The limitations of this type of time measurement are evident when comparisons are made with surveys where stopwatches or time-lapse movies are used for timing. The wristwatches used were synchronized prior to each survey. Bus trip times were recorded to the nearest whole minute for each bus as it crossed each cordon checkpoint. Buses were identified by means of prominently displayed serial numbers. Several bus trip times through the experimental area of 1 min were recorded using this method. The fastest time recorded by the time-lapse movies for any vehicle was one observation of a car with a trip time of 49 sec. This particular car was the only vehicle out the 793 observed and measured that did not stop at least once.

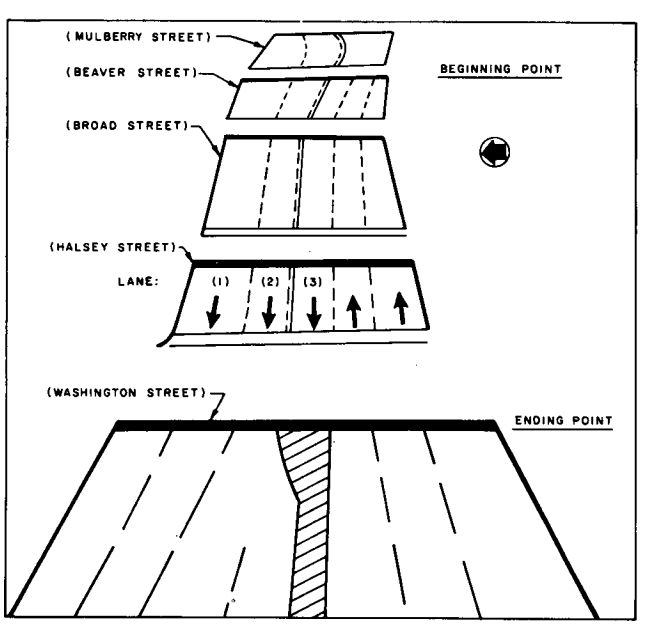


Figure G-227. Time-lapse movie field of view.

One hour of time-lapse movies was made for both the "before" and "after" conditions. Three 4:00 to 6:00 PM "before" and four 4:00 to 6:00 PM "after" surveys were conducted by the bus company, using four regular and nine additional supervisors. The dates of all surveys are given in Table G-223.

Analysis

Average bus trip times for all buses during the 4:30 to 5:30 PM time period from the time-lapse movies are given in Table G-224. This summary indicated small reductions in average bus trip time for each 15-min time interval except the 5:15 to 5:30 time period. For this time period, an average reduction in bus trip time of 1 min and 53 sec (31.9 percent) was observed. The hourly totals indicated an average reduction of 36 sec (11.8 percent).

Average bus trip times for all buses for the same 4:30 to 5:30 PM time period from the bus company surveys are given in Table G-225. This summary indicated greater reductions in average bus trip time for each 15-min period than were indicated in the time-lapse movie summary. The hourly totals indicated an average reduction of 55 sec (16.7 percent).

These differences probably result from the fact that the time-lapse movies represent only a single day of measurement and are more accurate than the individual bus times recorded by the bus company. In any event, the trend of trip time reduction is evident in both survey summaries.

The fact that individual passenger service operations affect over-all bus trip time is true without question. However, the effects of passenger service operations in the experimental area on bus trip times were apparently less than the effects of operating conditions. This is evident in the summary of survey data for the only comparable day of "before" and "after" conditions where bus passenger loads were recorded (Table G-226). This summary indicated an average increase in bus passenger loads of 4.4 persons (8.2 percent) in the 4:45 to 5:00 PM time period, with a net average reduction in trip time of 0.39 min (6.7 percent) for each bus. It is interesting to note that the bus company's Tuesday survey indicated the most improvement of all four days of "after" surveys.

Daily variations in bus operations do occur (Table G-227). This summary indicated an average reduction in bus trip time of 1.04 min (19.7 percent) between the "before" and "after" conditions for all surveys combined. Daily reductions varied between 0.07 min (1.6 percent) and 1.62 min (27.6 percent).

Two other comparisons between the time-lapse movie data and the bus company survey data were made. In the "before" condition, specific routes were required to oper-

TABLE G-223

SUMMARY OF SURVEILLANCE (1969)

CONDITION	BUS COMPANY SURVEYS TUES.	WED.	THURS.	FRI.	TIME-LAPSE MOVIES WED.
"Before"	—	4/30	5/1	5/2	5/14
"After" [a]	5/27	6/11	6/5	5/23	6/4

[a] Experiment implemented 5/19/69.

ate in either the curb or second lane. A summary of bus trip times by lanes for the "before" condition from the time-lapse movie is given in Table G-228, and identical data from the bus company's surveys are given in Table G-229. In these summaries the average bus trip times in the curb lane are almost identical for the two different methods of surveillance. Considerable variations are indicated, however, in the individual 15-min time periods. The average bus trip times in the second lane were considerably different for the two methods. These variations again suggest differences in days and/or in the accuracy of time measurements.

In the "after" condition, specific routes were required to operate in specific bus stop locations, permitting these

TABLE G-224

WESTBOUND BUS TRIP TIME,[a] BEAVER STREET TO WASHINGTON STREET, TIME-LAPSE MOVIES

TIME PERIOD PM	CONDITION	BUSES OBS.	AVERAGE TRIP TIME (MIN:SEC)
4:30–4:45	"Before"	17	4:30
	"After"	14	4:11
	Net change		−0:19
	Percent change		−7.0
4:45–5:00	"Before"	24	4:54
	"After"	23	4:49
	Net change		−0:05
	Percent change		−1.7
5:00–5:15	"Before"	24	4:49
	"After"	28	4:45
	Net change		−0:04
	Percent change		−1.4
5:15–5:30	"Before"	26	5:54
	"After"	23	4:01
	Net change		−1:53
	Percent change		−31.9
4:30–5:30	"Before"	91	5:05
	"After"	88	4:29
	Net change		−0:36
	Percent change		−11.8

[a] Does not include Route 21 express buses.

TABLE G-225

WESTBOUND BUS TRIP TIME,[a] BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

TIME PERIOD PM	CONDITION	BUSES OBS.[b]	AVERAGE TRIP TIME (MIN)
4:30–4:45	"Before"	61	5.11
	"After"	78	3.87
	Net change		−1.24
	Percent change		−24.3
4:45–5:00	"Before"	61	5.46
	"After"	84	4.85
	Net change		−0.61
	Percent change		−11.2
5:00–5:15	"Before"	71	5.92
	"After"	92	4.91
	Net change		−1.01
	Percent change		−17.1
5:15–5:30	"Before"	71	5.31
	"After"	99	4.48
	Net change		−0.83
	Percent change		−15.6
4:30–5:30	"Before"	264	5.46
	"After"	353	4.55
	Net change		−0.91
	Percent change		−16.7

[a] Does not include Route 21 express buses.
[b] Combined data from 3 days of "before" and 4 days of "after" surveys.

TABLE G-226

FRIDAY WESTBOUND BUS TRIP TIME,[a] BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

TIME PERIOD PM	CONDITION	BUSES OBS.	AVERAGE TRIP TIME (MIN)	AVERAGE PASS./BUS (LEAVING)
4:30–4:45	"Before"	20	5.65	51.0
	"After"	20	3.40	47.1
	Net change		−2.25	−3.9
	Percent change		−39.8	−7.6
4:45–5:00	"Before"	19	5.79	53.9
	"After"	20	5.40	58.3
	Net change		−0.39	+4.4
	Percent change		−6.7	+8.2
5:00–5:15	"Before"	20	5.70	58.1
	"After"	16	4.81	58.4
	Net change		−0.89	+0.3
	Percent change		−15.6	+0.5
5.15–5:30	"Before"	21	6.14	50.6
	"After"	25	4.92	49.2
	Net change		−1.22	−1.4
	Percent change		−19.9	−2.7
4:30–5:30	"Before"	80	5.83	53.4
	"After"	81	4.64	52.7
	Net change		−1.19	−0.7
	Percent change		−20.4	−1.3

[a] Does not include Route 21 express buses.

TABLE G-227

4:00 TO 6:00 P.M. WESTBOUND BUS DATA,
BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

DAY	DATE (1969)	CONDITION	BUSES OBS.	AVERAGE TRIP TIME (MIN)	AVERAGE PASS./BUS (LEAVING)
Wed.	4/30	"Before"	172	4.32	—
	6/11	"After"	157	4.32	44.9
		Net change		−0.07	
Thurs.	5/1	"Before"	168	5.60	—
	6/5	"After"	165	4.99	46.4
		Net change		−0.61	
Fri.	5/2	"Before"	164	5.86	44.8
	5/23	"After"	155	4.24	47.9
		Net change		−1.62	+3.1
Tues.	5/27	"After"	170	3.38	48.0
All		Mean "before"	168.0	5.27	44.8 [a]
		Mean "after"	161.8	4.23	46.8
		Net change	−6.2	−1.04	+2.0
		Percent change	−3.7	−19.7	+4.5

[a] Friday only.

stop groupings to be identified in the time-lapse movie. A summary is given in Table G-230 of average bus trip times by this grouping for both the "before" and "after" conditions from the bus company surveys and for the "after" condition from the time-lapse movie. This summary indicated a relationship between the bus company's "after" survey data and the time-lapse movie "after" data. The disproportionate assignment of scheduled buses to the various stop locations is indicated in these average bus trip times, reflecting the increased incidence of conflict at these stop locations.

The maximum percentage of scheduled buses for the 4:00 to 6:00 PM time period assigned to any one stop location grouping was 33.6 percent for Routes 1 and 25. Table G-230 indicates that this grouping experienced the highest average bus trip time of 5 min and 4 sec (5:04) from the time-lapse movie of the "after" condition. The other stop groupings also experienced average bus trip times in the same order as the percentage of scheduled buses—i.e., 26.4 percent for Routes 21 and 34, with an average "after" trip time of 4 min and 39 sec (4:39); 24.3 percent for Route 31, with an average "after" trip time of 4 min and 16 sec (4:16); and 15.7 percent for Routes 5, 54, and 72, with an average "after" trip time of 3 min and 52 sec (3:52). The bus company survey data were compatible with the time-lapse movie data.

Table G-230 also indicates very little improvement in bus trip time for Routes 1 and 25. This was not unexpected, in view of the number of scheduled buses assigned to these stop locations. All other route groupings experienced reductions in trip time, varying between 14.7 and 27.8 percent (0.81 and 1.48 min, respectively).

The bus company surveys permitted additional "before" and "after" comparisons of bus trip times and passenger loads by routes. These data are given for the "before" curb lane routes in Table G-231 and for the "before" second lane routes in Table G-232. This summary indicated that all routes experienced reductions in average bus trip times except for Route 1, which experienced a 5.6-percent increase.

Table G-233 gives the average number of buses and passengers entering and leaving the experimental area by routes for the 4:30 to 5:30 PM time period from the bus company's "after" condition surveys. This summary indicates an average of almost 4,600 passengers leaving, with a net increase of 26.7 passengers per bus. Table G-221 indicates an average of 17.5 boarding passengers per bus at Broad Street.

The time-lapse movies permitted the examination of the effects of this experiment on other westbound vehicles in the 4:30 to 5:30 time period through comparisons of passenger-car trip times, frequency of stops, and westbound traffic volumes. For the speed and stop data, the fourth car in line by lanes for each cycle at Broad Street was timed.

Table G-234 summarizes the speed and stop data for passenger cars in the second and third westbound lanes. For the second lane, the percentage of cars being stopped twice was reduced by 26.5 percent, and reductions of 10.1 and 14.4 percent in trip time were observed for cars making one or two stops, respectively. Trip time was reduced by 25.4 percent. For the third lane, the percentage of cars being stopped twice was reduced by 16.2 percent, with a reduction of 7.4 percent in trip time for cars making one stop and an increase of 4.0 percent in trip time for cars making two stops. Trip time was reduced by 14.2 percent. For all passenger cars in both lanes, this summary indicated a 21.9-percent reduction in the number of cars being stopped twice and a reduction of 25.5 sec (20.4 percent) in trip time, resulting in an increase in average speed of 1.97 to 9.66 mph. Expressed in different terms, this re-

duction in trip time was a saving of 1.6 min per mile of travel.

From the time-lapse movies, the total volume proceeding westbound through the Halsey Street intersection by lanes in the 4:30 to 5:30 PM time period is summarized in Table G-235. In the third lane, a reduction of 2.8 vehicles to 10.1 per cycle (21.7 percent) was observed, with a corresponding increase of 3.3 vehicles to 8.8 per cycle (60.0 percent) observed in the second lane. The greater freedom of lane choice permitted passenger cars is reflected in the increase of 3.6 cars per cycle (85.7 percent) in the second lane.

A slight increase of 0.8 to 20.6 vehicles per cycle (4.0 percent) was observed in the approach totals, with passenger cars increasing 1.0 to 17.6 per cycle (6.0 percent). These changes apparently indicated a trend of improved operations. The surveillance of westbound traffic at Washington Street for Experiment A33 indicated an insignificant reduction in vehicles through on the green indication and a statistically significant decrease in vehicles stopped by the red signal indication of 3.9 per cycle (28.1 percent), which may reflect the decreased congestion achieved by this experiment.

A statistical analysis was performed on the over-all trip times for all buses combined for both the time-lapse movies and the bus company surveys. A summary of this analysis for the time-lapse movies is given in Table G-236. The trip times analyzed included those for the Route 21 Express line in the "before" condition.

A similar summary of the statistical analysis of the bus company's surveys is given in Table G-237. No statistical changes were found for the variances of these latter data. The trip times analyzed included those for the Route 21 Express line. An additional analysis of the Route 21

TABLE G-228

"BEFORE" WESTBOUND BUS TRIP TIME BY LANE,[a]
BEAVER STREET TO WASHINGTON STREET, TIME-LAPSE MOVIE

TIME PERIOD PM	CURB LANE, BUSES: 1, 25, 31, 72		SECOND LANE,[b] BUSES: 5, 21, 34, 54		SECOND LANE, BUS: 21 EXPRESS[c]	
	BUSES OBS.	AVERAGE TRIP TIME (MIN:SEC)	BUSES OBS.	AVERAGE TRIP TIME (MIN:SEC)	BUSES OBS.	AVERAGE TRIP TIME (MIN:SEC)
4:30–4:45	9	4:38	8	4:22	0	—
4:45–5:00	19	4:52	5	5:05	2	2:59
5:00–5:15	13	5:26	11	4:04	2	3:33
5:15–5:30	19	6:19	7	4:48	2	6:07
4:30–5:30	60	5:25	31	4:28	6	3:12
Range		2:25–8:21		2:46–7:07		2:51–6:17

[a] Wednesday, May 14, 1969. Began at Beaver St. to through Washington St.
[b] Excluding 21 express buses.
[c] Bus turns right at Washington St. without performing passenger service operations on Market St. at Washington St.

TABLE G-229

"BEFORE" WESTBOUND BUS TRIP TIME BY LANE,[a]
BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

TIME PERIOD PM	CURB LANE, BUSES: 1, 25, 31, 72		SECOND LANE,[b] BUSES: 5, 21, 34, 54		SECOND LANE, BUS: 21 EXPRESS[c]	
	BUSES OBS.	AVERAGE TRIP TIME (MIN)	BUSES OBS.	AVERAGE TRIP TIME (MIN)	BUSES OBS.	AVERAGE TRIP TIME (MIN)
4:30–4:45	39	5.10	22	5.14	0	—
4:45–5:00	40	5.58	21	5.24	4	3.50
5.00–5:15	48	6.02	23	5.70	3	6.67
5:15–5:30	45	5.20	26	5.50	5	6.40
4:30–5:30	172	5.49	92	5.40	12	5.50

[a] Three days' averages.
[b] Excluding 21 express buses.
[c] Bus turns right at Washington St. without performing passenger service operations on Market St. at Washington St.

TABLE G-230

4:30 TO 5:30 PM WESTBOUND BUS TRIP TIME BY "AFTER" STOP LOCATION GROUPINGS, BEAVER STREET TO WASHINGTON STREET

CONDITION	A			B			C			D			—		
	BUSES: 1, 25			BUSES: 21, 34			BUSES: 5, 54, 72			BUS: 31			BUS: 21 EXPRESS		
	BUSES OBS.	AVG. TRIP TIME (MIN)	AVG. PASS./BUS (LV.)	BUSES OBS.	AVG. TRIP TIME (MIN)	AVG. PASS./BUS (LV.)	BUSES OBS.	AVG. TRIP TIME (MIN)	AVG. PASS./BUS (LV.)	BUSES OBS.	AVG. TRIP TIME (MIN)	AVG. PASS./BUS (LV.)	BUSES OBS.	AVG. TRIP TIME (MIN)	AVG. PASS./BUS (LV.)
(a) Bus company surveys															
"Before"	84	5.33	61.4	60	5.47	47.3[a]	41	5.32	50.5[a]	79	5.67	48.1	12	5.50	45.0[a]
"After"	109	5.20	55.3	79	4.59	46.4	58	3.84	52.8	107	4.22	48.8	13	4.69	42.2
Net change		−0.13	−6.1		−0.88	−0.9		−1.48	+2.3		−1.45	+0.7		−0.81	−2.8
Percent change		−2.4	−9.9		−16.1	—		−27.8	—		−25.6	+1.5		−14.7	—
(b) Time-lapse movie															
"After" (min:sec)	22	5.07 (5:04)	—	21	4.65 (4:39)	—	15	3.87 (3:52)	—	30	4.27 (4:16)	—	—	—	—

[a] Friday only.

TABLE G-231

4:30 TO 5:30 PM WESTBOUND BUS TRIP TIME FOR "BEFORE" CURB LANE ROUTES, BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

ROUTE NO.	CONDITION	BUSES OBS.	AVERAGE TRIP TIME (MIN)	AVERAGE PASS./BUS (LEAVING)
1	"Before"	28	5.14	59.3
	"After"	37	5.43	55.8
	Net change		+0.29	−3.5
	Percent change		+5.6	−5.9
25	"Before"	56	5.43	62.5
	"After"	72	5.08	55.2
	Net change		−0.35	−7.3
	Percent change		−6.4	−11.7
31	"Before"	79	5.67	48.1
	"After"	107	4.22	48.8
	Net change		−1.45	+0.7
	Percent change		−25.6	+1.5
72	"Before"	9	5.44	36.9
	"After"	12	4.08	36.3
	Net change		−1.36	−0.6
	Percent change		−25.0	−1.6

TABLE G-232

4:30 TO 5:30 PM WESTBOUND BUS TRIP TIME FOR "BEFORE" SECOND-LANE ROUTES, BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

ROUTE NO.	CONDITION	BUSES OBS.	AVERAGE TRIP TIME (MIN)	AVERAGE PASS./BUS (LEAVING)
5	"Before"	19	5.32	63.3[a]
	"After"	27	3.78	64.6
	Net change		−1.54	—
	Percent change		−28.9	—
21	"Before"	24	5.67	40.7[a]
	"After"	30	5.33	50.7
	Net change		−0.34	—
	Percent change		−6.0	—
34	"Before"	36	5.33	53.2[a]
	"After"	49	4.14	43.7
	Net change		−1.19	—
	Percent change		−22.3	—
54	"Before"	13	5.23	50.0[a]
	"After"	19	3.79	46.3
	Net change		−1.44	—
	Percent change		−27.5	—
21E	"Before"	12	5.50	45.0[a]
	"After"	13	4.69	42.2
	Net change		−0.81	—
	Percent change		−14.7	—

[a] Friday only.

TABLE G-233

4:30 TO 5:30 PM WESTBOUND BUS PASSENGERS, BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

ROUTE NO.	BUSES OBS.[a]	% OF TOTAL	PASSENGERS PER BUS: ARRIVING AT BEAVER[b]	LEAVING AT WASHINGTON[a]	NET DIFFERENCE	ALL PASS.[a]	% OF TOTAL
1	9.3	10.2	38.9	55.8	+16.9	516.5	11.2
5	6.8	7.4	40.1	64.6	+24.5	436.3	9.5
21	7.5	8.2	23.6	50.7	+27.1	355.5	7.7
21E	3.3	3.6	14.1	42.2	+28.1	137.3	3.0
25	18.0	19.7	23.1	55.2	+32.1	993.8	21.6
31	26.8	29.3	17.1	48.8	+31.7	1,305.5	28.4
34	12.0	13.1	26.1	43.8	+17.7	525.3	11.4
54	4.8	5.2	30.5	46.3	+15.8	220.0	4.8
72E	3.0	3.3	15.8	36.3	+20.5	109.0	2.4
All	91.5	100.0	23.7[c]	50.4[c]	+26.7[c]	4,599.2	100.0

[a] Means of 4 days "after" bus company survey measurements.
[b] Means of 3 days "after" bus company survey measurements only.
[c] Fourth day, 5/23/69, not measured. Average values.

Express buses alone indicated that the numerical reduction in trip time was not statistically significant.

Conclusions

The revision from "dual-lane" passenger service operations to a curbside "on-street" terminal-type transit operation by westbound buses on Market Street in Newark between Beaver Street and Washington Street in the 4:00 to 6:00 PM time period resulted in:

1. Average reductions in westbound bus trip time for the 4:30 to 5:30 PM time period of from 11.8 to 16.7 percent (36 to 55 sec).
2. Average reductions in westbound bus trip time by

TABLE G-234

4:30 TO 5:30 PM WESTBOUND CAR TRIP TIME AND STOP FREQUENCY, BEAVER STREET TO WASHINGTON STREET, TIME-LAPSE MOVIES

CONDITION	ONE STOP: NO. OF OBS.	% OF TOTAL	AVG. TRIP TIME (SEC)	TWO STOPS: NO. OF OBS.	% OF TOTAL	AVG. TRIP TIME (SEC)	ALL RUNS: NO. OF OBS.	AVG. TRIP TIME (SEC)	AVG. NO. OF STOPS
(*a*) Lane 2									
"Before"	23	57.5	100.7	17	42.5	181.7	40	135.2	1.4
"After"	21	84.0	90.5	4	16.0	155.5	25	100.9	1.2
Net change		+26.5	−10.2		−26.5	−26.2		−34.3	−0.2
Percent change		—	−10.1		—	−14.4		−25.4	−14.3
(*b*) Lane 3									
"Before"	29	74.4	98.4	10	25.6	161.6	39	114.6	1.3
"After"	29	90.6	91.1	3	9.4	168.0	32	98.3	1.1
Net change		+16.2	−7.3		−16.2	+6.4		−16.3	−0.2
Percent change		—	−7.4		—	+4.0		−14.2	−15.4
(*c*) Both lanes									
"Before"	52	65.8	99.4	27	34.2	174.3	79	125.0	1.3
"After"	50	87.7	90.9	7	12.3	160.9	57	99.5	1.1
Net change		+21.9	−8.5		−21.9	−13.4		−25.5	−0.2
Percent change		—	−8.6		—	−7.7		−20.4	−15.4

TABLE G-235

WESTBOUND VEHICULAR VOLUME ON MARKET STREET AT HALSEY STREET, TIME-LAPSE MOVIES

TIME PERIOD PM	CURB LANE	SECOND LANE				THIRD LANE			ALL LANES			
	BUSES	BUSES	CARS	TRUCKS	ALL	CARS	TRUCKS	ALL	BUSES	CARS	TRUCKS	ALL
					(*a*) "Before"							
4:30–4:45	8	9	42	5	56	107	7	114	17	149	12	178
4:45–5:00	14	12	43	5	60	136	6	142	26	179	11	216
5:00–5:15	17	9	37	0	46	119	2	121	26	156	2	184
5:15–5:30	18	10	46	3	59	132	5	137	28	178	8	214
4:30–5:30	57	40	168	13	221	494	20	514	97	662	33	792
Per cycle	1.4	1.0	4.2	0.3	5.5	12.4	0.5	12.9	2.4	16.6	0.8	19.8
					(*b*) "After"							
4:30–4:45	10	4	87	5	96	97	1	98	14	184	6	204
4:45–5:00	12	11	65	2	78	100	3	103	23	165	5	193
5:00–5:15	22	6	61	3	70	113	3	116	28	174	6	208
5:15–5:30[a]	18	5	118	1	124	44	2	46	23	162	3	188
4:30–5:30	62	26	331	11	368	354	8	363	88	685	20	793
Per cycle[b]	1.6	0.7	7.8	0.3	8.8	9.8	0.2	10.1	2.4	17.6	0.5	20.6

[a] Truck in intersection blocked third lane for four complete cycles.
[b] Four affected cycles not included.

TABLE G-236

4:30 TO 5:30 PM WESTBOUND BUS TRIP TIME, BEAVER STREET TO WASHINGTON STREET, TIME-LAPSE MOVIES

CONDITION	BUSES OBS.	MEAN TRIP TIME (MIN)	VARIANCE
"Before"	96	5.14	1.94
"After"	91	4.37	1.01
Net difference	—	−0.77	−0.93
Level of sig.	—	0.0005	0.01

TABLE G-237

4:30 TO 5:30 PM WESTBOUND BUS TRIP TIME, BEAVER STREET TO WASHINGTON STREET, BUS COMPANY SURVEYS

CONDITION	BUSES OBS.	MEAN TRIP TIME (MIN)
"Before"	275	5.5
"After"	363	4.5
Net difference	—	1.0
Level of sig.	—	0.0005

15-minute intervals between 4:30 and 5:30 PM of from 1.4 to 31.9 percent (4 sec to 1 min and 53 sec).

3. Average reductions in westbound bus trip time by individual routes for the 4:30 to 5:30 PM time period of as much as 28.9 percent (1 min and 54 sec).

4. Average reductions in westbound passenger-car trip time for the 4:30 to 5:30 PM time period of 20.4 percent (25.5 sec).

5. A decrease of 21.9 percent in the number of cars being stopped twice, with an accompanying increase in total volume of 4.0 percent (0.8 vehicle per cycle).

6. Removal of potentially hazardous "in the street" bus loading areas.

7. Reduced concentrations of waiting bus passengers.

The effectiveness of the revised transit operations is affirmed by the bus company's plans to remove two westbound buses from the schedule for this time period as a sole result of the time saving realized, making the revised operations a permanent procedure, and by the bus company's stated intent to consider similar operations for other areas of maximum transit activity in the CBD.

The significance of the acceptance by all concerned of the "on-street" bus-terminal type of design operation in preference to the "dual-lane" passenger service operation, which was considered to be superior to exclusive lane bus operation, should not be overlooked. For the 100 buses per hour in the direction of dominant traffic flow encountered in the experiment, it can be surmised from this acceptance that the terminal-type operation was superior to the application of an exclusive bus lane.

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