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Contents

Crossroute Access Design in Interchange Areas

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•WITH THE rapid expansion of the Interstate Highway System, a critical need to eliminate or to minimize future congestion at freeway interchanges has developed. A major cause of this problem is that important new highway facilities tend to attract new uses of land that are heavy generators of traffic. Congestion or loss of functional efficiency may then result if: (a) the additional traffic volumes generated by the new land use have not been anticipated and exceed the capacity of the interchange facilities, or (b) the access demands of the land development are incompatible with the design of the interchange or crossroute.

In many cases the crossroute, rather than the expressway itself, is the scene of interchange area congestion. Although desiring to locate as close as possible to the expressway, land development in interchange areas often creates undesirable access conditions along the crossroutes. Private access points and public streets often serving substantial volumes of traffic are sometimes located in such close proximity to ramp terminals that hazardous and congested conditions result from conflicts between ramp, access point, and through traffic.

At least a partial solution to these problems may be found in an extension of access control for a certain distance along the crossroute. The added length of protected roadway would allow heavy ramp or access point volumes to enter the route in a more orderly fashion without impeding flow through the interchange. Although the need for (and benefits from) incorporating this additional access control near interchanges has been recognized for some time, the questions of "how much" or "how far" have not thus far been satisfactorily answered.

This paper describes an analysis of the design factors which would influence such an extension of access control. The objective of this research was to develop a design aid which would identify the elements controlling the desirable distance between a ramp terminal and the nearest access point along the crossroute. Inasmuch as this distance will vary for different types and combinations of ramp and access point design (as well as for various route speeds and volumes), it was necessary to determine the controlling elements for many possible situations. (Factors such as speed and volume do not themselves represent distance, but rather, they influence design elements, such as weaving sections, which do represent distance.) Because of the many variables involved, the mathematically possible number of ramp terminal-access point combinations runs into the thousands. However, not all of these combinations represent practical designs and even fewer represent the most common situations encountered by highway designers. In this research, approximately 60 of the most common and important design situations were studied in detail. For each of these situations the controlling design elements (i.e., merging, deceleration distances, storage, etc.) were determined and combined into equations. Given normal design criteria, such as route and ramp speeds and the magnitudes of the volumes involved, these equations can be solved to give the proper spacing between ramp terminals and access points along the crossroute. Their application in highway design is shown through the use of illustrative design problems later in this report.

In addition, a coding or reference system was developed to aid in the definition or description of all possible design situations. Using this reference system, it would seem feasible to eliminate all those mathematically possible combinations of ramp terminals and access points which did not represent practical designs. This would

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produce a virtually complete list of all practical design combinations for which equations could then be written.

The material in this paper was developed as part of a comprehensive study of highway-land use relationships in the vicinity of interchanges, and is only one of several areas of investigation included in the study. Special assistance was rendered on this phase of the study by the Bureau of Design of the Illinois Division of Highways and particularly by W. A. Frick of that agency.

Other aspects of the study investigated such areas as the use or application of land-use control techniques to reduce conflicts and congestion around interchanges. The over-all study was conducted under the joint sponsorship of the Illinois Division of Highways and the U. S. Bureau of Public Roads.

GENERAL DEFINITIONS

Divided Highway—a route whose directional traffic lanes are separated by a barrier median or divider.

Off Ramp—a ramp or roadway used by traffic to exit from a controlled-access route (expressway).

On Ramp—a ramp or roadway used by traffic to enter a controlled-access route (expressway).

Expressway—a divided highway with full or partial control of access and generally with grade separations at intersecting routes.

Interchange—a system of interconnecting roadways (ramps) with one or more grade separations providing for the interchange of traffic between two or more routes or highways.

Crossroute—the route interchanging with a controlled-access facility or expressway. (In some cases, the crossroute may also be a controlled-access facility—at least in the vicinity of the interchange.)

Access Point—a point of ingress and/or egress along the crossroute. It may provide local access to adjacent land or it may indicate the intersection of another street or highway with the crossroute.

Entrance Terminal—a ramp terminal where traffic leaves a ramp and enters the through traffic lanes of the crossroute.

Exit Terminal—a ramp terminal where traffic leaves the through lanes of the crossroute and enters a ramp.

Access Point Exit—point where traffic leaves the through lanes of the crossroute and enters a local driveway, street, or other roadway intersecting the crossroute.

Access Point Entrance—point where traffic enters the through lanes of the crossroute from a local driveway, street, or other roadway intersecting the crossroute.

IDENTIFICATION AND DESCRIPTION OF DESIGN SITUATIONS

As noted previously, there can be many possible combinations of ramp terminal and access point design—with each of these, in turn, influenced by certain characteristics of the crossroute. This multiplicity of factors produced one of the early problems encountered in this research, in that it made the definition or description of a design situation subject to the omission of important factors. In an attempt to systematize the consideration of these factors, a reference system was developed which classified and grouped them into six major categories, containing the possible factors which might be used to describe one part or aspect of a ramp terminal-access point situation:

A. Type of ramp and crossroute,
B. Type of ramp terminal operation,
C. Possible movements at ramp terminal,
D. Locational relationship of ramp terminal to crossroute access point,
E. Type of access point operation, and
F. Possible movements at access point.

Tables 1 and 2 give the various factors comprising these groups. The use of this system is illustrated in the following steps as it might be applied to the typical design

TABLE 1

GROUP CLASSIFICATION OF FACTORS DESCRIBING RAMP TERMINALS ALONG A CROSSROUTE

Group	Ramp Terminal Description
A: Type of ramp and crossroute	1. Off ramp to divided crossroute 2. On ramp from divided crossroute 3. Off ramp to undivided crossroute 4. On ramp from undivided crossroute
B: Type of ramp terminal operation	1. Free-flow ramp terminal with change in number of crossroute through lanes (i.e., lane(s) added or dropped at ramp terminal) 2. Free-flow ramp terminal without change in number of crossroute through lanes 3. Signal-controlled ramp terminal with change in number of crossroute through lanes 4. Signal-controlled ramp terminal without change in number of crossroute through lanes 5. Stop-or-yield-sign-controlled ramp terminal with change in number of crossroute through lanes 6. Stop-or-yield-sign-controlled ramp terminal without change in number of crossroute through lanes
C: Possible movements at ramp terminals	Ramp Entrance Terminal: 1. Left turns from ramp prohibited 2. Left turns from ramp permitted Ramp Exit Terminal: 3. Left turns into ramp prohibited 4. Left turns from ramp prohibited but left turns into ramp permitted from through lane from direction of approach on crossroute where access point is located 5. Left turns into ramp permitted from separate lane from direction of approach on crossroute where access point is located 6. Left turns into ramp permitted from through lane from direction of approach on crossroute opposite to where access point is located 7. Left turns into ramp permitted from separate lane from direction of approach on crossroute opposite to where access point is located

TABLE 2

GROUP CLASSIFICATION OF FACTORS DESCRIBING ACCESS POINTS ALONG A CROSSROUTE

Group	Access Point Description
D: Locational relationship of ramp terminal to access point	Access Point Exit: 1. From crossroute on same side of divided crossroute as ramp terminal 2. From crossroute on same side of undivided crossroute as ramp terminal 3. From crossroute on opposite side of divided crossroute as ramp terminal 4. From crossroute on opposite side of undivided crossroute as ramp terminal Access Point Entrance: 5. To crossroute on same side of divided crossroute as ramp terminal 6. To crossroute on same side of undivided crossroute as ramp terminal 7. To crossroute on opposite side of divided crossroute as ramp terminal 8. To crossroute on opposite side of undivided crossroute as ramp terminal
E:	1. Free-flow access point with change in number of crossroute through lanes (i.e., lane(s) added or dropped at access point) 2. Free-flow access point without change in number of crossroute through lanes 3. Signal-controlled access point with change in number of crossroute through lanes 4. Signal-controlled access point without change in number of crossroute through lanes 5. Stop-or-yield-sign-controlled access point with change in number of crossroute through lanes 6. Stop-or-yield-sign-controlled access point without change in number of crossroute through lanes
F: Possible movements at access point	Access Point Exit: 1. Left turns into access point prohibited 2. Left turns into access point permitted from through lane from direction of approach on crossroute where ramp terminal is located 3. Left turns into access point permitted from separate lane from direction of approach on crossroute where ramp terminal is located 4. Left turns into access point permitted from through lane from direction of approach on crossroute opposite to where ramp terminal is located 5. Left turns into access point permitted from separate lane from direction of approach on crossroute opposite to where ramp terminal is located Access Point Entrance: 6. Left turns from access point permitted 7. Left turns from access point prohibited

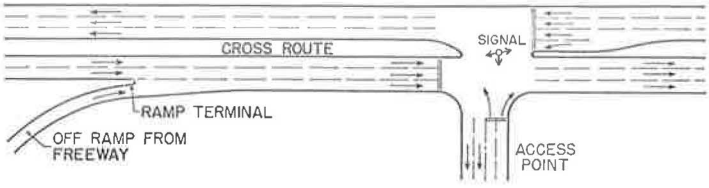


Figure 1. Typical ramp terminal-access point design situation.

situation shown in Figure 1. Entrance and exit movements are listed and analyzed separately; thus, the two-way access point in Figure 1 requires a description of both entering and exiting movements. The ramp terminal, however, serves one-way entering traffic and requires only one description.

Step 1. —Describe the ramp terminal condition. From Table 1, select Group A, Factor 1; Group B, Factor 1; and Group C, Factor 1.

There is no traffic exiting from the crossroute into the ramp at its terminal, thus the description is complete.

Step 2. —Describe the access point condition. The access point has both entering and exiting traffic; thus, each movement must be described. For the access point entrance condition, from Table 2, select: Group D, Factor 5; Group E, Factor 4; and Group F, Factor 6.

For the access point exit condition, from Table 2, select: Group D, Factor 1; Group E, Factor 3; and Group F, Factor 5.

Finally, the descriptions for the situation in Figure 1 are 1-1-1-5-4-6 (considering access point entrance) and 1-1-1-1-3-5 (considering access point exit). Note that the first three digits, which describe the ramp terminal condition, are identical in both descriptions.

Using this system, it may be possible to identify all of the design situations that are practical or significant, and then develop distance equations for each of these cases. Assuming that the six groups of factors described in Tables 1 and 2 represent all the factors needed to define any ramp terminal-access point situation, there are a finite number of combinations or situations which they could produce. Many combinations could quickly be eliminated because they would represent impossible or highly improbable situations. Those remaining could be further analyzed and distance equations developed.

The following sections of this paper discuss the development of equations for some of the most common situations, and illustrate their use with sample problems.

IDENTIFICATION OF DESIGN ELEMENTS

The preceding section discussed primarily the problems involved in defining or describing the many ramp terminal-access point design situations encountered by highway designers. In the course of developing this material approximately 60 of the most common and significant design cases were identified. Essentially, these consist of ramp terminal-access point relationships associated with simple diamond and full cloverleaf interchanges.

The immediate objective at this point in the research was to identify the design elements that would influence the proper spacing between a crossroute ramp terminal and the nearest crossroute access point. If these elements were known for a particular design case or situation, they could be combined into equation form, and the equation could be solved for various traffic volume and route speed conditions.

Adhering to this procedure, 60 design situations were studied individually to determine the design elements which were critical for each case. Basically, this involved a logical process which considered: (a) the traffic movements that were permitted or possible under each situation, (b) the various maneuvers which these movements might logically make, and (c) the manner in which the highway designer (from a standpoint of traffic efficiency and safety) would desire these movements to be made.

As this analysis continued through the various design situations, certain elements kept reappearing, whereas others were eliminated, replaced, or combined. Table 3 describes the major design elements which were determined as being applicable in these cases. Despite the fact that these elements were evolved from an analysis of only 60 design situations, there are two facts which suggest that this list may be nearly complete: (a) the extent to which many of them kept recurring in one design situation after another, and (b) the 60 situations that were studied represent probably the most common and important design cases.

The following section discusses the development of equations using these design elements as equation components.

TABLE 3

DESIGN ELEMENTS INFLUENCING SPACING BETWEEN CROSSROUTE RAMP TERMINALS AND ACCESS POINT

L	=	allowable distance between ramp terminal and nearest crossroute access point
R_R	=	radius of ramp at crossroute connection
R_{AR}	=	radius of right turn to or from access point
R_{AL}	=	radius of left turn to or from access point
S_1	=	required storage distance for right turns at a signalized intersection
S_2	=	required storage distance for left turns at a signalized intersection
S_3	=	required storage distance for through traffic at a signalized intersection
S_{L2}	=	required storage distance for left turns with no signal control (function of gaps in the opposing traffic stream)
w	=	width of access point roadway
N	=	number of lanes in one direction on crossroute
M_L	=	distance required to merge (converge separate streams of traffic into a single stream) traffic to the left (i.e., ramp traffic to through traffic)
W	=	distance required to weave (cross) traffic streams moving in the same general direction
G	=	distance traveled while seeking a suitable gap in adjacent lane traffic
C	=	distance traveled while actually changing lanes
D	=	distance required to decelerate from design speed of crossroute to design speed of turning radius at ramp terminal or access point or to stop condition
T	=	appropriate signing distance including perception and reaction time and required deceleration distance

DEVELOPMENT OF EQUATIONS

Once the controlling design elements had been determined, the establishment of equations became a simple matter of combining the pertinent elements for each particular design situation. The equations were then tested by applying them to design problems such as those illustrated in the following chapter. In every case tested, the distances obtained appeared to be reasonable in relation to the volumes and speeds which were assumed. Appendix B contains a list of references for the various design elements used in this study.

There are certain basic principles which, in effect, set the "ground rules" for the use of the equations and describe the conditions of their application. A listing and discussion of these principles are contained in the following paragraphs:

1. Method of measurement—the allowable distance, L, is measured along the crossroute from the centerline of the access point roadway to the near edge (projected) of the ramp roadway, or, in the case of a free-flow ramp terminal, to the gore.
2. Multiple values—when more than one equation can be written for a particular designation, the equation resulting in the largest total value for L should be used, thereby satisfying the most critical condition.
3. Adjustments for specific conditions—the calculated allowable distance may be adjusted for: unusual vertical or horizontal alignments; certain volume and traffic characteristics peculiar to the area (such as those found on commuter routes); frontage conditions regulating signal and sign placement; physical barriers such as bridge abutments, retaining walls, and piers.
4. Equation conditions relating to signalized intersections—whenever storage distance at a signalized intersection (either ramp terminal or access point) is an equation component, the equation for L should be written for the last car to be stored at the intersection on the end of the red signal phase. For example, the deceleration distance used in this particular equation should be measured to the end of the stored vehicles and not to the intersection stopline.
5. Equation components relating to speed—in determining the numerical value for any component which is partially or solely a function of speed, the design speed of the facility should be used.

6. Vehicle maneuvers—an important consideration in selecting elements comprising equations is that the driver of the vehicle be required to make only one decision or accomplish only one element at a time. For example, when seeking a gap in a stream of adjacent traffic (preceding a lane change maneuver), the driver should not at the same time be required to decelerate in order to enter a ramp terminal or access point. Thus, the equations developed reflect maximum consideration of traffic safety as well as high standards of design.

7. Intersection capacity analysis—should be made to determine storage and laning requirements at major intersections.

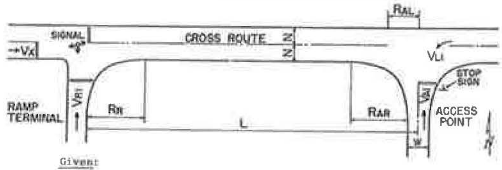
Possibly the most important principle of all, underlying the application of these equations is that they be used as guides only, and as such, must be tempered with sound engineering judgment.

APPLICATION OF EQUATIONS TO DESIGN PROBLEMS

Equations developed during the course of this research are applied to three different design situations. Route volumes, speed data, and locational and physical characteristics which would normally be available to the design engineer are supplied as "givens" in these examples. The design situations have also been identified by the coding or classification system discussed in this report.

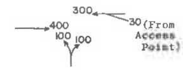
Within a single design situation, it may be possible to write two or more equations, any of which might yield the controlling distance under certain volume and speed conditions. These equations generally correspond to the various traffic movements possible in a particular design case.

EXAMPLE 1 - CODE NO.'S 343264 Access Point Exit
343666 Access Point Entrance



Given:

Rural Area
Design speed of CROSS ROUTE = 70 MPH
10% commercial vehicles
N = 1 lane
W = 20'
R_R = 150'-50'-150' (5' offset)
R_{AR} = 150'-50'-150' (5' offset)
R_{AL} = 50'
V_X = 400 DHV
V_{RI} = 200 DHV
V_{AI} = 50 DHV
V_{LI} = 20 DHV



Traffic volumes (DHV) at Ramp Terminal

To Find: L

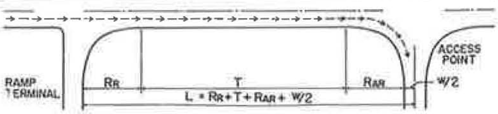
STEP 1 - Examine possible movements between RAMP TERMINAL and ACCESS POINT and select the most critical.

Movement I: From eastbound CROSS ROUTE (V_X) to a right turn at the ACCESS POINT. (Code 343264)

Movement II: Left turn from the ACCESS POINT (V_{AI}) to a stop at the RAMP TERMINAL - CROSS ROUTE signalized intersection. (Code 343666)

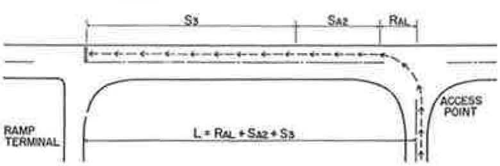
STEP 2 - Write equations for critical movements.

Movement I:



S_{A2} = Storage distance for left-turn from ACCESS POINT
R_{AL} = Left-turn Radius from ACCESS POINT
R_R = Ramp Radius
T = Appropriate signing distance (including perception and reaction time, and deceleration distance)
R_{AR} = Right turn Radius from ACCESS POINT
W = Access Point roadway width
S_3 = Storage distance for through traffic at RAMP TERMINAL - CROSS ROUTE signalized intersection

Movement II:



NOTE: An examination of the design volumes to be stored in determining S_{A2} or S_3 indicates that these combined distances will be much less than the appropriate signing distance (T) in Movement I.

Therefore, it appears that the equation for L in Movement I will result in the appropriate distance from RAMP TERMINAL to ACCESS POINT.

STEP 3 - Determine Value - Guides for equation unknowns.

Movement I:

1) T = Perception and reaction time + Deceleration distance.
T = 2.5 seconds at design speed of CROSS ROUTE (70 MPH) + Deceleration from CROSS ROUTE design speed (70 MPH) to Right-turn radius design speed (15 MPH).
T = (2.5 seconds x 103 ft./sec.) + 450'
T = 710'

2) $R_R = R_{AR}$ = Tangent distance for 150'-50'-150' three-centered curve with $\Delta = 90°$ and a 5' offset.
$R_R = R_{AR} = 85'$

STEP 4 - Compute L for critical movement.

Movement I: $L = R_R + T + R_{AR} + \frac{W}{2}$

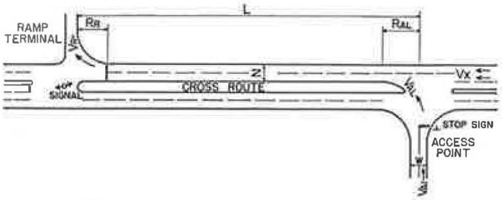
$$L = 85 + 710 + 85 + \frac{20}{2}$$

$$L = 890'$$

CONCLUSION: If a 10% tolerance in the computed Value-Guides is assumed, the length which satisfies the most critical condition is found in the range of:

$\boxed{L = 820' \text{ to } 960'}$

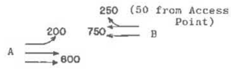
EXAMPLE 2 - CODE NO'S. 247365 Access Point Exit
247766 Access Point Entrance



Given:

Urban Area
Design speed of CROSS ROUTE = 40 MPH
10% commercial vehicles

N 2 lanes
W 20'
RR 50'
RAL 50'
Vx 920 DHV
VR 250 DHV
VAL 80 DHV
VAI 100 DHV



Traffic Volumes (DHV) at Ramp Terminal

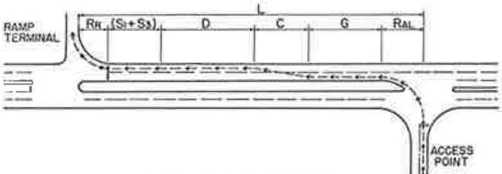
To Find: L

STEP 1 - Examine possible movements between RAMP TERMINAL and ACCESS POINT and select most critical one.

Movement I: Left turn (V_{AL}) from the ACCESS POINT to a right turn (V_R) at the RAMP TERMINAL.

STEP 2 - Write equation for critical movement

Movement I:



$L = R_{AL} + G + C + D + (S_1+S_3) + R_R$

R_{AL} = Left-turn radius from ACCESS POINT
G = Dist. travelled while seeking a gap in adjacent lane
C = Dist. travelled while changing lanes
D = Deceleration distance to stop condition
(S_1+S_3) = Storage dist. for right-turn and through traffic
R_R = Ramp Radius

STEP 3 - Determine Value - Guides for equation unknowns

Movement I:

1) Determine phasing and related green time at the CROSS ROUTE - RAMP TERMINAL intersection by means of a capacity analysis.

Phase A	A	green = 25%
Phase B	A, B	green = 65%
		amber = 10%
60 sec. cycle		TOTAL = 100%

2) Determine $(S_1 + S_3)$ for approach B :

$(S_1 + S_3) = 1000\ (1.10) \times 2 \times 25 \times (1 - 0.65) \times \frac{60}{3600 \times 2} = \frac{1000\ (55)\ (0.35)}{120} = \underline{160'}$

3) Deceleration distance from CROSS ROUTE design speed of 40 MPH to stop condition:

$D = \underline{300'}$

4) Distance travelled while changing lanes:

C = 3 seconds at CROSS ROUTE design speed of 40 MPH

$C = 3 \times \frac{40}{60} \times 88 \quad C = \underline{175'}$

5) Distance travelled while seeking a gap in adjacent land:

G = 5.8 seconds at average speed[1] of 27.5 MPH

$G = 5.8 \times \frac{27.5}{60} \times 88 \quad G = \underline{235'}$

STEP 4 - Compute L for critical movement:

Movement I: $L = R_{AL} + G + C + D + (S_1 + S_3) + R_R$

$L = 50 + 235 + 175 + 300 + (160) + 50$

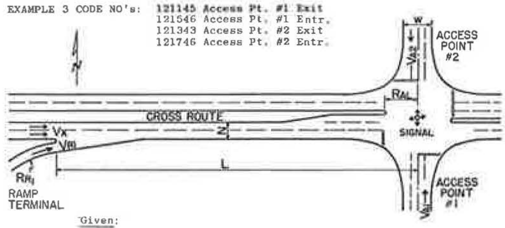
$L = 970'$

[1] Average speed = ½(design speed of CROSS ROUTE + design speed of left-turn radius at ACCESS POINT) $= \frac{40 + 15}{2} = \underline{27.5}$ MPH

CONCLUSION: If a 10% tolerance in the computed Value - Guides is assumed the length which satisfies the most critical condition is found in the range of:

$\boxed{L = 885' \text{ to } 1055'}$

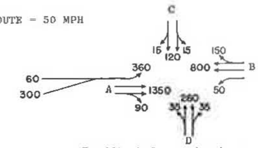
EXAMPLE 3 CODE NO's: 121145 Access Pt. #1 Exit
121546 Access Pt. #1 Entr.
121343 Access Pt. #2 Exit
121746 Access Pt. #2 Entr.



Given:

Intermediate Area
Design speed of CROSS ROUTE = 50 MPH
5% commercial vehicles

N 2 lanes
W 40'
RRI 230'
RAR 30'
RAL 50'
Vx 1200 DHV
VRI 600 DHV
VR2 300 DHV
VAI 350 DHV
VA2 150 DHV



Traffic Volumes (DHV) at ACCESS POINT

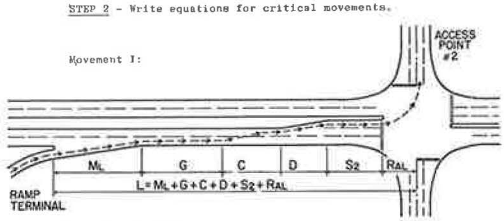
To Find: L

STEP 1 - Examine possible movements between RAMP TERMINAL and ACCESS POINT and select most critical.

Movement I: From entrance RAMP TERMINAL (V_{RI}) to a left turn at the ACCESS POINT #2 (Code no. 121145)

STEP 2 - Write equations for critical movements.

Movement I:



M_L = Merging distance
G = Distance travelled while seeking a gap in adjacent lane
C = Distance travelled while changing lanes
D = Deceleration distance to stop condition
S_2 = Storage distance for left turn
R_{AL} = Left turn radius at ACCESS POINT #2

STEP 3 - Determine Value - Guides for equation unknowns.

Movement I:

1) Determine phasing and related green time for CROSS ROUTE - ACCESS POINT intersection by means of a capacity analysis:

Phase A	A	green = 30%
Phase B	A, B	green = 40%
Phase C	C, D	green = 15%
		amber = 15%
		TOTAL = 100%

2) Determine S_2 for approach A :

$S_2 = 360\ (1.05) \times 2 \times 25 \times (1-0.30) \times \frac{60}{3600 \times 1}$

$S_2 = \frac{360\ (52.5)\ (0.70)}{60} \quad S_2 = \underline{220'}$

3) Distance required for ramp traffic to merge to the left (M_L):

$M_L = (20' - 8') \times 50{:}1$ Taper

$M_L = 12 \times 50 \quad M_L = \underline{600'}$

4) Distance travelled while seeking a gap in adjacent lane (G):

G = 7 seconds at CROSS ROUTE design speed of 50 MPH[1]

$G = 7 \times \frac{50}{60} \times 88 \quad G = \underline{515'}$

5) Distance travelled while changing lanes (C) :

C = 3 seconds at CROSS ROUTE design speed of 50 MPH

$C = 3 \times \frac{50}{60} \times 88 \quad C = \underline{220'}$

6) Deceleration distance from CROSS ROUTE design speed of 50 MPH to stop condition (D) :

$D = \underline{400'}$

STEP 4 - Compute L for critical movement:

Movement I: $L = M_L + G + C + D + S_2 + R_{AL}$

$L = 600 + 515 + 220 + 400 + 220 + 50$

$L = 2005'$

CONCLUSION: If we assume a 10% tolerance in the computed Value - Guides the length which satisfies the most critical condition is found in the range of:

$\boxed{L = 1810' \text{ to } 2200'}$ (for Code 121145)

CONCLUSIONS AND RECOMMENDATIONS

This study has demonstrated that a design aid to determine the proper spacing between interchange ramp terminals and crossroute access points can be developed. This design aid takes the form of distance equations that can be solved for various traffic volume and route speed values. A method or system for describing the possible design situations has also been developed and has produced equations for 60 of the most common and important cases. The equations comprise the design elements controlling the proper spacing for each individual design case.

The results of this limited-scope study suggest two major areas of further research:

1. An extension of the research initiated by this study in which all possible ramp terminal-access point design situations are identified, possibly through the use of the descriptive system developed herein. Equations could then be developed for all practical cases and published in chart or graphic form for use by highway designers. This material would have particular application in establishing highway department access policies toward local land developers. Ultimately, the material might be put into a form that could be used by local planning agencies in planning street systems and land uses in interchange areas.

2. Continued research on the values used for design elements comprising the equations. Lane changing distance requirements, lane speeds through interchange areas, signing distances, refinement of weaving standards, and merging distance requirements are some of the areas in greatest need of research.

In conclusion, it would seem appropriate to reiterate one of the basic principles underlying the development of these equations or value guides. They, like any other chart, graph, or design aid, are to be used as guides only, and not as a replacement of sound engineering judgment.

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Experimentation with Manual and Automatic Ramp Control

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This paper describes the planning, conduct and evaluation of a series of experiments with freeway ramp control undertaken for the purpose of improving network operations.

The development of the control plan included the identification of the critical section, determination of the period of time and degree of control, estimation of the redistribution of traffic, and re-evaluation of system operations. A comprehensive set of measurements was obtained for the network, including expressway and major arterials, for three weeks without control and for three weeks with freeway ramp control. The ramp control consisted of partially closing one on-ramp and metering traffic at a second on-ramp. The effect on the network of the freeway ramp control was evaluated on the basis of vehicle-minutes and vehicle-miles of travel for each link, route, and for the total network. The results indicate that travel time on a network basis was reduced by freeway ramp control.

•AN EXPRESSWAY Surveillance Project was established in April 1961 as a part of the research program of the Illinois Division of Highways, under the supervision of the Bureau of Research and Planning. The project is being financed with Highway Planning Survey funds made available through the Federal-Aid Highway Acts, with the State of Illinois, Cook County, and the City of Chicago contributing the necessary matching funds. An Advisory Committee, consisting of representatives of the four cooperating agencies, was appointed and meets frequently to review the progress of the project and to review and to advise on steps recommended for future experimental work.

The immediate objective of the project is to develop, operate and evaluate a pilot network information and control system to reduce travel time and to increase traffic flow. Successful progress could lead eventually to a centralized information and control system for the entire Chicago Metropolitan Expressway and major street network system.

The approach being used in accomplishing this objective is frequently referred to as a case study or pilot study approach. A typical portion of the Metropolitan Chicago highway network system was selected as the laboratory or demonstration area, and a pilot detection system is being used to measure the existing traffic patterns from which control plans can be developed. Through experimentation with control, the pilot detection system gradually will be converted to a pilot network information and control system. When complete, the system will be evaluated in terms of road user benefits and system costs. Administrative decisions can then be made as to the possible extension of the system.

The first major phase of the project, development of a pilot detection system, consisted of operational studies, surveillance equipment evaluation, system design, and installation. The pilot detection system became operational in October 1962 and the work leading to this development was described by May et al. (1).

This paper describes the planning, conduct and evaluation of a series of experiments

Paper sponsored by Committee on Freeway Operations.

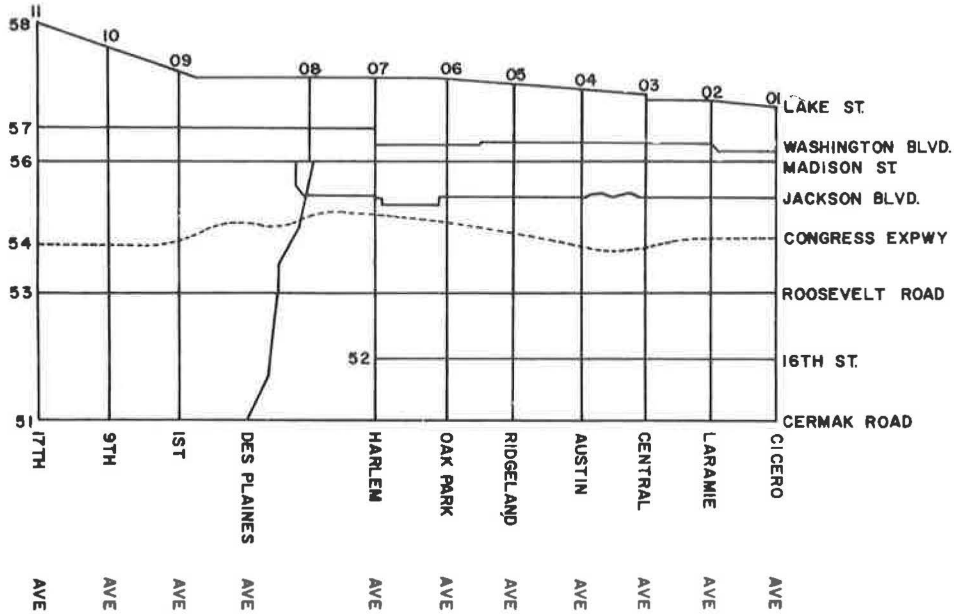

Figure 1. Network study area and inventory coding system.

with freeway ramp control which were undertaken for the purpose of improving network operations following development of the pilot detection system.

INITIAL EXPERIMENTATION WITH MANUAL CONTROL

The first step toward experimentation with control was the comprehensive collection and analysis of traffic volume and travel time data under uncontrolled conditions on the expressway and the major arterial streets. This knowledge permitted evaluation of the effectiveness of various devised control schemes, and resulted in the selection of the control plan with the least estimated total travel time and one which was feasible under actual traffic conditions. The conduct of the experiment was designed so that the plan could be evaluated on a network basis by field measurements, and so that it could be carried out with a minimum of additional equipment. The final stage of this initial experimentation with control was to evaluate the effectiveness of the control plan and to determine means for future improvement with control.

Network System Inventory

A comprehensive collection of traffic volume and travel time data was obtained for the network study area during a three-week study period (October 4 to October 24, 1962) under uncontrolled conditions. Each major arterial between intersecting major arterials was defined as a link, and volume and travel time data were summarized on a link basis. The network study area and the link coding system are shown in Figure 1. In this manner the vehicle-miles and vehicle-minutes of travel, as well as traffic volume and speeds, were computed for each link and for various periods of time. In addition to volume and travel time data, aerial observations were made during daylight hours to record any unusual events such as accidents, disabled vehicles, or other special events. An origin-and-destination study of vehicles entering the expressway within the study area was conducted in order to evaluate later the effect of various diversion schemes. Numerous secondary studies were conducted simultaneously such as an aerial density study, transit use study, persons per vehicle study, and expressway shoulder usage study.

Following the traffic data collection phase, a physical inventory was completed for the expressway and major arterials and all data were summarized on a link basis. The physical inventory included the number of moving lanes, parking conditions, width of streets, surface conditions, pavement markings and signs, and details pertaining to traffic signal equipment and existing settings.

Developing the Control Plan

The tasks performed in developing the control plan included the identification of bottlenecks, determination of period of time and degree of control, estimation of the redistribution of diverted traffic, re-evaluation of surface street system, and final evaluation of control plan on expressway, ramp and surface street traffic.

Identification of Bottleneck. —There are essentially two bottlenecks on the outbound Congress Expressway within the study area and each weekday afternoon congestion exists from approximately 4 to 6 PM. The one farthest upstream is caused by a reduction from 4 to 3 lanes without a corresponding reduction in traffic demand. The second bottleneck farther downstream and the last bottleneck on the outbound expressways are caused by fairly heavy on-ramp traffic (600 veh/hr) which has short periods of very heavy flow (20 to 25 veh/min) and is located at the top of an approximate 1,000-ft, 3 percent upgrade. The upgrade three-lane section is preceded by a reverse curve over which three, wide, closely spaced overhead structures appear to present a "tunneling effect" to the expressway motorists.

The initial experimentation with control was limited to the downstream portion of the study area which includes the bottleneck caused by the upgrade and on-ramp, and downstream from which free-flow conditions exist for the remaining portion of the outbound expressway. The reasons for this were to simplify the initial experimentation and to evaluate closely the effect of a particular controlled action. Further, if the expressway flow can be increased through control, it is apparent that the bottleneck farthest downstream should be placed under control first, and at later stages the control extended upstream.

Period of Time and Degree of Control. —Essentially the degree of control was determined by comparing the expressway free-flow capacity downstream of the on-ramp with the traffic demand prior to the on-ramp. The difference indicated the approximate maximum allowable ramp volume which could enter without resulting in congestion. A comparison of the computed maximum allowable ramp volume with the ramp demand indicated the approximate time interval when controls might be required and gave the first approximation of the amount of ramp traffic which might require diversion.

These analyses were performed using data collected in the October study, and are presented in graphical form in Figures 2 and 3. The bottleneck output (Fig. 2) for the 30-min period preceding congestion varied between hourly rates of 6,000 to 6,150 veh/hr and a free-flow bottleneck capacity of 6,000 veh/hr was assumed and is indicated. The traffic demand approaching the bottleneck on the expressway was obtained from volume measurements in advance of the bottleneck and modified to take into consideration the storage of vehicles which occurred once congestion developed. The lightweight line indicates the approach volumes, and the heavier line indicates the expressway traffic demand approaching the bottleneck. The shaded area indicates the approximate maximum allowable ramp volume for the various periods of the afternoon.

The comparison of present ramp demand with the foregoing estimated maximum allowable ramp volume is shown in Figure 3. The present ramp demand is either northbound traffic making a left into the ramp, southbound traffic making a right into the ramp, or traffic originating in the adjacent parking lot serving the transit station. This demand is depicted by the three curves across the lower portion of Figure 3. The shaded area indicates the allowable ramp volume, and it can be noted that the demand exceeds the allowable volume from 4:30 to 6:00 PM. Limiting the control at the ramp to metering without diversion was considered, but it was not deemed appropriate because the demand considerably exceeded the allowable ramp volume. A technique for handling such a situation by advanced displays on the surface streets is considered later in this paper.

Inasmuch as a portion of the diverted ramp traffic would enter the expressway at a

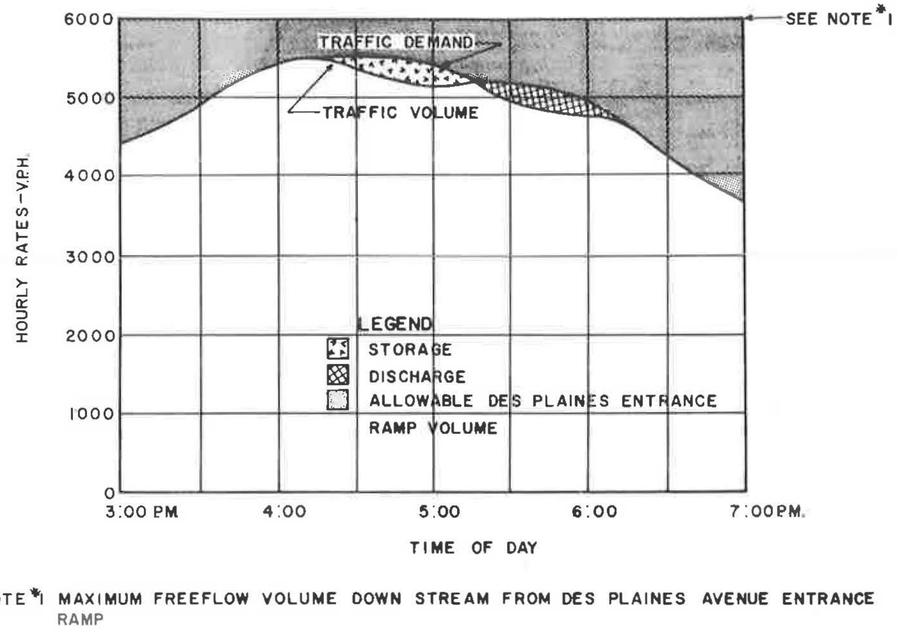

Figure 2. Traffic volume and demand—westbound Congress Street Expressway at Des Plaines Avenue.

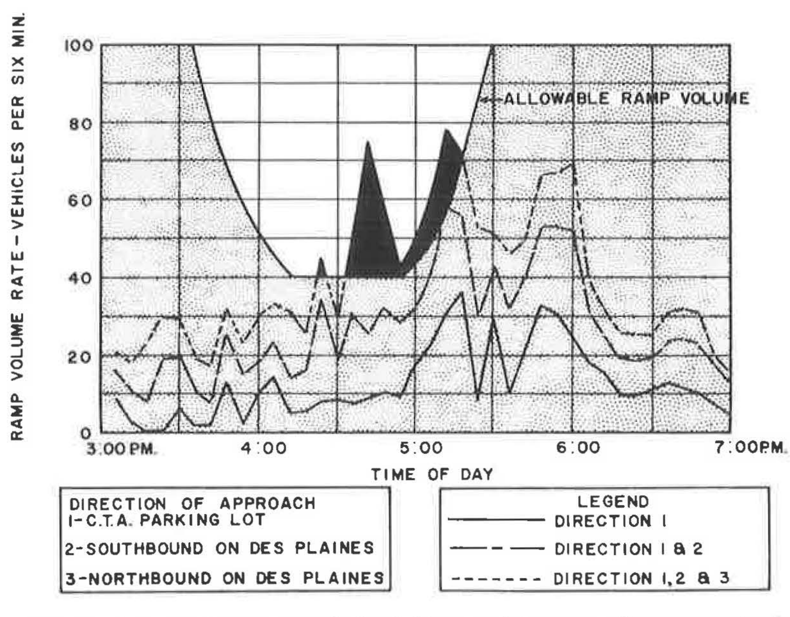

Figure 3. Estimation of period of time and degree of ramp control.

ramp upstream of the bottleneck, this would further reduce the allowable ramp volume at the bottleneck. Therefore a rather conservative action was initially proposed permitting only the traffic originating in the adjacent parking lot serving the transit station, to use the ramp. Also, if the ramp were closed to part of the ramp traffic at a specified time, some traffic might advance the time of their trip in order to enter the ramp before ramp closure. This possible increase in ramp flow might cause congestion to occur earlier and so the period of time initially proposed for partial ramp closure was changed from 4:30 to 6 PM to 4 to 6 PM.

Estimated Redistribution of Traffic. —The origin-and-destination data for the diverted ramp traffic combined with the travel time study results were employed to estimate the redistribution of the approximate 700 vehicles which would be diverted between 4 and 6 PM. The travel patterns for the ramp traffic under uncontrolled conditions (Fig. 4) indicate that approximately 80 percent of the traffic from the north originates on or north of the major arterial (Madison Street) located ½ mi north of the expressway and that 60 percent of the traffic from the south originates on or south of the major arterial (Roosevelt Road) located ½ mi south of the expressway. This traffic could proceed to the next ramp downstream without adverse distance.

The reason for selecting the traffic from the parking area as the traffic which could continue to use the ramp during control can be seen in Figure 4. Each of these vehicles would encounter an adverse distance of 1 mi. Approximately one-third of the ramp traffic from the north originates east of the on-ramp upstream and therefore would logically enter the expressway at that point and could actually save time without adverse distance. In fact, this illustrates that the traffic recognizing the normal expressway congestion postpones entering the expressway until it is downstream from the bottleneck.

The estimated redistribution of the proposed diverted traffic is shown in Figure 5. Of the 700 ramp vehicles being diverted between 4 to 6 PM, it was estimated that approximately 300 vehicles would divert to the upstream on-ramp, 300 vehicles to the downstream on-ramp and 100 vehicles would abandon the use of the expressway primarily because of the short length of trip.

Re-Evaluation of System Operation. —The redistribution of traffic discussed in the preceding section might well change the operation on the major arterials as well as on the expressway. Therefore the next task was to evaluate the effect of this change and, if necessary, to provide for additional modifications and/or controls.

In regard to the major arterials, the estimated change in traffic volumes varied from -10 percent to +5 percent. It was reasoned that this relatively small volume change on major arterials would, at most, adversely affect the traffic at signalized intersections of major arterials. This position was reinforced further by the fact that the two parallel major arterials carried a substantial portion of the expressway traffic before the opening of the expressway in 1960.

Some 30 signalized intersections which might be affected were reviewed and from these, eight signalized intersections were selected for detailed study. Intersection delays, queue lengths, turning movements under controlled conditions and estimated changes in turning movements in the event of diversion were analyzed. These studies indicated that the traffic demand would exceed substantially the capacity at only one intersection because of an estimated increase of 160 left-turn movements in the 2-hr period. A second capacity study indicated that the addition of a left-turn phase would result in below capacity conditions and satisfactory operations. The detailed study also revealed that two other intersections, the intersections of major arterials at the mouth of the on-ramps immediately upstream and downstream from the ramp in question, would operate near or slightly above capacity. However, no changes were made at these two locations. All calculations indicated that the other intersections would operate satisfactorily, although some increase in delay could be anticipated. These signals were inspected and adjusted for best operations for normal traffic conditions before control.

The estimated redistribution of traffic did require a re-evaluation of the anticipated expressway operations at the on-ramp location immediately upstream, at the upgrade near the bottleneck, at the on-ramp near the top of the grade, and at the next on-ramp downstream.

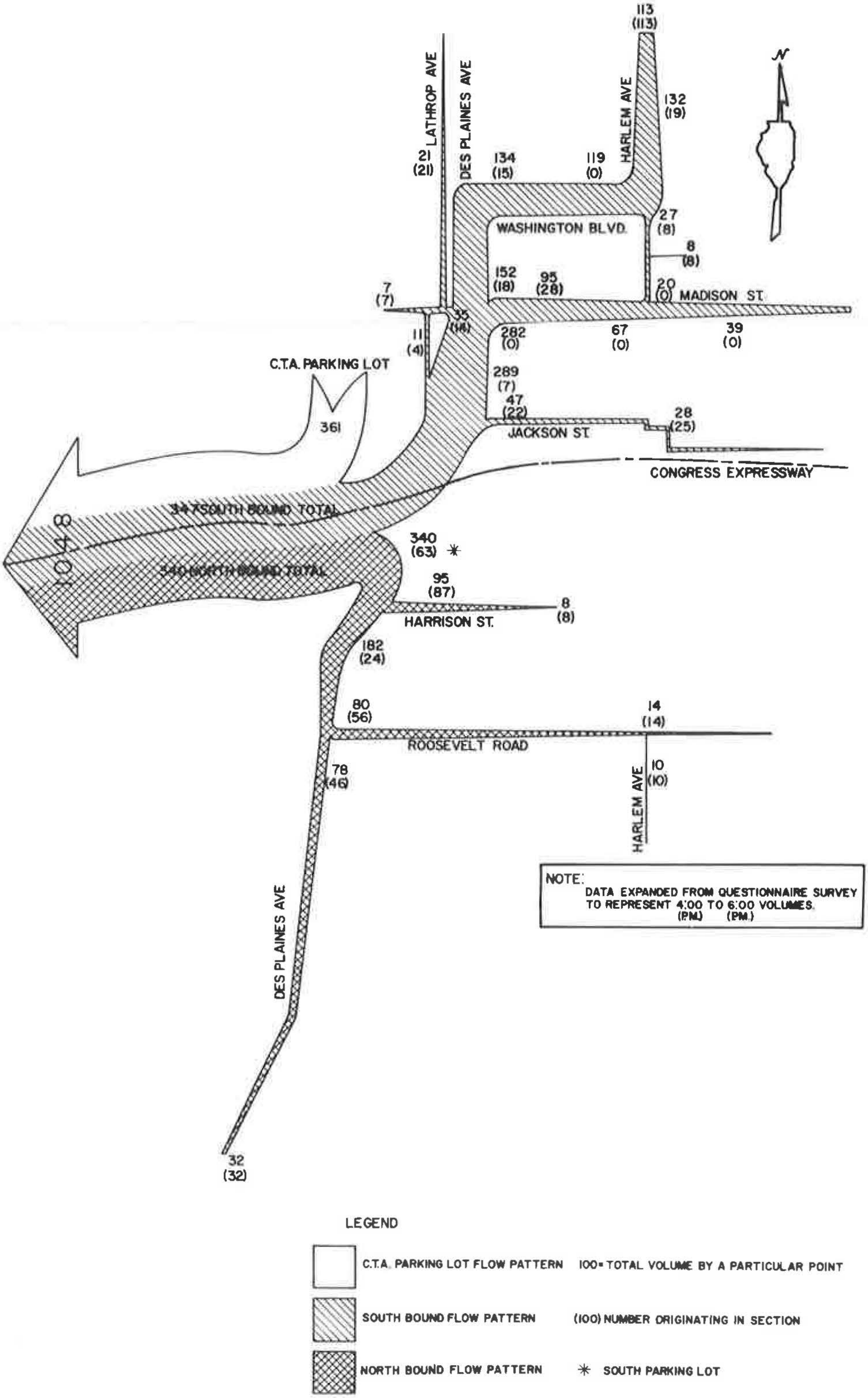

Figure 4. Present flow pattern of Des Plaines ramp traffic.

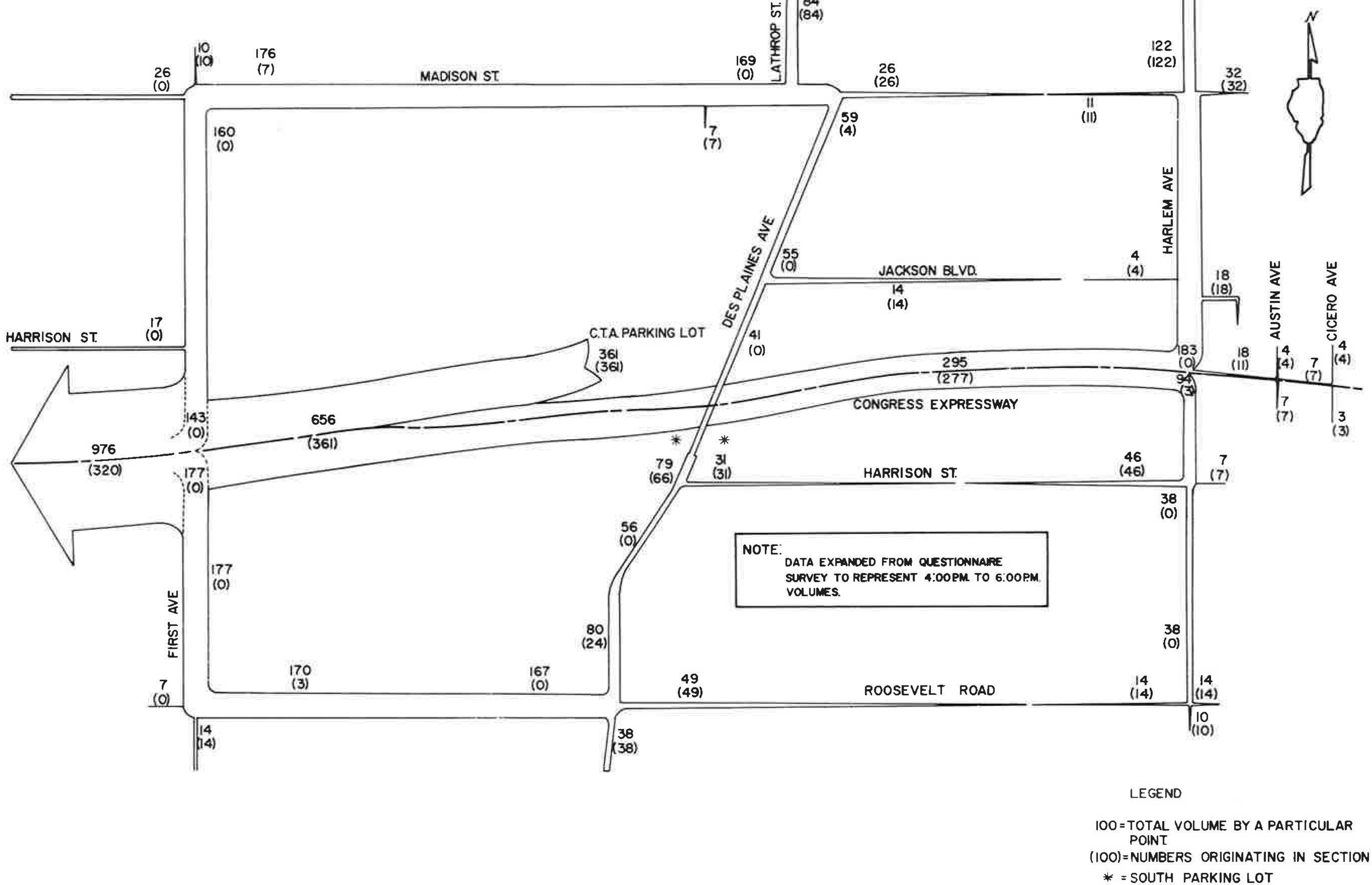

Figure 5. Estimated flow pattern of Des Plaines ramp traffic with partial diversion.

At the on-ramp location immediately upstream the anticipated increase in ramp traffic was of some concern. It was estimated that the merging volumes (5,300 veh/hr on the mainline and 400 veh/hr on the ramp) would not result in congestion, and it was decided that metering would be the most severe control required. In analyzing the expressway upgrade conditions (5,700 veh/hr), it was decided that if the merging free-flow capacity immediately downstream was 6,000 veh/hr and provided free flow could be maintained on the grade, the upgrade should not produce a capacity restraint.

The diversion of 700 vehicles from the ramp located near the top of the grade appeared to be more than adequate to maintain free-flow conditions. Estimates were made which indicated that diverting as few as 400 vehicles during the 2-hr period would be sufficient to maintain free-flow conditions. However, there did not appear to be any practical means of permitting any other vehicles on the ramp in addition to the traffic from the transit parking lot on a time-clock basis.

The location of greatest concern was the next on-ramp downstream where it was estimated that 300 additional vehicles would be diverted. The estimated merging (5,500 veh/hr and 600 veh/hr) indicated that congestion might exist and considerable thought was given to means of alleviating this potential problem. Fortunately, a 3-lane, one-way frontage road originated at this ramp and continued beyond two off-ramps, and terminated at the next on-ramp further downstream. It was concluded that ramp metering might be required, and a metering plan was developed which is described in the section pertaining to conduct of field experimentation. In the event that metering resulted in a sufficiently long queue causing undue delay to the motorists and hazard at the nearby intersection, the traffic would be directed to use the frontage road and the next on-ramp. An estimated 200 veh/hr could be directed to the next ramp without creating a merging problem on the expressway.

With the completion of this final operational re-evaluation, the next and last step before conducting the field experimentation was to estimate the effect of the total control plan on the motorists in terms of travel time and vehicle-miles of travel.

Evaluation of Control Plan. —The three groups of motorists affected by the control plan were the diverted ramp traffic, expressway traffic and the major arterial traffic. The effect on each is described in the following paragraphs.

The 700 vehicles which were diverted from the ramp were classified into origin-and-destination groups and their individual trip distances and trip travel times under normal conditions and under the proposed control plan were determined. The determination of travel time under normal conditions and trip distances under both conditions were obtained easily. However, estimation of travel time under controlled conditions had to take into account the increased travel time, particularly at two or three of the more critical signalized intersections, due to increased flow on major arterials. These additional delays due to increased flow were considered as time penalties for vehicles making a particular turning movement at an intersection which was affected by the control plan. The estimated effect on travel time and distance for each origin-and-destination group is shown in Figure 6. From the figure it can be seen that the 111 trips originating in zone 6 would have an increased trip travel time of 4.5 min and an increased trip travel distance of 0.7 mi; also, some of the diverted ramp traffic would be benefited. For example, it was estimated that the 130 trips originating in zone 15 would have a reduced trip travel time of 1.5 min, and a reduced trip travel distance of 0.1 mi. The estimated over-all effect on the diverted ramp traffic for the 2-hr period was an increase of 300 veh-min and an increase of 11 veh-mi.

The approach used to evaluate the effect of the proposed control plan on expressway traffic was to estimate the flow rate under control conditions for various locations along the expressway study section, and then from previously determined flow-speed curves, to calculate the travel time. The difference between the calculated travel time and measured travel time under normal conditions multiplied by the number of expressway vehicles served, resulted in a reduced travel time of 13,900 veh-min.

The effect of the proposed control plan on major arterial traffic was determined by comparing the results of previously conducted travel-time studies with estimates of travel time with control. The estimates of travel time with control were based on the previously obtained travel-time data modified to include additional delays for particular

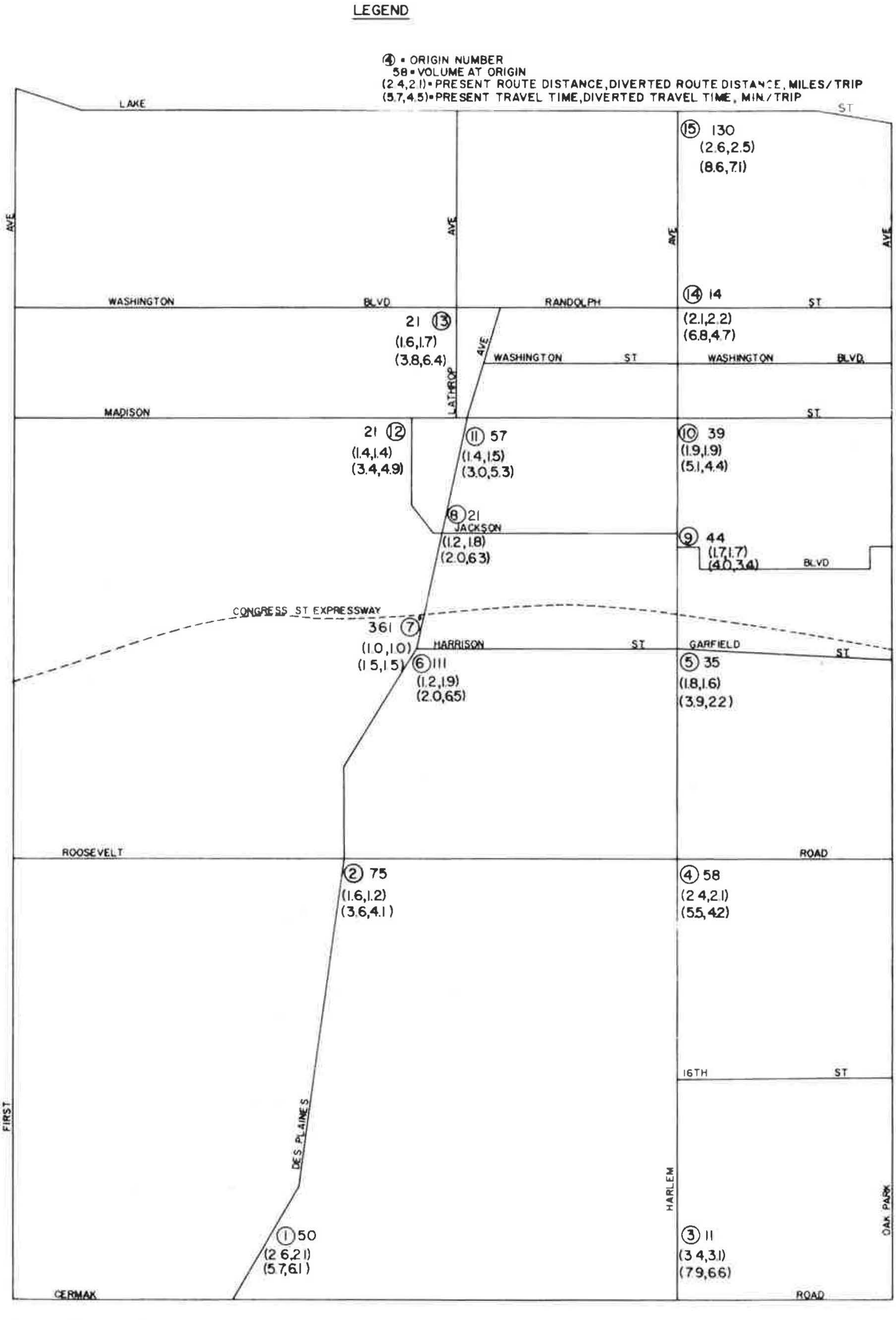

Figure 6. Ramp traffic travel time and mileage as affected by Des Plaines ramp control (4-6 PM travel time and volume).

TABLE 1

Traffic Type	Travel Time (veh-min)		
	Normal Conditions	Controlled Conditions	Effect on Travel Time
Diverted ramp	3,900	4,200	+ 300
Expressway	44,600	30,700	-13,900
Major arterial	35,100	37,600	+ 2,500
Total	83,600	72,500	-11,100

turning movements at selected intersections. The estimated amount of increased travel time to the major arterial traffic was 2,500 veh-min.

Table 1 gives a summary of the travel-time investigations discussed in the preceding paragraphs.

In April 1963, the Project Advisory Committee approved the recommendation of the staff that final plans be made to conduct the proposed experimentation with control.

Conduct of Control Experimentation

The general plan developed for control experimentation is shown in Figure 7 and consists of three parts: before measurements, publicity and coordination, and control with measurements. The before-measurement phase was designed to provide the standard or basis which the operations with control could be compared and evaluated. The publicity and coordination activity consisted of informing the public, particularly the motorists, and representatives of several governmental agencies of the plans for control experimentation. The final phase of the plan was the application of control and operational measurements.

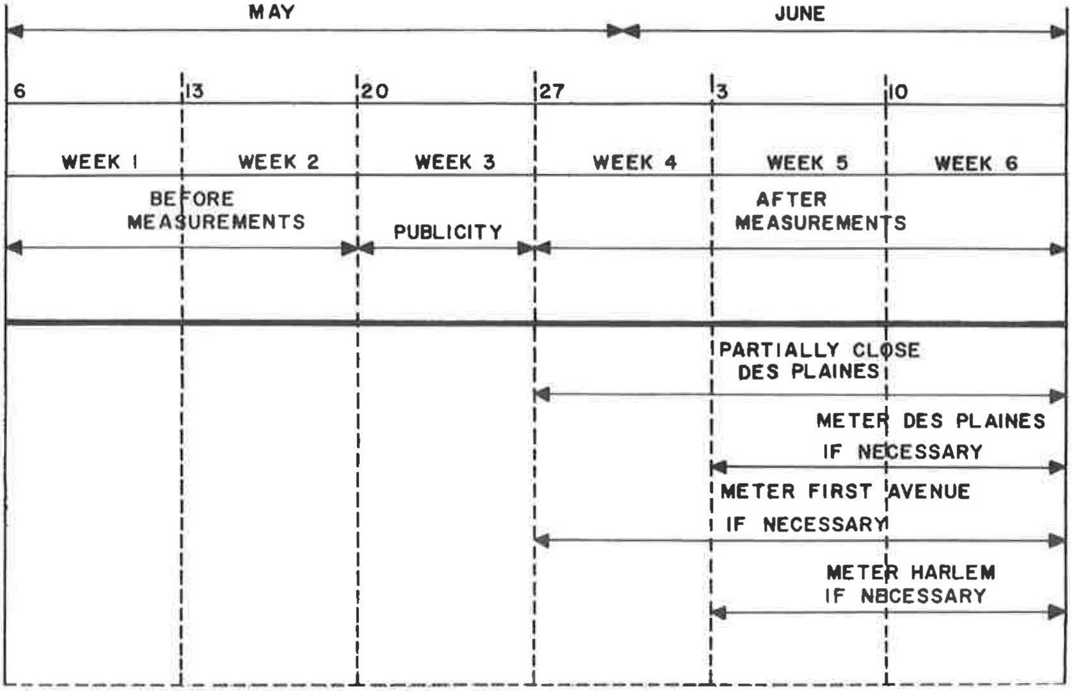

Figure 7. Study calendar.

Before Measurements.—It was planned that the measurements obtained during the 2-week-before study would be identical to measurements obtained during the following 3-week study with control. The period of time for control was 4 to 6 PM and the period of time selected for measurement was 3 to 7 PM.

On the expressway the pilot detection system recorded on punched tape the minute volume and occupancy or speed for the seven mainline stations and for each off- and on-ramp. Arrangements were made with the U. S. Bureau of Public Roads for the use of its impedance vehicle which is a specially equipped vehicle for recording individual vehicle performance. An emergency patrol vehicle was assigned to the expressway study section to inform the project office by radio of unusual conditions and to remove any hazards as quickly as possible.

On the surface streets approximately 25 portable 15-min traffic recorders were installed for purposes of obtaining volume and vehicle-miles of travel on the various links of network system. A license plate travel-time study was made on the major arterials for purposes of obtaining vehicle-minutes of travel on the various links of the network system. Plans were made for obtaining turning movements at selected intersections and for general evaluation of intersection performance.

In addition to the specific measurements planned for the expressway and major arterials, ten afternoons of aerial photography were planned for the before study and the study with the control. The purpose of the aerial photography was to record permanently the conditions with and without control, to permit the determination of queue lengths at intersections and ramps, and most important, to investigate a proposed technique for estimating total travel time in a network. The simplicity of measurement and the importance of network travel time have created considerable interest in this particular activity. Essentially, time lapse photographs were taken throughout the afternoon period from an airplane which flew back and forth over the network study area. The photographs are analyzed by counting the number of vehicles in each directional link, and travel time was obtained by multiplying the number of vehicles by the time interval between photographs. Average travel time can be obtained by dividing the travel time in vehicle-minutes by the vehicle-miles computed from the traffic counts.

Publicity and Coordination.—The publicity phase consisted of preparing news releases for newspapers, radio stations and television stations, placement of temporary roadway signs in advance of the ramp to be partially closed, and traffic bulletins handed to the motorists using the ramp which was to be partially closed. In addition, the office address and phone number were included with the news releases and traffic bulletins and arrangements were made for communicating additional information in response to phone calls.

An important function of this advance planning included the coordination with representatives of governmental agencies. Arrangements were made for meetings with several of the villages which might be affected by the control plan, with State and local police, with transit authority representatives and with the various agencies sponsoring the project. Providing advance information to those agencies and obtaining ideas and comments from them proved valuable.

It should be stressed that in accordance with the control plan shown in Figure 7, the before measurements were completed before the publicity began and discussions held with governmental agencies were of a confidential nature until the completion of the before measurements. There was some concern that otherwise the publicity and coordination might have affected the before measurements.

Control with Measurement.—During the last three weeks of the experimentation it was planned that control would be exerted and measurements taken. The first week of control was generally thought of as a transitional period for the changing traffic pattern and as a period for fine tuning the metering technique. This first week included a national holiday which may have affected the normal traffic pattern. The last two weeks were planned to be used for evaluational purposes.

The partial ramp closure was handled by placing a barricade across the major entrance to the ramp at 4 PM each weekday afternoon and removing the barricade at 6 PM. Arrangements were made for a State Police officer to be present during the period the ramp was closed. Signs were posted well in advance of the ramp reminding the motorists of the ramp closure.

Two metering techniques were developed for the on-ramp immediately downstream if it was found that such action was required. One technique utilized an occupancy measurement on the mainline just upstream of the on-ramp; the other utilized a volume measurement on the mainline about ½ mi in advance of the on-ramp, and an off-ramp volume between the mainline volume measurement and the on-ramp. After further study the former technique based on occupancy was selected.

From previous measurements a relation was established between the mainline occupancy in the middle lane (in advance of the on-ramp) and the maximum safe ramp volume provided that free flow existed further downstream. From this information a metering rate was established for various levels of occupancy. The rules of the game for ramp metering were developed and the essential elements are, as follows:

1. Metering commenced as soon as the mainline lane two occupancy equaled or exceeded 15 percent (yellow indication on map display) and provided it was after 4 PM.

2. The metering rate was set based on the 1-min digital percent occupancy of the mainline lane two. The permitted metering rate for various levels of occupancy is

% of Occupancy	Metering Rate
15-16	13 vpm
17-18	10 vpm
19-20-21	8 vpm
22-23-24	6 vpm

3. The ramp entrance was closed as soon as the mainline lane two occupancy equaled or exceeded 25 percent (red indication on ramp display) and provided it was after 4 PM. The queue already on the ramp was to be discharged at a metering rate of not more than 6 veh/min.

4. The metering rate was never to exceed the metering rates previously stated, but could have been reduced under the following specific circumstances: (a) ramp demand less than metering rate, (b) ramp occupancy equal to or greater than 25 percent (red indication on map display), and (c) ramp was closed and queue was small.

5. Metering operation was to be terminated as soon as the mainline lane two occupancy was less than 15 percent (green indication on map display) and provided it was after 6 PM.

To implement this metering technique, the metering decision was made in the central office and transmitted by phone to the field station. The field personnel then set a timer which, at regular intervals, sounded a bell and indicated to the State Police officer directing ramp traffic that a vehicle could enter the expressway. For example, if a metering rate of 10 veh/min was selected, the bell would ring every 6 sec. This use of simplified equipment was in keeping with the thought that more sophisticated equipment should not be developed until the possible benefits of metering could be evaluated.

Evaluation of Control Experimentation

The experimentation with control was conducted essentially as described in the previous section. Of the four possible control actions indicated in Figure 7, partially closing the Des Plaines ramp and metering the First Avenue ramp was the only control that was necessary. Inasmuch as the required action only affected a portion of the total pilot network, a revised study area was selected. The locations of ramp control and the revised study area are shown in Figure 8.

Unpredictable traffic and weather events occurred during the 10 days of before measurements and the 14 days of after measurements which affected the traffic flow on the expressway to varying degrees and for various durations of time. The time and occurrence of these unpredictable events such as accidents, vehicle disabilities, foreign objects on the pavement, pedestrians on the roadway, emergency maintenance, and adverse weather are summarized in Figure 9. To directly relate the differences between the before and after measurements to the ramp control experimentation, it was necessary to select days which were free of the unpredictable traffic and weather events. Therefore three consecutive event-free midweek days (Tuesday, Wednesday, Thursday) were selected from the before study and the after study, and the comparison of the two sets of data served as a basis for evaluating the control experimentation.

It is unfortunate that there is not a similar direct method of evaluating the effect of the ramp control experimentation during periods which included the unpredictable events. Obviously similar events did not occur at the same location and at the same time in both the before and after studies and therefore an objective evaluation was not possible. However an unpredictable event occurred on two days during the period when controls were being applied which permit some subjective observations. On Friday, May 31 at 4:58 PM and on Monday, June 3 at 4:47 PM, an incident occurred on the expressway lanes between the First Avenue on-ramp and the Seventeenth Avenue on-ramp (see Fig. 8 for ramp locations). These incidents caused a severe reduction in expressway capacity and resulted in congestion extending upstream. Before congestion reached the First Avenue on-ramp, the metering rate was reduced and soon afterwards the ramp was closed. The traffic normally enter ing at this ramp was diverted along the frontage road and entered the expressway by the Seventeenth Avenue on-ramp which was downstream of the incident location. This controlled action benefited the diverted ramp traffic as well as the expressway traffic. Because of the large numbers of such events occurring on the expressway, and because of the serious consequences that often result, the control of entering traffic during periods when these events do occur will most likely result in benefits greater than during periods of time when unpredictable events do not occur.

Effect on Traffic Volumes. — Changes in traffic volumes on the various links of the study area were

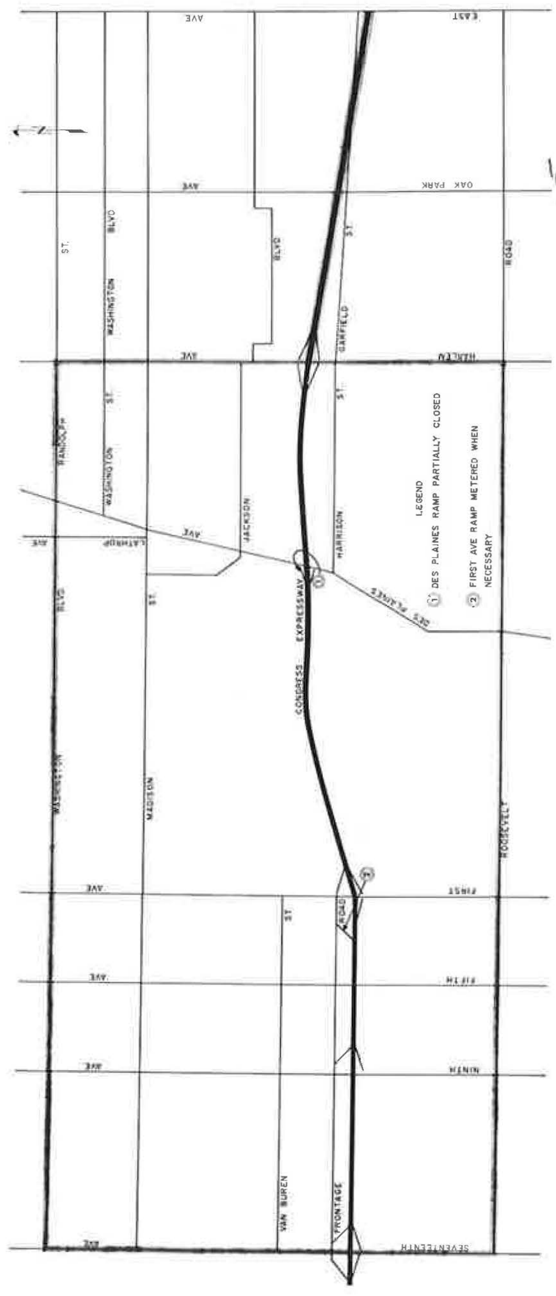

Figure 8. Location of ramp control and revised study area.

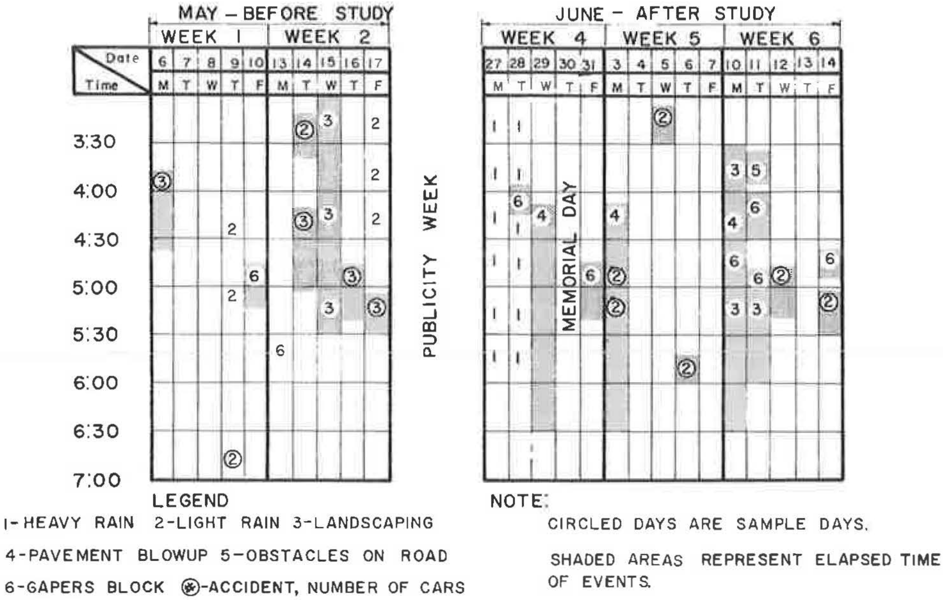

Figure 9. Traffic and weather conditions during, before and after measurements.

obtained for 4 to 5 PM, 5 to 6 PM, and for the total 2-hr period, and are graphically presented in Figures 10, 11, and 12, respectively.

Between 4 and 5 PM, the expressway volume upstream of the Des Plaines ramp was only slightly increased while the expressway volume downstream of the Des Plaines ramp was reduced by approximately 250 vehicles. The volumes on the surface streets increased, except for Des Plaines Avenue between Roosevelt and Madison, and for several streets parallel to the expressway between Harlem and Des Plaines. The total westbound corridor volume between Des Plaines and First Avenue was only slightly increased.

Between 5 and 6 PM the expressway volume upstream of the Des Plaines ramp was increased by 280 vehicles while the expressway volume downstream of the Des Plaines ramp was reduced by 60 to 110 vehicles. The volumes on the surface streets increased except for Des Plaines between Roosevelt and Madison, and for the several streets parallel to the expressway between Harlem and Des Plaines. The total westbound corridor volume between Des Plaines and First Avenue was increased by approximately 90 vehicles.

The measured changes in traffic volumes for the total 2-hr period (Fig. 12) indicate that the westbound volume through the critical corridor (Des Plaines to First Avenue) was increased by 120 vehicles during the 2-hr period. The original estimated change in flow pattern (Fig. 5) compares favorably with the measured change in traffic volumes for the 2-hr period. On the expressway the volume change immediately upstream of Des Plaines ramp was estimated to be an increase of 295 vehicles, while the measured change was an increase of 330 vehicles. Similarly, the volume change immediately downstream of the Des Plaines ramp was estimated to be a decrease of 392 vehicles, while the measured change was a decrease of 320 vehicles. The estimated increase in volume of traffic moving toward the expressway on First Avenue compared closely with the measured change (160 compared with 149 vehicles and 177 compared with 149 vehicles). Similar close agreements in traffic volume changes occurred on Harlem and Roosevelt. The greatest difference between the estimated change in volume and the measured change occurred on the expressway downstream of First Avenue. A decrease of 72 vehicles had been estimated while the measured change was a decrease of 330 to 360 vehicles. This greater than expected decrease in expressway volume is compensated for by the increase in the frontage road volume. Apparently 300 to 350

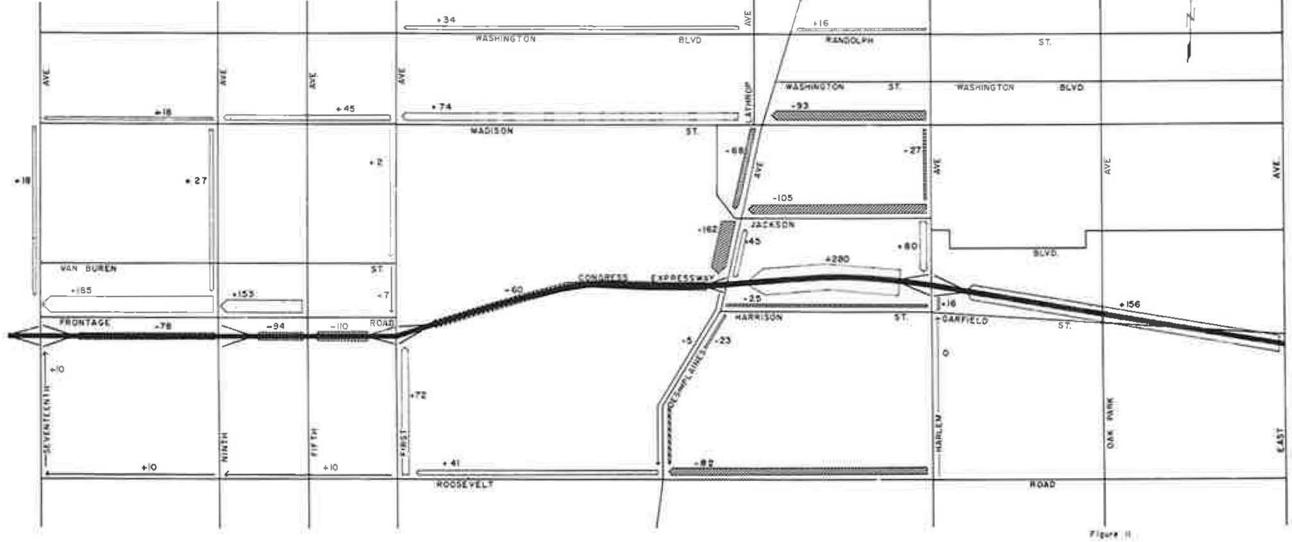

Figure 10. Change in network volume (4 to 5 PM) due to ramp control.

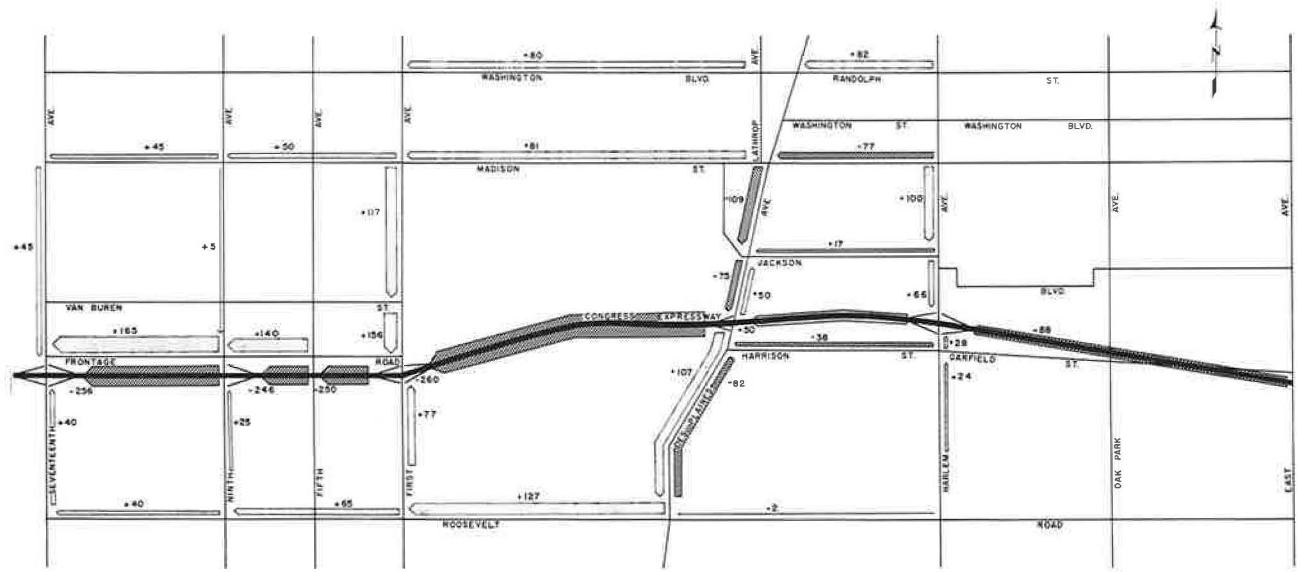

Figure 11. Change in network volume (5 to 6 PM) due to ramp control.

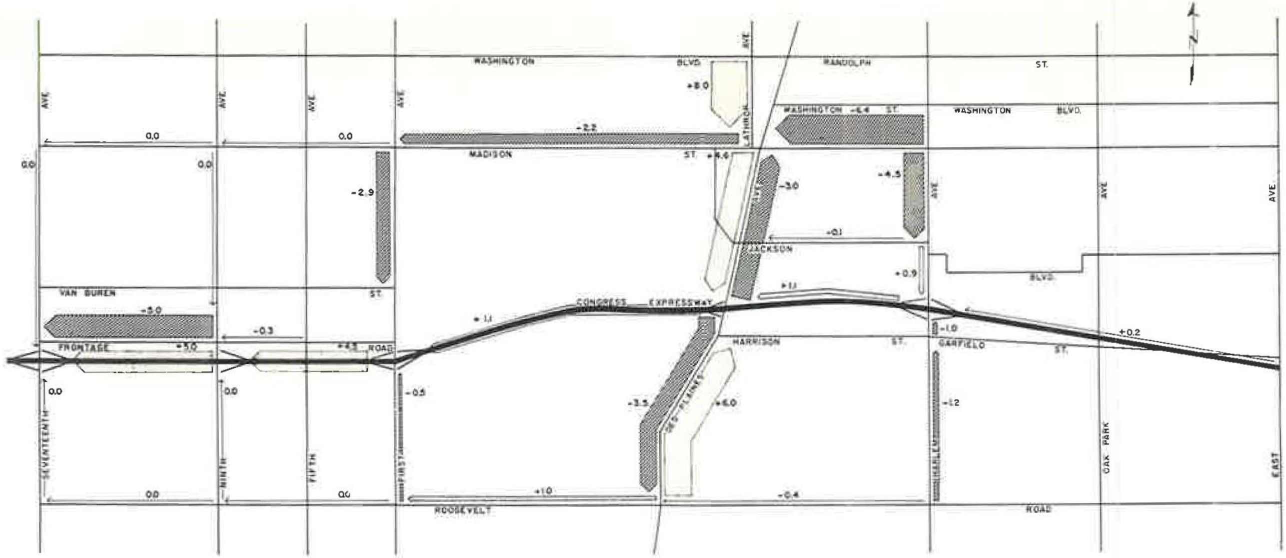

Figure 12. Change in network volume (4 to 6 PM) due to ramp control.

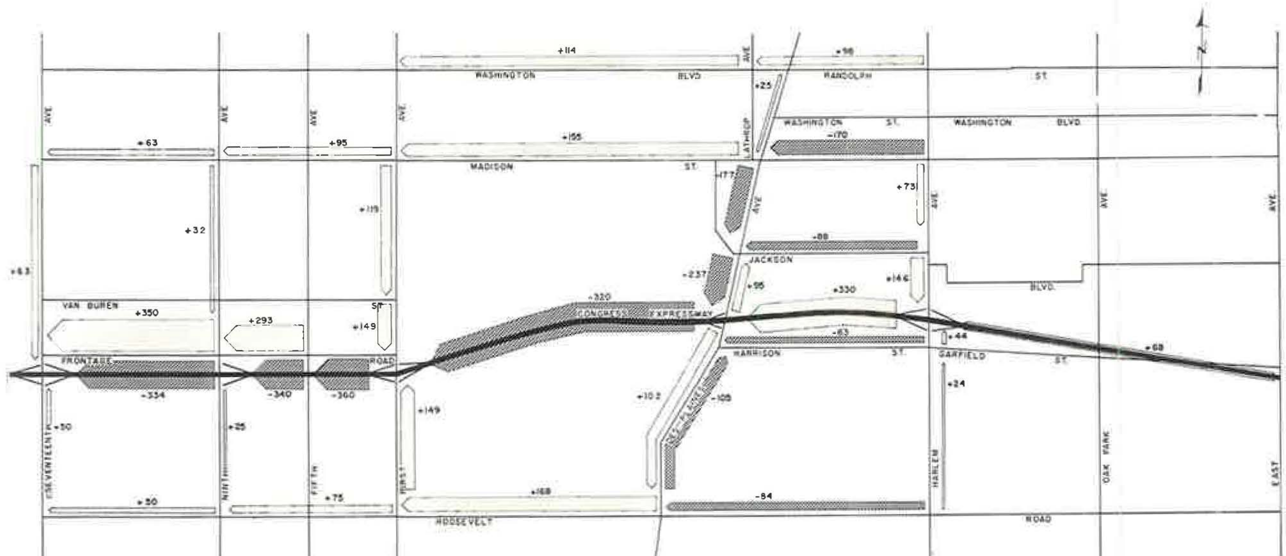

Figure 13. Change in network speed (4 to 5 PM) due to ramp control.

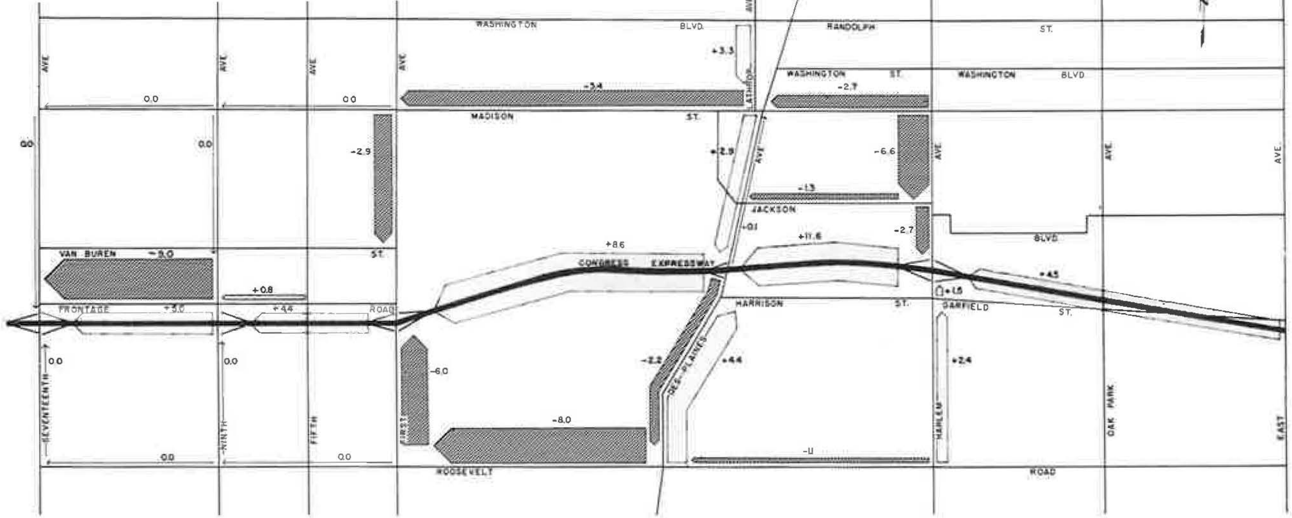

Figure 14. Change in network speed (5 to 6 PM) due to ramp control.

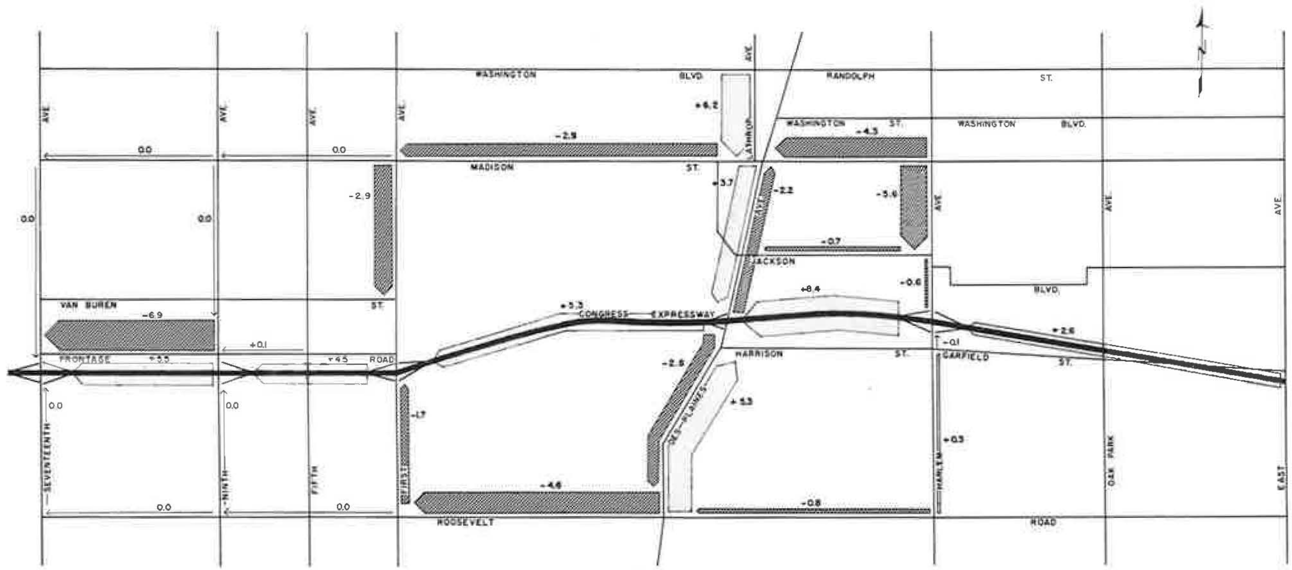

Figure 15. Change in network speed (4 to 6 PM) due to ramp control.

vehicles either proceeded west on the surface streets beyond First Avenue and then traveled to the Seventeenth Avenue ramp via the frontage road or, on approaching the metered First Avenue ramp, the traffic selected the frontage road and entered the expressway by the Seventeenth Avenue on-ramp.

Effect on Average Speeds.—Changes in average speeds on the various links of the study area were obtained for 4 to 5 PM, 5 to 6 PM, and for the total 2-hr period, and are graphically presented in Figures 13, 14 and 15, respectively.

Between 4 and 5 PM average speeds on the expressway were only slightly increased (0.2 to 5.0 mph). Average speeds on the surface streets varied from an increase of 8.0 mph to a decrease of 6.4 mph. Generally speaking, surface streets with increased traffic volumes had slightly reduced average speeds, while surface streets with decreased traffic volumes had slightly increased average speeds.

Greater changes in average speeds occurred between 5 and 6 PM. On the expressway, speeds increased by 4 to 12 mph. Average speeds on the surface streets varied from an increase of 4.4 mph to a decrease of 9.0 mph.

The measured changes in average speeds for the total 2-hr period (Fig. 15) indicate that speeds on the expressway and on Des Plaines Avenue toward the expressway increased, while speeds on the remainder of the surface streets decreased. The largest increase in average speeds occurred on the expressway between Harlem and Des Plaines (8.4 mph), while the largest decrease in average speeds occurred on the frontage road

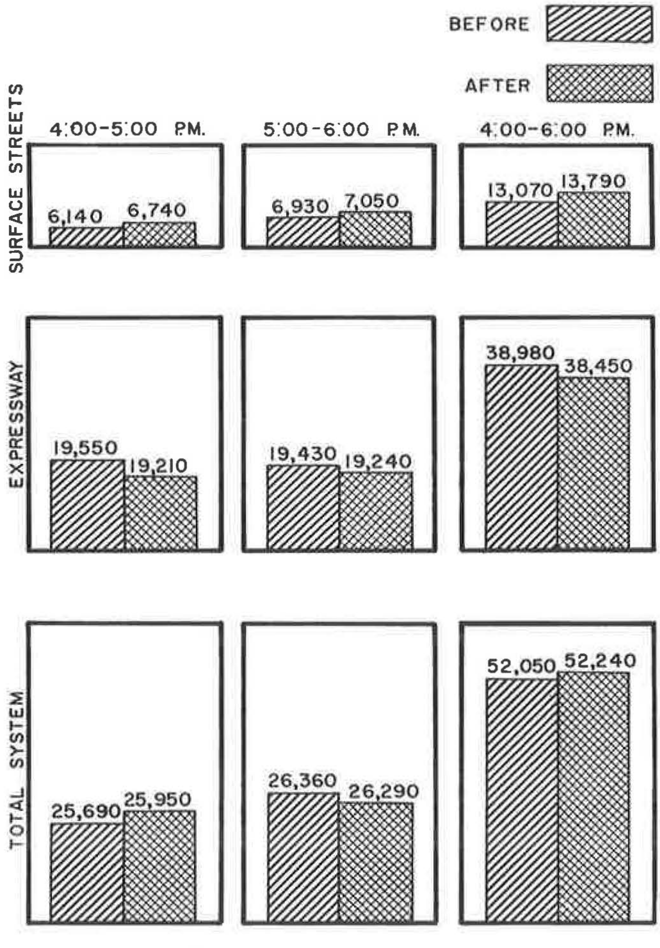

Figure 16. System travel—vehicle-miles.

between Ninth and Seventeenth Avenue (6.9 mph). In addition to the fact that more vehicles were affected by the speed changes on the expressway, the magnitude of the increase speeds on the expressway was slightly larger than the decrease speeds on the surface streets.

Evaluation Summary.—The volume measurements obtained for each link of the study area during the before and after studies permitted the calculation of vehicle-miles of travel for each link, route, network sub-system, and for the total network. A summary of the effect of control on vehicle-miles of travel is shown in Figure 16.

There was an increase of 600 (9.8 %), 120 (1.7 %), and 720 (5.5%) veh-mi of travel on the surface streets for the period of 4 to 5, 5 to 6, and 4 to 6 PM, respectively. On the expressway the travel for the period 4 to 5, 5 to 6, and 4 to 6 PM decreased 340 (1.7%), 190 (1.0%), and 530 (1.4%) veh-mi, respectively. For the total system of surface streets and expressway the travel for the period 4 to 5, 5 to 6, and 4 to 6 PM changed by 260 (+1.0%), -70(-0.3%), and 190(+0.4%), respectively. The results indicated that between 4 and 5 PM more traffic was diverted to the surface streets than required, and for each of the 1-hr periods there was only a slight change in travel on the total system.

The average speeds combined with the volume measurements obtained for each link of the study area during the before and after studies permitted the calculations of vehicle-minutes of travel for each link, route, network sub-system, and for the total network. A summary of the effect of control on vehicle-minutes of travel is shown in Figure 17.

There was an increase of 4,400 (22.6%), 3,700 (18.0%), and 8,100 (20.2%) veh-min of travel on the surface streets for the period of 4 to 5, 5 to 6, and 4 to 6 PM, respectively. On the expressway the travel for the period 4 to 5, 5 to 6, and 4 to 6 PM decreased 1,400 (5.1%), 9,200 (23.8%) and 10,600 (16.0%) veh-min, respectively. For the total system of surface streets and expressway the travel for the period 4 to 5, 5 to 6, and 4 to 6 PM changed by +3,000 (+6.4%), -5,300 (-8.9%), and -2,500 (-2.4%), respectively.

In comparing the estimated change in vehicle-minutes with the measured change, it is apparent that the diversion of ramp traffic increased the travel time on the surface streets more than anticipated, and did not decrease the travel time on the expressway as much as anticipated. It was estimated that, with control, the vehicle-minutes of travel on the surface streets would be 43,100 veh-min, while actual measurements indicated 48,100 veh-min, or a further increase of 5,000 veh-min. A review of the increases on the various links indicated that on First Avenue between Madison Avenue and the expressway the estimated increase was 600 veh-min, while the measured increase was 3,700 veh-min, or a further increase of 3,100 veh-min. The error in the estimate for this one link accounted for more than one-half of the difference between the estimated and measured vehicle-minutes of travel on the total surface streets. In reference to the expressway, it was estimated that with control the vehicle-minutes of travel would be 51,800, while actual measurements indicated 55,100 veh-min, or a difference of 3,700 veh-min. This smaller than expected reduction in vehicle-minutes of travel on the expressway was apparently due to greater volumes than expected on the expressway section upstream of the Des Plaines on-ramp and an overestimation of the capacity of the section upstream of the Des Plaines on-ramp where there is a reverse curve and a 1,000-ft three percent up-grade.

In addition to the vehicle-miles and vehicle-minutes of travel on the expressway for the selected six days (three days of the before study and three days of the after study), similar calculations were made for the remaining days of the study. Vehicle-miles, vehicle-minutes, and average speeds for 4 to 5, 5 to 6, and 4 to 6 PM for each study day are given in Table 2.

Conclusions.—The following are the conclusions drawn from the initial experimentation with manual control:

1. Origin-and-destination studies of ramp traffic proved extremely valuable in estimating the redistribution of traffic on the network.

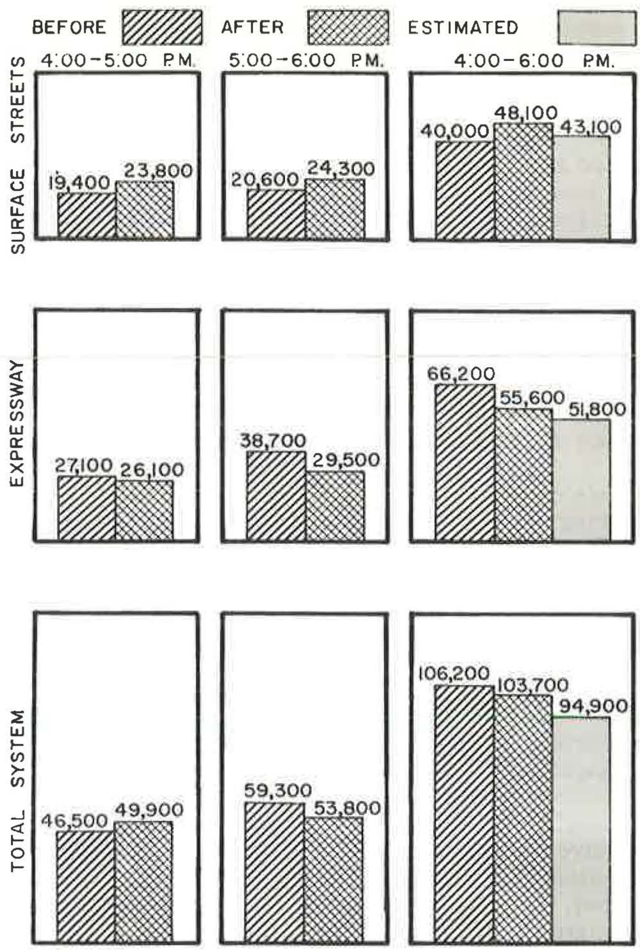

Figure 17. System travel—vehicle minutes.

2. Considerable difficulty was encountered in accurately estimating the changes in travel time, particularly on those links operating at or near capacity.

3. The selection of the period of time and the degree of control is extremely critical in optimizing network operations and, in the case of the Des Plaines ramp, control on most days was exerted too soon and was too restrictive.

4. An informed public will cooperate when reasonable controls are exerted and are generally quick to adjust to a revised control of network operations.

5. The high frequency of unpredictable events, which may have serious consequences on expressway traffic, focuses attention on the need for rapid detection of such events and requires that any control system must have flexibility for handling traffic when such events are detected.

6. Ramp metering as a technique for controlling expressway operations proved satisfactory in maintaining free-flow conditions, permitting maximum entry to the expressway, and providing smooth merging operations. However, metering on a uniform time spacing basis was not practical due to the presence of certain longer vehicles which required greater time for clearance.

7. In this particular control experimentation the vehicle-miles of travel on the surface streets slightly increased, while the vehicle-miles of travel on the expressway slightly decreased. The vehicle-miles of travel on the total network remained essentially unchanged.

TABLE 2

CONGRESS EXPRESSWAY

Week	Day	Veh-Mi 4-5	Veh-Mi 5-6	Veh-Mi 4-6	Veh-Min 4-5	Veh-Min 5-6	Veh-Min 4-6	Avg. Speeds 4-5	Avg. Speeds 5-6	Avg. Speeds 4-6
(a) Before Study										
	May 6	16,880	19,960	36,840	27,300	42,200	69,500	37.1	28.4	31.8
	7	19,250	20,060	39,310	25,900	40,000	65,900	44.5	30.2	35.8
	8	19,800	20,110	39,910	27,200	33,200	60,400	43.6	36.3	39.7
One	9	19,600	18,120	37,720	29,500	43,000	72,500	39.9	25.3	31.2
	10[1]									
	13	19,440	19,520	38,960	29,500	34,300	63,800	39.4	34.1	36.6
	14	16,090	18,700	34,790	24,700	34,400	59,100	39.1	32.6	35.3
	15	19,400	20,200	39,600	23,800	28,900	52,700	48.9	42.0	45.1
Two	16	17,730	18,110	35,840	21,700	27,400	49,100	48.9	39.6	43.7
	17	19,010	19,150	38,160	25,000	32,800	57,800	45.5	34.9	39.5
Averages		18,580	19,330	37,910	26,100	35,100	61,200	42.7	33.0	37.1
(b) After Study										
	June 3	18,710	17,590	36,300	-[1]	-[1]	-[1]	-[1]	-[1]	-[1]
	4	19,350	18,730	38,080	24,900	29,000	53,900	46.4	38.6	42.4
	5	19,090	19,450	38,540	26,400	29,000	55,400	43.4	40.1	41.7
Five	6	19,180	19,540	38,720	27,000	30,500	57,500	42.6	38.5	40.4
	7	18,810	19,250	38,060	27,200	33,200	60,400	41.5	34.7	37.8
	10	16,790	18,080	34,870	29,400	33,100	62,500	34.3	32.8	33.4
	11	18,420	18,760	37,170	27,300	39,000	66,300	40.5	28.9	33.6
	12	18,290	18,450	36,740	27,100	39,800	66,900	40.5	27.8	32.9
Six	13	19,260	19,730	38,990	25,900	34,500	60,400	44.7	34.3	38.8
	14	19,280	18,890	38,170	26,400	39,700	66,100	43.7	28.5	34.6
Averages		18,720	18,850	37,570	26,800	34,200	61,000	41.9	33.1	36.9

[1] Data not complete.

8. The vehicle-hours of travel expended on the surface streets between 4 and 6 PM increased by 135 veh-hr, while the expressway travel decreased by 177 veh-hr. The vehicle-hours of travel on the total network remained essentially unchanged.

INITIAL EXPERIMENTATION WITH AUTOMATIC CONTROL

The knowledge gained from the initial experimentation with manual control and the results of a theoretical study of peak period control of an expressway system (2) clearly indicated the distinct advantage of ramp control utilizing traffic-adjusted metering as compared with time-clock complete ramp closure. Therefore, the next phase of ramp control was the development and evaluation of an automatic ramp metering device.

General Description

It is envisioned that the ramp metering scheme will require a metering device on the ramp and also changeable message displays at the entrance to the ramp and at locations some distance in advance of the ramp, so that intended ramp traffic could select parallel routes with little or no adverse distance. These changeable displays would indicate whether the ramp in question was open, metered, or closed (metering rate of zero). To confine this first step toward automatic control to the ramp metering device, a single on-ramp was selected on a portion of the study area where a frontage road existed (First Avenue on-ramp, see Fig. 8). By selecting this location, if the ramp is metered or closed, the intended ramp traffic could choose the frontage road as a parallel route, encounter no adverse distance, and postpone its entrance to the expressway to the Seventeenth Avenue on-ramp. The only advance display required would be at the entrance to the ramp, and it was decided that in this initial experimentation this display would be manually controlled.

Therefore, the primary purpose of this initial experimentation was to develop, test and finalize a ramp metering device. It was not intended at this stage to provide major expressway operational improvements except those which would result from improved merging operations at the First Avenue ramp entrance to the expressway. However, from previous studies it is clear that s successfully developed ramp metering device operating at the First Avenue ramp in conjunction with the next ramp upstream (Des Plaines Avenue ramp), would result in substantially improved expressway operations. Consequently, while this initial work is limited to developing the ramp metering device, the next logical steps would include developing advance displays and installing ramp metering devices at ramps further upstream, and then operational improvements on a continuous basis could be obtained. This series of developments is a necessary step toward the immediate objective of the Expressway Surveillance Project, which is the operation of an automatic pilot network information-control system.

Development of Automatic Ramp Metering Device

Three forms of metering devices were initially considered. The essential element of each was either movable gates, changeable message signs, or modified traffic signals. The movable gate had the distinct advantage of a physical barrier which is a desirable feature for ramp control. However, considerable sophisticated control appeared to be required during operation to insure against damaging vehicles or the gate itself. The use of a changeable message sign received attention, but disregard by some motorists for such control information in previous similar situations (3, 4) made this method less attractive. A modified traffic signal appeared most promising because of its simplicity and its previously proven effectiveness in controlling traffic. Therefore, the standard traffic signal should be modified for this application to convey to the motorist its unique application, and by so doing, enhance driver observance.

Before experience with the manual ramp metering, only a single detector located in advance of the ramp signal seemed to be required, and the delay until the next green signal indication could be controlled by a simple timer. In this way if the desirable ramp volume was 600 veh/hr, the timer would be set for 6 sec (3,600 sec/600 veh). However, experience with metering large trucks and concern about limiting the flow to one vehicle per green indication gave rise to the consideration of using two detectors. The detector in advance of the signal would indicate a ramp vehicle demand, and the second detector beyond the signal would indicate that a vehicle was in the process of passing or had passed. Another benefit of the two-detector system was that a light and bell alarm could be actuated if the vehicle violated the ramp signal indication (the situation when the ramp signal is red although a vehicle is detected entering the second detector zone).

Having decided to use the two-detector system in this initial installation, the exact locations for the detectors had to be selected. The first detector had to be close enough to the ramp signal so that a waiting vehicle would not be stopped between the detector and the ramp signal, and yet far enough away from the ramp signal so that the next ramp vehicle would not stop in advance of the detector. The second detector had to be close enough to the ramp signal so that a reasonably high metering rate could be maintained, while far enough away from the ramp signal so that a vehicle waiting for the green indication would not actuate the second detector. The locations for the two detectors in relationship to the ramp signal and stop line were established and are shown in Figure 18. The distance from the ramp signal to the ramp nose is 282 ft and the distance back to the frontage road is 158 ft.

The actual installation is shown in Figure 19. In the top photograph are the three advance signs. The first sign can be manually changed to read "Ramp Open Ahead," "Ramp Metered Ahead," or "Ramp Closed Ahead." The legend of the second sign on the left reads "Ramp Signal Ahead," while the third sign (located on right) indicates "Form Single Lane." The 3-lane one-way frontage road is shown in the top photograph. The lower photograph, a close-up showing the pavement markings, ramp signals, and controller, was taken during the first week of operation with a police officer present.

The basic steps in controlling the ramp metering device are shown in Figure 20. (The ramp metering device was designed by William L. Parker of the Expressway Surveillance Project and the hardware development was a joint venture with O. T. Gustus of Bell and Gustus.) In reference to Figure 21, measurement is obtained on the expressway and transmitted to the computer center for computation. The levels of the

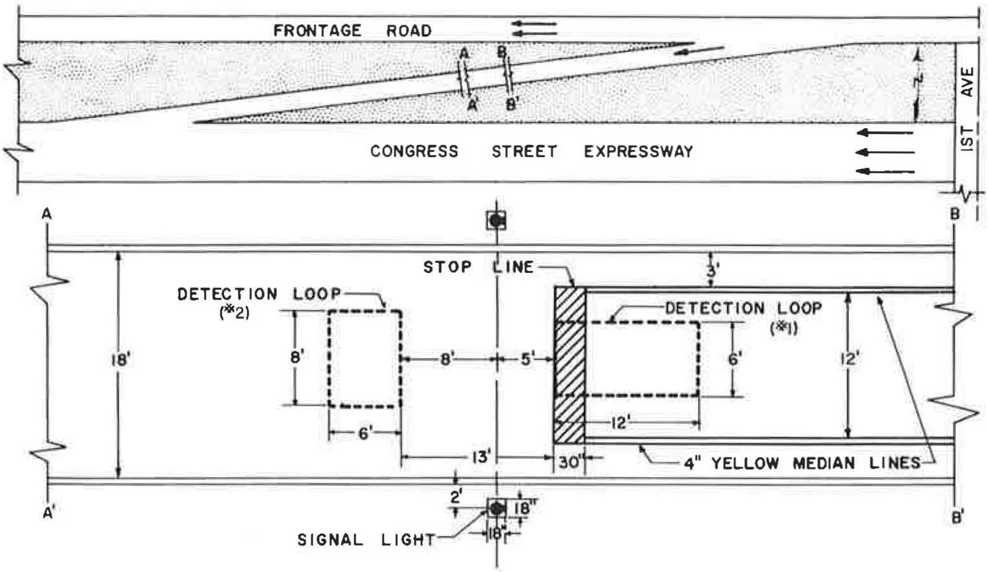

Figure 18. Ramp metering equipment.

measurement signifying free-flow conditions, impending congestion, or the presence of congestion are indicated by the level monitor. A control panel was constructed and connected for the transmission of control action on an automatic basis, but with a manual override. In the event that no control is required (free-flow conditions), the ramp signal rests in green. In the event of impending congestion, one of five metering rates is selected. The five metering rates are obtained by disengaging the timer and operating in a vehicle-actuated mode (basic metering), or by connecting the timer and pre-selecting four time delays (reduced metering). In the event of expressway congestion, the ramp is manually closed and when the vehicles already on the ramp are cleared, the ramp signal is locked in red.

Conduct of Control Experimentation

In late July 1963, the Project Advisory Committee approved the récommendation of the staff for automatic ramp metering control at First Avenue. The activities connected with the development and evaluation of the ramp metering device are shown in Figure 21. The design and construction of the ramp metering device were completed in August, and it was installed and tested during early September. The planning for the conduct of the evaluation was undertaken in August and early September. Publicity and handout of traffic bulletins at the on-ramp were accomplished during the second week of September. Operations with the ramp metering device began on Monday, September 16. To handle any difficulties that might arise and to encourage initial compliance, arrangements were made for a State Police officer to be present at the ramp during the initial week of operations. Starting on Monday, September 23, a police officer was not present except for normal routine police activities, and a preliminary evaluation of the performance of the metering device and associated equipment, and its effect on driver behavior and compliance began. In addition to this evaluation, schemes for metering based on mainline volume and occupancy combined, and mainline occupancy were tested and refined. The initial procedure used for metering operations was based on mainline occupancy, and the rules for control are, as follows:

1. Metering commences as soon as the mainline lane two occupancy equals or exceeds 15 percent provided it is after 3:30 PM and will begin regardless of percent occupancy at 3:50 PM.

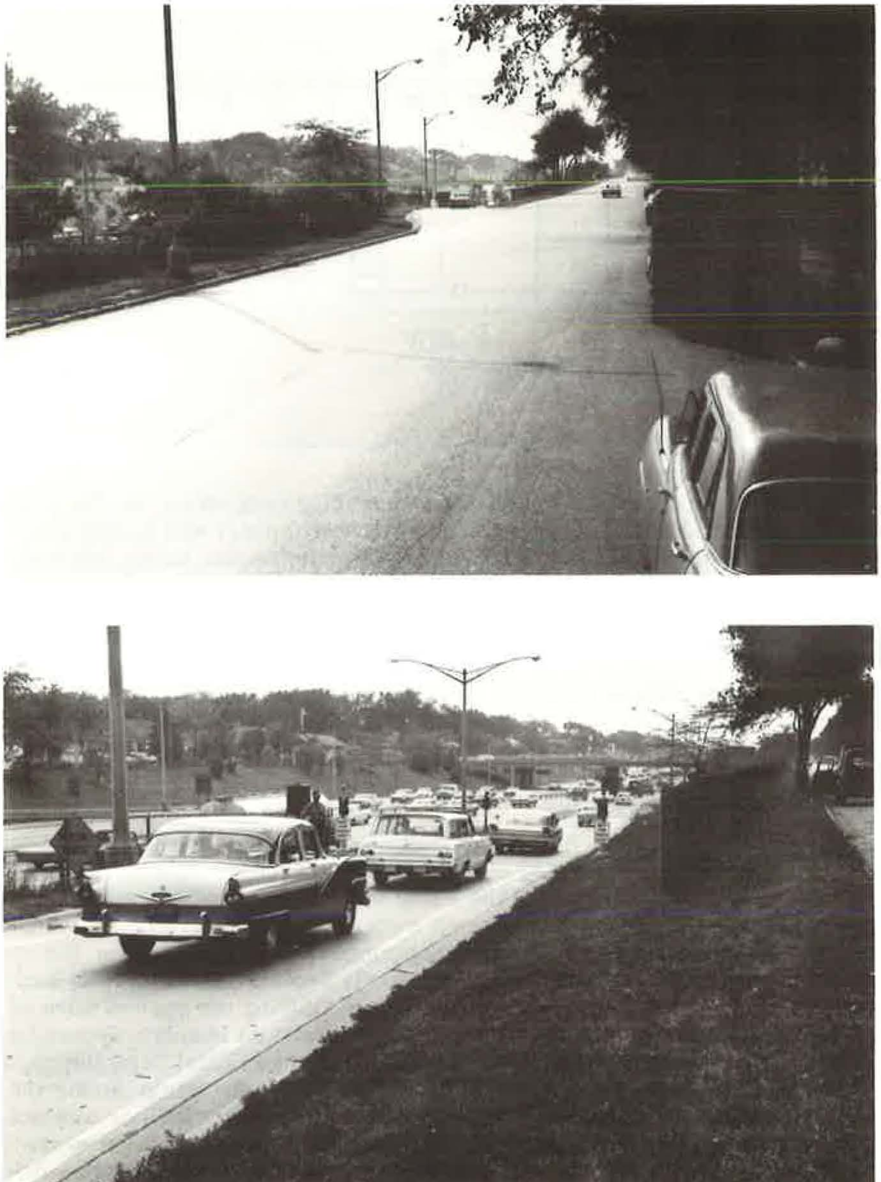

Figure 19. Ramp metering equipment.

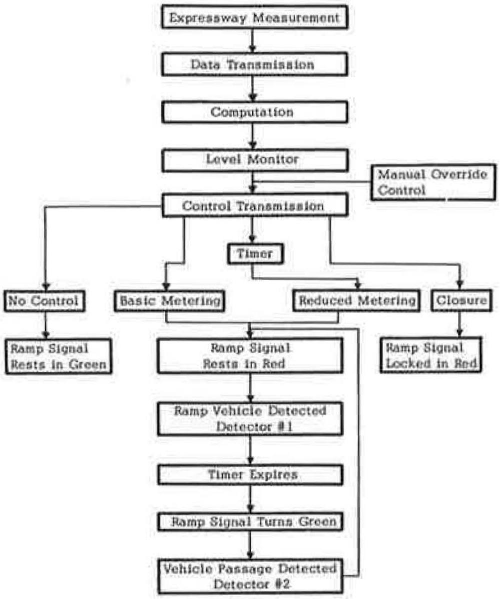

Figure 20. Ramp metering operation.

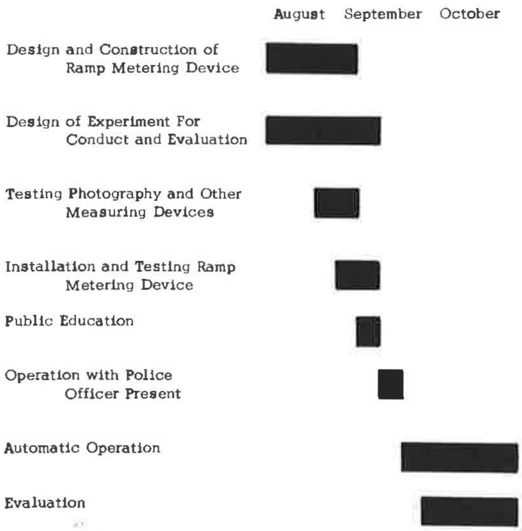

Figure 21. Study calendar.

TABLE 3

Percent Occupancy	Metering Rate Number	Clearance Delay[1] (sec)	Timer Setting (sec)	Total Clearance Time (sec)	Metering Rate (veh/min)
15	1	5	0	5	12
16	1	5	0	5	12
17	1	5	0	5	12
18	2	5	2	7	8.5
19	2	5	2	7	8.5
20	2	5	2	7	8.5
21	3	5	3	8	7.5
22	4	5	4	9	6.5
23	5	5	7	12	5.0
24	5	5	7	12	5.0

[1] Time required for driver of a ramp vehicle to see that signal has changed to green, accelerate, and clear second detector. Estimated clearance time of 5 sec is based on a starting delay of 2 sec; a detector configuration as shown in Figure 19; 85 percent of ramp vehicles being passenger vehicles, having an average length of 20 ft, an average acceleration rate of 10 ft/sec; and 15 percent of ramp vehicles being trucks, having an average length of 40 ft, an average acceleration rate of 6 ft/sec.

2. The metering rate will be automatically set based on percent occupancy obtained from an occupancy computer and level monitor. The permitted metering rate for various levels of occupancy is given in Table 3.

3. The ramp entrance to be closed as soon as the mainline lane two occupancy equals or exceeds 25 percent and provided it is after 3:30 PM. The queue already on the ramp to be discharged at metering rate number 5.

4. Metering operation will be terminated as soon as the mainline lane two occupancy is less than 15 percent and provided it is after 6:00 PM.

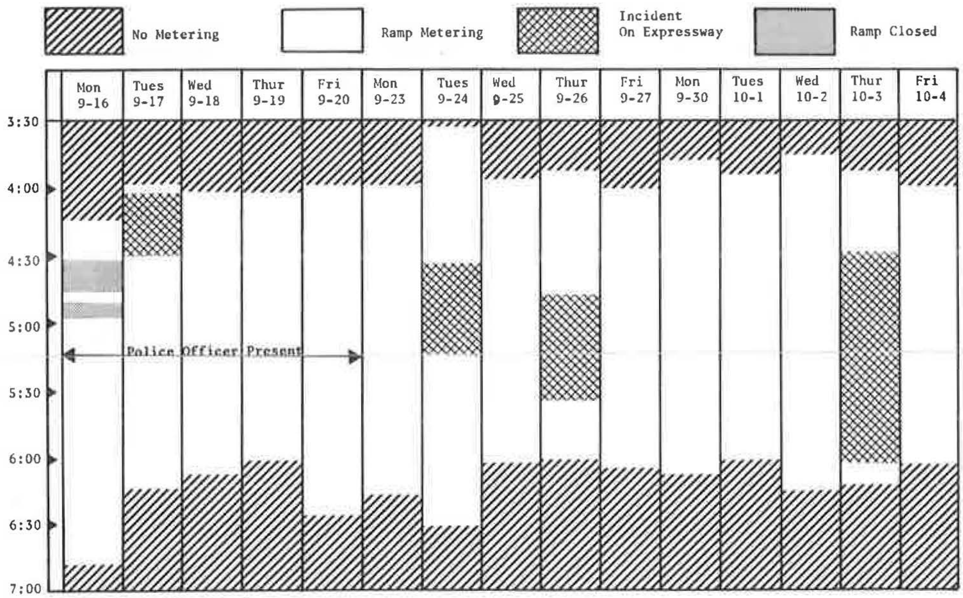

Figure 22. Degree of control and incidents during first three weeks of metering operations.

The period of time that the First Avenue on-ramp was controlled and a record of incidents on the expressway during the first three weeks of operations are shown in Figure 22. Metering normally began between 3:30 and 3:50 PM and ended between 6:00 and 6:20 PM. The weather was excellent during the 3-wk period and there was no precipitation during metering operation. Considerable congestion was encountered on the first day of operations and resulted in temporary ramp closures, and extended the period of metering. Apparently the public was interested in observing the ramp metering after being informed of the initial operations by radio and newspaper. A "gaper's block" occurred in both directions and was further affected by low-flying helicopters operated by local radio stations. Other incidents occurring on the expressway which affected the ramp control are also shown in Figure 22.

Measurements were taken on the expressway and at selected ramps using the pilot detection system. In addition, field observations were made at the ramp to record violations, hazardous maneuvers, queue lengths at the foot of the ramp and behind the ramp signal, and any malfunctions or difficulties with the metering device. These measurements permitted the preliminary evaluation of the initial automatic ramp metering operations at the First Avenue on-ramp.

Evaluation of Control Experimentation

Effect of Metering on Ramp Volume.—The average 4 to 6 PM accumulative ramp volume for a three-day period (Tuesday, Wednesday, and Thursday) before metering is compared with a similar three-day period during the first two weeks of metering in Figure 23. Before metering, the 2-hr volume was 810 vehicles, whereas with metering, the 2-hr volume was reduced to an average of 770 the first week and 780 the second week, or a reduction of 30 to 40 vehicles. Apparently during metering operations a small percent (4 to 5%) of the vehicles approaching the ramp and seeing a queue of vehicles, select the option of continuing west on the frontage road. The diversion is greatest between 4:30 and 4:45 PM when the queues on the ramp are generally the longest. The percent of traffic diverted was surprisingly low, particularly considering the attractive alternate route, and would indicate the acceptance of the metering device by a large proportion of the ramp traffic.

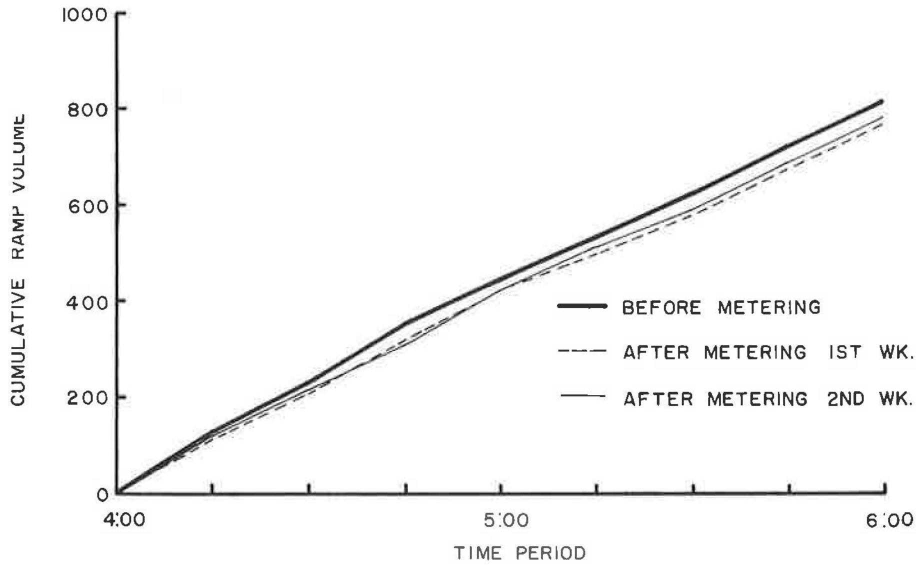

Figure 23. Cumulative ramp volume before and after metering.

Effect of Metering on Expressway and Merging Operations.—One measure of the effect of ramp metering on expressway operations is the percent occupancy level on the expressway just upstream of the metered ramp. Earlier studies had indicated that free-flow conditions existed when the percent occupancy was less than approximately 15 percent, and congested flow occurred when the percent occupancy was greater than 25 percent. Therefore the average minute percent occupancy in lane two at the First Avenue mainline station was recorded for the period of 4 to 6 PM for three weekdays (Tuesday, Wednesday, and Thrusday) before metering operations, and for the same time of day and for the same days of the week during metering operations. The cumulative frequencies of the average minute occupancies between 4 and 5 PM and between 5 and 6 PM are given in Table 4. The cumulative frequency data indicate that ramp metering had little effect either on the level or the distribution of average minute percent occupancies. As mentioned earlier and as confirmed by the tabulated values, expressway congestion did not occur at this location either before or during metering operations.

A measure of satisfactory merging is the absence of stopped ramp vehicles at the foot of the on-ramp. This is particularly critical at this on-ramp because of the short acceleration lane and the proximity of an off-ramp a short distance downstream. The number of stopped vehicles per 15-min period between 4 and 6 PM for two days before and after metering operations were recorded and are shown in Figure 24. During the 2-hr period the average number of vehicles stopped before metering and with metering was 110 and 92 vehicles, respectively. This was a reduction of 18 stopped vehicles or 16 percent. It appears possible that with further refinements in the metering technique the number of stopped vehicles can be reduced further. In addition, observation of the merging operations, a review of time lapse movies, and comments from a number of ramp users substantiated that merging operations had been improved by uniformly spacing vehicles and adjusting the space based on expressway conditions.

Effect of Ramp Metering Device on Ramp Traffic Behavior.—In addition to studying the merging operations and ramp volumes, measurements were taken of the length of queue behind the ramp metering device and of undesirable ramp maneuvers. The length of queue resulting from the ramp metering for each minute between 4 and 6 PM for three days is shown in Figure 25. The maximum queue length was 20 vehicles with the longest queues being observed from 4:00 to 4:10 PM and from 4:30 to 4:40 PM. Approximately 10 to 12 vehicles could be stored on the ramp and about the same number on the frontage road back to First Avenue.

After the first week of operations when the police officer was present, a record of

TABLE 4

CUMULATIVE FREQUENCY OF AVERAGE MINUTE OCCUPANCIES, FIRST AVENUE MAINLINE, LANE TWO

Percent Occupancy	Cumulative Frequency 4-5 PM Without Metering	With Metering	Difference	5-6 PM Without Metering	With Metering	Difference
7	0	0	-	0	0	-
8	1	0	+1	0	0	-
9	1	1	-	1	0	+1
10	2	1	+1	1	1	-
11	4	3	+1	1	1	-
12	12	7	+5	6	4	+2
13	21	16	+5	8	10	-2
14	35	30	+5	24	21	+3
15	51	46	+5	42	38	+4
16	67	65	+2	60	58	+2
17	79	80	-1	74	76	-2
18	87	91	-4	88	89	-1
19	95	96	-1	97	96	+1
20	97	99	-2	98	98	-
21	99	99	-	100	99	+1
22	99	99	-	100	100	-
23	99	99	-	100	100	-
24	100	99	+1	100	100	-
25	100	99	+1	100	100	-
26	100	99	+1	100	100	-
27	100	100	-	100	100	-

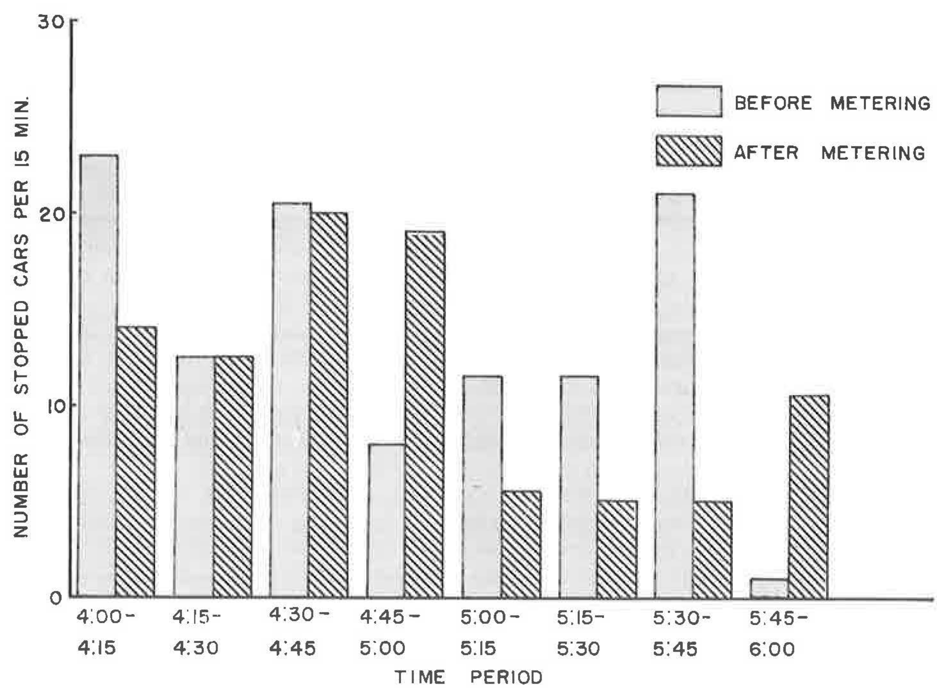

Figure 24. Frequency of stopped vehicles in the merging area.

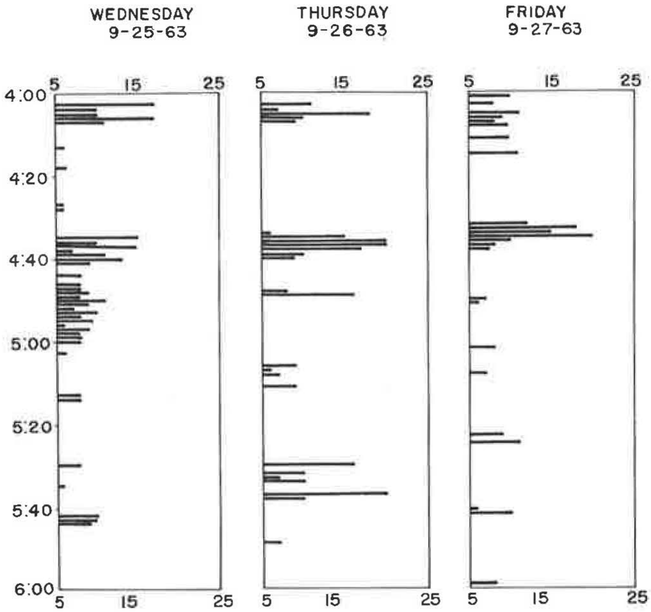

Figure 25. Length of queue due to ramp metering.

the number of vehicles proceeding through the red indication was kept for each afternoon period for three weeks. The number of violations varied from 14 to 32 vehicles per afternoon, with an average of 20 vehicles. The average percent of violations was 2.6 percent. Most of the violations occurred when a queue of vehicles was not present, and more than 90 percent were passenger vehicles. Apparently about one-third of these vehicles violated the ramp metering device unintentionally because when the alarm light and bell were actuated these vehicles either stopped or, in some cases, stopped and attempted to back up.

The most noticeable defect in the automatic ramp metering device has been the occasional situation when a ramp vehicle stops in advance of the first detector loop and is not detected by the system. When this happens the signal indication remains red and can only be changed to green by the stopped vehicle advancing to the first detector loop. It has been observed that either the driver of the vehicle releases his brake and finally advances far enough to be detected on his own, or the driver receives encouragement from the drivers of vehicles stopped behind him. Although this situation is not too common (an average of six such occurrences per afternoon), it does cause an unnecessary increase in the queue length and total delays.

Two minor modifications have been made in an attempt to eliminate this difficulty. First, the informational message on the two signal pedestals has been changed from "Wait for Green Light" to "Wait for Green at Line." Second, the stop line has been moved 5 ft downstream to the location of the ramp signals. Studies are now under way to evaluate the effectiveness of these measures.

Conclusions.—The following are the conclusions drawn from the initial experimentation with automatic ramp control:

1. The utilization of a modified traffic signal appears to be satisfactory for metering ramp traffic. A large proportion of the ramp traffic (over 95%) operates in a satisfactory manner and complies with the regulations pertaining to the ramp metering operations.
2. The most noticeable defect of the ramp metering operation has been vehicles stopping in advance of the first detector and consequently not receiving a green indication. Minor modifications have been made to handle this situation.
3. The percent of traffic selecting the frontage road in preference to the metered

TABLE 5

Mainline Occupancy %	Metering Rate (veh/min)	
	Initial	Revised
15	12	9.5
16	12	9.5
17	12	9.5
18	8.5	9.5
19	8.5	7.2
20	8.5	7.2
21	7.5	7.2
22	6.5	5.3
23	5.0	5.3
24	5.0	3.9

TABLE 6

Upstream Expressway Vol. (veh/min)	Metering Rate (veh/min)[1]
86	14
86-91	9.5
91-93	7.2
93-95	5.3
95	3.9

[1] When lane 2 occupancy exceeds 22%, a metering rate of 3.9 veh/min is used.

ramp was surprisingly low, and would indicate the acceptance of the metering device by a large proportion of the ramp traffic.

4. The traffic operations in the freeway-ramp merging area were improved by ramp metering as indicated by a reduction in the number of stopped ramp vehicles in the merging area.

5. The queue of vehicles formed by the ramp signal were normally less than ten vehicles in length except for the two brief periods immediately after 4:00 and 4:30 PM when queue lengths varied between 10 and 20 vehicles.

6. The number of vehicles which violated the ramp signal varied from 14 to 32 vehicles per afternoon, with an average of 20 vehicles (2.6% of ramp traffic).

7. The relationship between mainline occupancy and the permitted metering rate was modified to reduce the number of stopped vehicles in the merging area. The initial and revised metering schemes are given in Table 5.

8. Metering schemes using mainline occupancy and volume combined have been tested and a revised metering scheme is given in Table 6.

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Influence of Off-Ramp Spacing on Traffic Flow Characteristics on Atlanta Freeway and Arterial Street System

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The spacing of interchanges on freeways can have a pronounced effect on the operating efficiency of a freeway in a downtown area and can therefore affect the entire transportation system within a city. Proper spacing of interchanges on freeways is also important from a cost and a vehicle-time viewpoint. The purpose of this study was to determine the influence of off-ramp spacing on the operational characteristics of the Atlanta Freeway System and the city streets influenced by the Freeway.

The southbound off-ramps at 14th St., 10th St., and North Ave. were closed for a period of two weeks each during the morning peak traffic period. During these closures and during normal operations, time-lapse movie photography was used to collect data at four locations along the Freeway. Speed and delay studies were made on the freeways and surface streets during each ramp closure and during normal operation of the Freeway.

Analysis of variance techniques were used to make comparisons of volumes, speeds, densities, over-all travel, and over-all travel time data observed on all the streets and freeways within the system.

It was found that closing any one of the southbound off-ramps on the North Freeway during the morning peak hour caused little or no improvement in the over-all operating characteristics of the Freeway. When a significant change was noted, it was usually a reduction in the quality of traffic flow.

As a result of this study, it was found that off-ramps leading to the central business district of a city should be spaced as close together as possible, consistent with design factors and the ability of the surface streets in the vicinity of the off-ramp to accommodate the traffic flow from the ramp.

•THE PURPOSE of this research was to determine the influence of off-ramp spacing on the traffic flow characteristics on the Atlanta Freeway System and city streets influenced by the Freeway. This is the second phase of a study sponsored by the Georgia State Highway Department in cooperation with the U. S. Bureau of Public Roads under a contract with the Georgia Institute of Technology.

The first phase was concerned with the influence of on-ramp spacing on the traffic flow characteristics on the Atlanta Freeway System and city streets influenced by the Freeway.

In the first phase it was found that lengthening the spacing by closing any one of the northbound on-ramps (by closing an on-ramp) on the North Freeway during the afternoon peak hour improved the over-all operating characteristics of the Freeway. This improvement was shown by a smoother and more uniform flow of traffic as reflected by the speeds and densities observed on the Freeway. It was also found that increasing the spacing of on-ramps caused a significant increase in the total travel time in vehicle-minutes within the entire system. The over-all travel in vehicle-miles was not changed significantly by closing any one of the northbound on-ramps within the system.

Paper sponsored by Committee on Freeway Operations.

DATA COLLECTION

The study area (Fig. 1) for this phase of the research lies north of the central business district of Atlanta. It was chosen because any alterations or changes in ramp spacing would influence traffic flow conditions in this area. Changes in traffic flow outside this area would be rather small and the methods used to detect changes in conditions would not be sensitive enough to measure this change.

The land use in this area consists mainly of small business, apartment houses, boarding houses, insurance offices, small and medium size hotels, service stations, old residences, and schools. The majority of traffic in the study area during the morning peak hour is traffic which is destined to or is passing through rather than originating in the area.

The street system consists of three east-west arterials and five arterials running generally in a north-south direction. The Freeway runs generally north-south near the center of the study area. The southbound off-ramps exit from the North Freeway at 14th St., 10th St., North Ave., and Williams St.

Within the study area, the Freeway consists of three 12-ft lanes in each direction between the south limits of the study area and the junction of the Northwest and Northeast Freeways. From this junction north, to both the Northwest and Northeast Freeways, it consists of two 12-ft lanes in each direction. The Freeways and ramps have a concrete surface, and the concrete on the ramps has been darkened to provide color contrast.

The interchanges on the North Freeway at 14th St., 10th St., and North Ave. are of diamond-type design with the off-ramps exiting directly from the Freeway with no deceleration lanes provided. The Williams St. off-ramp exits directly from the Freeway onto Williams St., again with no deceleration lane provided on the Freeway. Fifth St. is a grade separation only, not an interchange.

The design speed used on the Freeway was 50 mph and the design speed used on the ramps was 35 mph. The design hour volume used on the Freeway was 1,500 vehicles per lane per hour. The maximum horizontal curve used was 3 deg and the maximum vertical grade used was 5 percent.

The method used for varying the ramp spacing was to close certain ramps during the peak period of flow. This method allowed the ramp spacing to be altered without making permanent or semi-permanent changes in the Freeway and ramps. When a ramp was closed, the interchange was effectively eliminated from the system and the ramp spacing of the "remaining interchanges" was changed. The distances between interchanges are given in Table 1. Each of the southbound off-ramps located at 14th St., 10th St., and North Ave. was closed during the morning peak peak period between 7:00 and 9:00 AM for a period of two weeks. The ramps were closed on the following dates: 14th St.—April 2-6, 9-13, 1962 (Monday through Friday of each week); 10th St.—April 23-27, April 30-May 4, 1962 (Monday through Friday of each week); and

TABLE 1
INTERCHANGE DISTANCES

Interchange	Distance (mi)			
	Normal Operation[1]	Southbound Off-Ramp Closed at		
		North Ave.	10th St.	14th St.
Peachtree Rd.				
	0.891	0.891	0.891	
14th St.				1.250
	0.359	0.359		
10th St.			1.067	
	0.708	0.997		0.708
North Ave.				
	0.189		0.189	0.189
Williams St.				

[1] All ramps open.

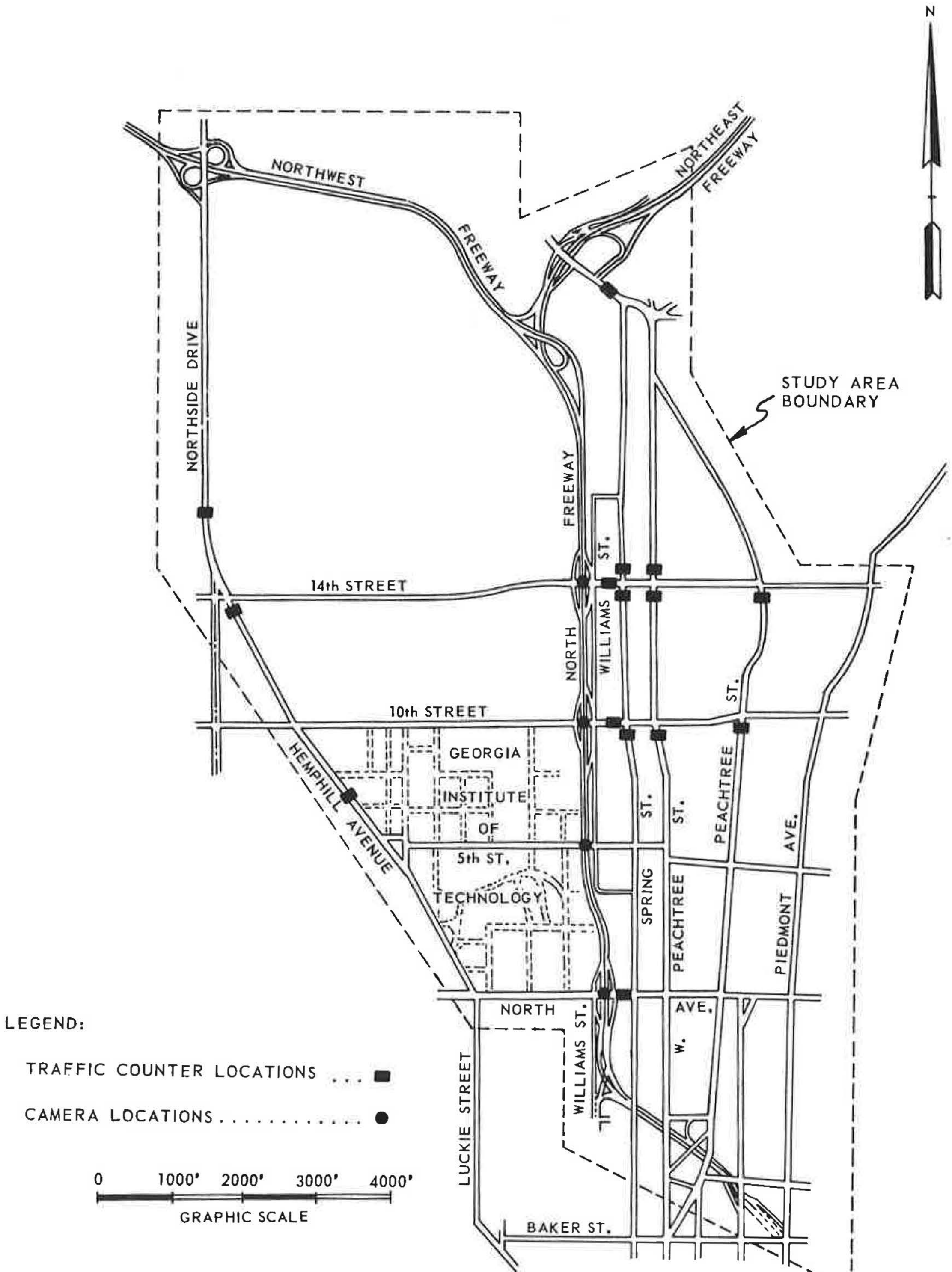

Figure 1. Study area.

Figure 2. Method of closing North Ave. southbound off-ramp.

North Ave.—May 14-18, 21-25, 1962 (Monday through Friday of each week). Each off-ramp was closed separately and only one ramp was closed at any one time. Table 1 gives the distance between interchanges for each ramp condition. Figure 2 shows a typical method of closing an on-ramp.

Most of the studies were made during the second week that each ramp was closed. The first week that each ramp was closed the traffic within the study area was allowed to stabilize to permit motorists to establish new patterns of travel. It was assumed that the motorists would distribute themselves throughout the study area in such a way as to optimize their travel before the beginning of the second week.

Studies were made at four different locations on the North Freeway under normal operation of the Freeway, and with the southbound off-ramp closed at 14th St., 10th St., and North Ave. The data collected at these locations were volume, speed, and density in each lane on the North Freeway and the ramp volume. These data were collected between 7:20 and 8:00 AM and between 8:10 and 8:50 AM by the use of time-lapse movie photography. The cameras used to make the movies would hold only enough film for 40 min of continuous filming when exposed at the filming rate of 100 frames per min. All data collected during these two periods were summarized in 5-min time increments.

Fifteen mechanical volume counters were placed on the arterial streets throughout the study area to determine how the travel patterns changed when a ramp was closed. Figure 1 shows the locations of these counters. The counters were placed such that only the southbound and eastbound traffic was counted on the north-south and east-west arterials, respectively. The assumption was made that the traffic flow in opposite directions would not be significantly affected by the ramp closures.

The travel time on all freeways and arterials within the study area was measured by the use of speed and delay studies. The data which were obtained from each speed and delay run were over-all travel time, total running time (time that vehicle was actually moving), total delay time and cause of each delay, over-all travel speed, and running speed. These data were summarized between major intersections and for the total trip.

The total vehicle-minutes of travel time were computed for all vehicles in the study area during each ramp condition on the Freeway by using the data obtained with the speed and delay studies and volume counts.

Time-lapse motion picture photography was used to collect data at the four study locations on the Freeway. Figure 3 shows a typical field setup with a camera on a bridge passing over the North Freeway at the study location, together with stripes of alternate dark and light color painted on the Freeway at 50-ft intervals.

The film was analyzed by projecting it with a time and motion study projector onto a screen having a grid which matched the lines painted on the pavement at each study

Figure 3. Field location of time-lapse movie camera at 10th St.

location. Vehicular speeds were obtained by measuring the distance of movement of a vehicle for a specified number of movie frames and dividing this distance by the time required for filming that specified number of movie frames.

Vehicle volumes and vehicle speeds per lane were collected from the movie film. From this data, the densities in each lane were computed. These data were summarized in 5-min increments of time. By the use of the final counter on the projector, each 5-min increment could be determined by counting each 500-frame interval. Volume data were also obtained for each off-ramp.

Average speed for each lane and for each 5-min period was determined by taking a systematic sample of 20 vehicles per time period. Statistical analysis based on sample survey theory indicated that a sample of this size was sufficient to give the true average speed within 2 mph 95 percent of the time.

ANALYSIS OF DATA

To analyze the data which were obtained it was necessary to take the raw data, such as vehicle volume and vehicle speeds, from the film and refine these data to a usable form. The film was divided into 5-min time intervals for each of the four study locations. The following information was evaluated from the time-lapse movie film: (a) lane volumes (before ramp entrance), (b) lane speeds, (c) lane densities (before ramp entrance), (d) average lane volumes (before ramp entrance), (e) average lane speeds (before ramp entrance), (f) average lane densities (before ramp entrance), (g) ramp volume, and (h) total volume.

The major street system and the Freeway in the study area were divided into links to determine the total over-all travel time in vehicle minutes and total travel in vehicle-

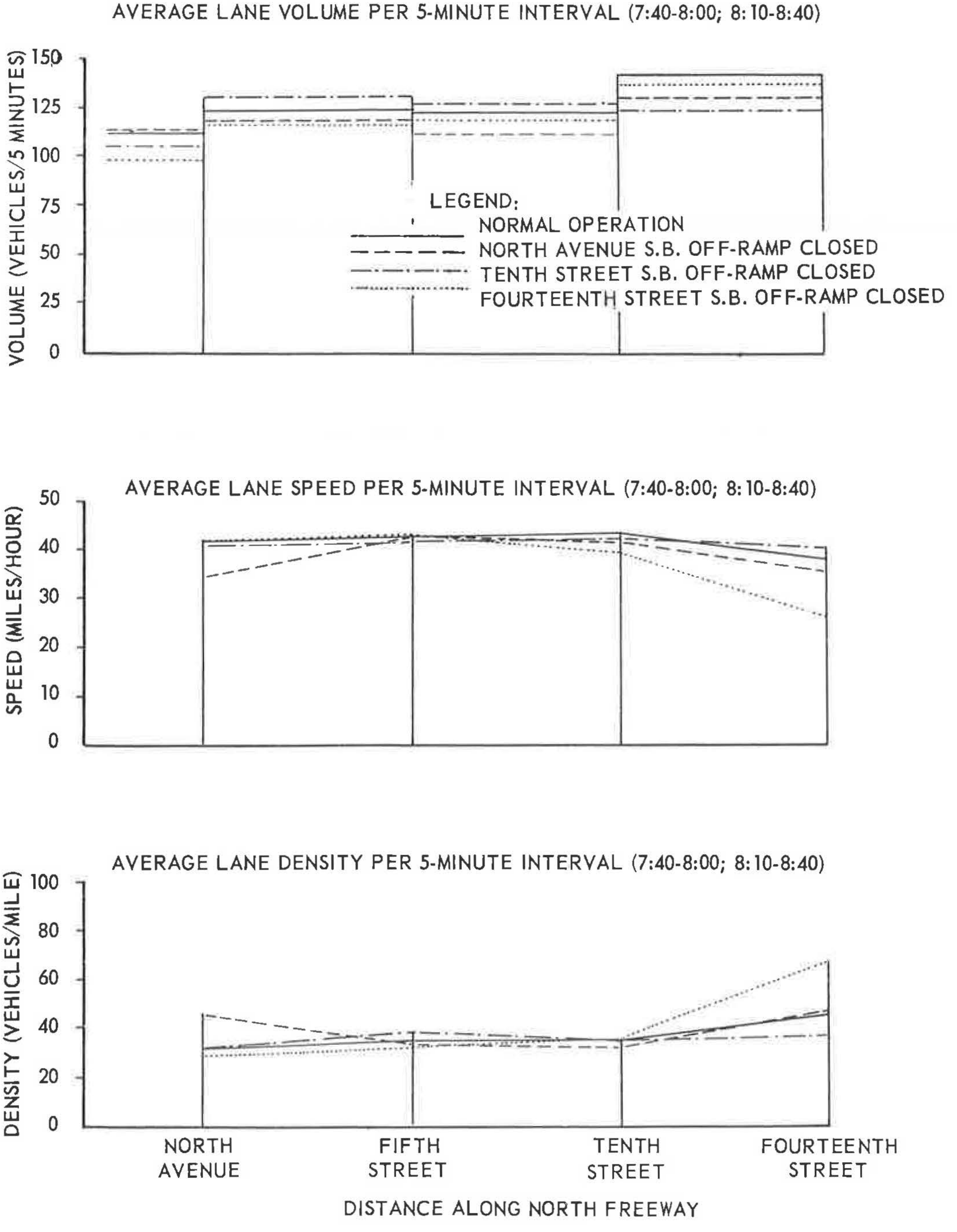

Figure 4. Average volume, speed, and density per 5-min interval on North Freeway.

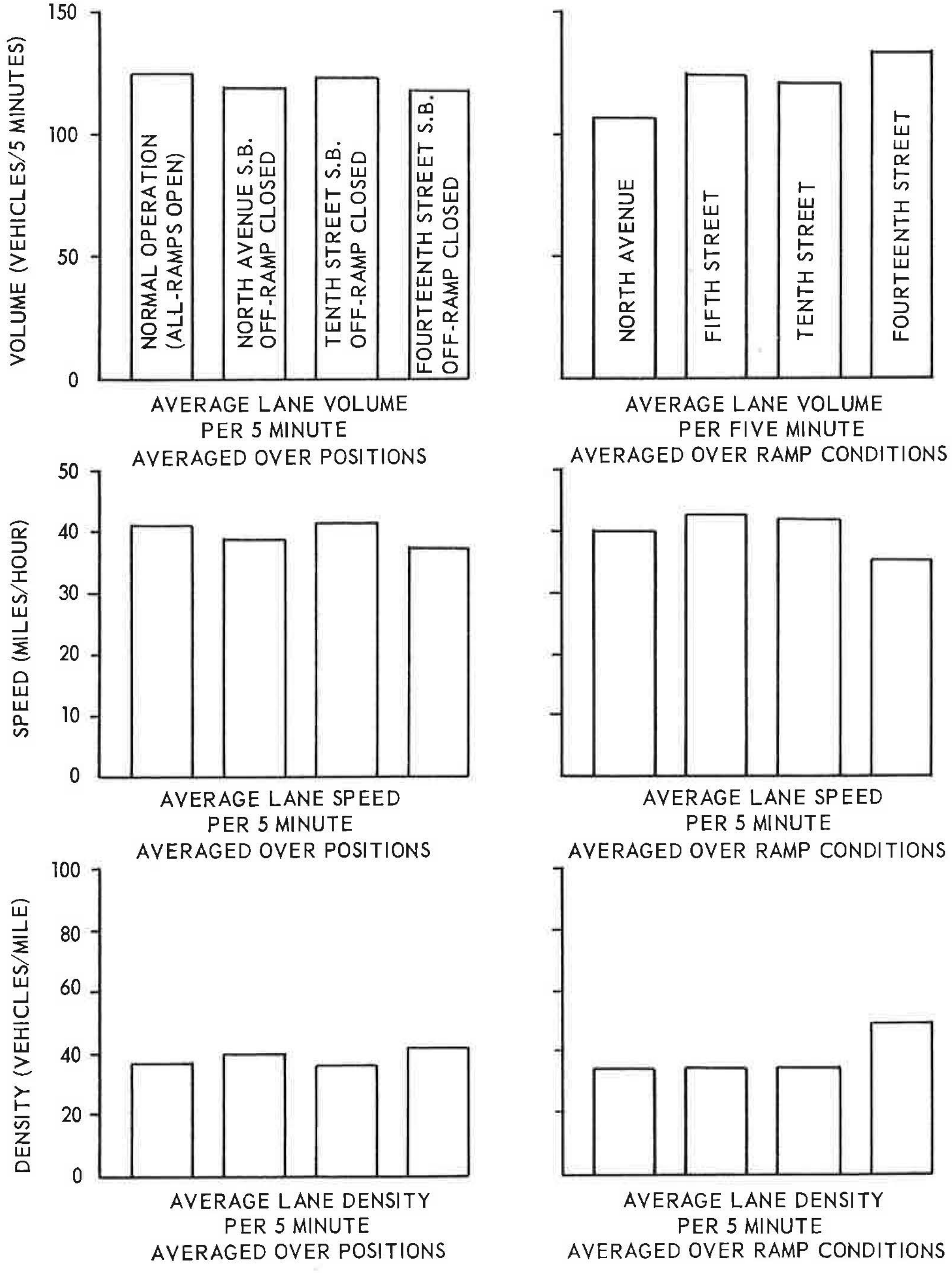

Figure 5. Average North Freeway traffic characteristics for 5-min interval.

TABLE 2

RANK ORDER OF FREEWAY VOLUMES AND SIGNIFICANT DIFFERENCES OF STUDY LOCATIONS[1]

Ramp Condition	Rank Order of Volume			
	Lowest	2nd Lowest	2nd Highest	Highest
Normal operation	5th St.	North Ave.	14th St.	10th St.
14th St. closed	North Ave.	5th St.	10th St.	14th St.
10th St. closed	North Ave.	10th St.	5th St.	14th St.
North Ave. closed	North Ave.	5th St.	14th St.	10th St.

[1]No significant differences in factors underlined together.

TABLE 3

RANK ORDER OF FREEWAY VOLUMES AND SIGNIFICANT DIFFERENCES IN RAMP CONDITIONS[1]

Study Location	Rank Order of Volume with Ramp Closed at			
	Lowest	2nd Lowest	2nd Highest	Highest
North Ave.	10th St.	14th St.	North Ave.	Normal
5th St.	14th St.	North Ave.	Normal	10th St.
10th St.	10th St.	North Ave.	14th St.	Normal
14th St.	14th St.	Normal	10th St.	North Ave.

[1]No significant differences in factors underlined together.

TABLE 4

RANK ORDER OF FREEWAY SPEEDS AND SIGNIFICANT DIFFERENCES OF RAMP CONDITIONS[1]

Study Location	Rank Order of Speed with Ramp Closed at			
	Lowest	2nd Lowest	2nd Highest	Highest
14th St.	14th St.	North Ave.	Normal	10th St.
10th St.	14th St.	North Ave.	10th St.	Normal
5th St.	10th St.	North Ave.	Normal	14th St.
North Ave.	North Ave.	10th St.	14th St.	Normal

[1]No significant differences in factors underlined together.

TABLE 5

RANK ORDER OF FREEWAY SPEEDS AND SIGNIFICANT DIFFERENCES OF STUDY LOCATIONS[1]

Ramp Condition	Rank Order of Speed			
	Lowest	2nd Lowest	2nd Highest	Highest
Normal operation	14th St.	North Ave.	5th St.	10th St.
14th St. closed	14th St.	10th St.	North Ave.	5th St.
10th St. closed	North Ave.	14th St.	5th St.	10th St.
North Ave. closed	North Ave.	14th St.	10th St.	5th St.

[1]No significant differences in factors underlined together.

miles which accrued during the peak hour. Each of these links consisted of a portion of a street between two other major streets or a portion of the Freeway between interchanges. The total number of vehicle-minutes was computed for each link by multiplying the vehicle volume by the respective travel times. The total travel time on each of the arterial streets, Freeways and ramps was obtained from the calculations of the link times.

Analysis of variance methods were used to analyze the data. Mathematical models were formulated in terms of the unknown parameters and the associated random variables. The dependent variables of interest in this study consist of the following: (a) volume, (b) speed, (c) density, (d) total travel time in system (expressed in vehicle-minutes), (e) total travel distance in system (expressed in vehicle-miles), (f) over-all running speed, and (g) over-all travel speed. The independent variables of interest in this study consist of the following: (a) ramp condition (ramp open or closed), (b) position (location of ramp along Freeway), (c) lane number, (d) day, (e) street, and (f) replication (replication is each 5-min time interval). The 10 percent level of significance was used for testing the variables in the analysis of the variance of the data.

FREEWAY VOLUMES

Figures 4 and 5 show that the volume decreases in the direction of travel along the Freeway. Figure 5 also shows that the highest volumes occur when the Freeway is operating normally with all ramps open.

Table 2 gives the rank order and significant differences of ramp conditions at each study location and Table 3 gives the rank order and significant differences of the volumes at study locations under each ramp condition. These data are represented graphically in Figure 4. The locations are underscored where the differences in volumes are not significant, according to Duncan's "Multiple Range and Multiple F Tests" (5).

The highest volumes generally occurred on the North Freeway when it was operating normally. The volumes observed under normal operations were significantly different from those observed when any off-ramp was closed. The volumes observed when any of the off-ramps were closed were not significant from each other, which is to say that no matter which off-ramp was closed, the volumes were significantly less than those which occurred under normal operation.

The traffic volumes decreased in magnitude as one moved southward along the North Freeway from 14th St. to North Ave., although the volumes observed at 10th St. and at 5th St. were not significantly different from each other.

FREEWAY SPEEDS

Figures 4 and 5 show that there were insignificant differences in the speeds at any one of the study locations or under any of the ramp conditions except at 14th St. with the 14th St. off-ramp closed and at North Ave. with the North Ave. off-ramp closed.

Table 4 gives the rank order and significant differences of ramp conditions at each location and Table 5 gives the rank order of speeds and significant differences of locations. These data are also shown graphically in Figure 4. The factors are again underscored where the differences are not significant.

The speed generally increased on the Freeway as one moved southbound along the North Freeway. The speeds observed at 14th St. were significantly lowest. The speeds at the other positions were not significantly different from each other.

When averaged over all the locations, the speeds were significantly lowest when the 14th St. off-ramp was closed. The highest speeds occurred when the 10th St. off-ramp was closed, but these speeds were not significantly different from those observed under normal operation of the Freeway.

FREEWAY DENSITIES

Figures 4 and 5 show that the density generally decreased as the volume decreased in the direction of travel southward along the Freeway. It can be noted from Figure 4

TABLE 6

RANK ORDER OF FREEWAY DENSITIES AND SIGNIFICANT DIFFERENCES OF RAMP CONDITIONS[1]

Study Location	Rank Order of Density with Ramp Closed at			
	Lowest	2nd Lowest	2nd Highest	Highest
14th St.	10th St.	North Ave.	Normal	14th St.
10th St.	No. Ave.	10th St.	Normal	14th St.
5th St.	14th St.	North Ave.	Normal	10th St.
North Ave.	14th St.	Normal	10th St.	North Ave.

[1]No significant differences in factors underlined together.

TABLE 7

RANK ORDER OF FREEWAY DENSITIES AND SIGNIFICANT DIFFERENCES OF STUDY LOCATIONS[1]

Ramp Condition	Rank Order of Density			
	Lowest	2nd Lowest	2nd Highest	Highest
Normal operation	North Ave.	5th St.	10th St.	14th St.
14th St. closed	5th St.	North Ave.	10th St.	14th St.
10th St. closed	10th St.	14th St.	5th St.	No. Ave.
North Ave. closed	10th St.	5th St.	North Ave.	14th St.

[1]No significant differences in factors underlined together.

that the density was least uniform and tended to be highest when the 14th St. and North Ave. off-ramps were closed. It may also be seen that the most uniform densities occurred when the Freeway was operating normally.

Table 6 gives the rank order of densities and significant differences of ramp conditions and Table 7 gives the rank order of densities and significant differences of study locations. Again the factors are underscored when the differences are not significant.

SPEED AND DELAY

The speed and delay studies which were made on the Freeway verified the results obtained from the analysis of the movie films. The speed and delay studies were analyzed in several different ways. The over-all travel and running speeds were analyzed on the North Freeway, Northeast Freeway, Northwest Freeway, North Northeast Freeways combined and the North and Northwest Freeways combined.

The over-all speeds and running speeds tended to be lowest on the North Freeway when the North Ave. off-ramp was closed, but speeds under this condition were not significantly different than when the 14th St. off-ramp was closed. The highest speeds occurred under normal operation, but these speeds were not significantly different from the speeds occurring when the 10th St. or the 14th St. off-ramps were closed.

When the North Freeway was considered in conjunction with either the Northeast or Northwest Freeways, the speeds were significantly lowest when the North Ave. off-ramp was closed and were significantly highest under normal operation.

The speed and delay studies which were made on the surface streets were analyzed in two groups. The east-west streets were analyzed together and the north-south streets were analyzed together.

The analysis of variance investigation indicated there were no significant differences in the over-all travel speeds or running speeds on the east-west streets. Similar results were indicated for the north-south streets.

SURFACE STREET VOLUMES

The volumes which were observed on the north-south streets and the east-west

TABLE 8

RANK ORDER OF TOTAL OVER-ALL TRAVEL TIME AND SIGNIFICANT DIFFERENCES OF RAMP CONDITIONS[1]

Location	Rank Order of Total Over-all Travel With Ramp Closed at			
	Lowest	2nd Lowest	2nd Highest	Highest
North-South St.	Normal	North Ave.	10th St.	14th St.
East-West St.	Normal	14th St.	North Ave.	10th St.
North Freeway	10th St.	Normal	14th St.	North Ave.
Northeast Freeway	10th St.	Normal	North Ave.	14th St.
Northwest Freeway	14th St.	Normal	North Ave.	10th St.
Freeway and Sts.	Normal	14th St.	10th St.	North Ave.

[1] No significant differences in factors underlined together.

TABLE 9

TOTAL OVER-ALL TRAVEL TIME

Facility	Total Over-all Travel Time (veh-min)			
	Normal Operation	14th St. Ramp Closed	10th St. Ramp Closed	North Ave. Ramp Closed
Streets	34,836	39,408	41,117	37,275
Freeways and ramps	30,711	33,277	32,767	47,663
Total	65,547	72,685	73,884	84,938

streets were analyzed in a similar manner as were the speed and delay studies. The volumes which occurred between 7:00 and 8:00 AM were analyzed separately from the volumes which occurred between 8:00 and 9:00 AM.

The surface street volumes tended to be lowest when the Freeway was operating under normal conditions. But, the analysis of variance investigation indicated no significant differences among the ramp conditions for any of the groups of surface street volumes analyzed.

TOTAL OVER-ALL TRAVEL TIME

There were no significant differences in the total over-all travel time on the east-west streets under any of the ramp conditions. There were also no significant differences noted in total over-all travel time on the north-south streets under any of the ramp conditions.

The over-all travel time on the North Freeway was significantly highest when the North Ave. off-ramp was closed. There were no significant differences among the other ramp conditions for over-all travel time on the North Freeway.

The over-all travel times on the Northeast Freeway and the Northwest Freeway were not significantly different for any of the ramp conditions.

The total over-all travel time in the entire system of freeways and surface streets considered together was significantly highest when the North Ave. off-ramp was closed. There were no significant differences noted under any of the other ramp conditions. The travel times tended to be lowest under normal operation of the Freeway, though not significantly so.

Table 8 gives the rank order of total over-all travel times and significant differences of ramp conditions, and Table 9 gives a summary of the observed total over-all travel times.

TOTAL TRAVEL

The total travel on the freeways and streets expressed in vehicle-miles was analyzed

TABLE 10

RANK ORDER OF TOTAL OVER-ALL TRAVEL AND SIGNIFICANT DIFFERENCES OF RAMP CONDITIONS[1]

Location	Rank Order of Total Over-all Travel With Ramp Closed at			
	Lowest	2nd Lowest	2nd Highest	Highest
North-south	Normal	North Ave.	10th St.	14th St.
East-west	North Ave.	Normal	14th St.	10th St.
North Freeway	14th St.	North Ave.	10th St.	Normal
Northwest Freeway	14th St.	10th St.	Normal	North Ave.
Northeast Freeway	10th St.	14th St.	North Ave.	Normal
Freeway and Sts.	14th St.	Normal	North Ave.	10th St.

[1] No significant differences in factors underlined together.

TABLE 11

TOTAL OVER-ALL TRAVEL

Facility	Normal Operation	14th St. Ramp Closed	10th St. Ramp Closed	North Ave. Ramp Closed
Streets	9,821	10,731	10,560	10,316
Freeways and ramps	15,285	14,190	14,761	14,812
Total	25,106	24,921	25,321	25,128

in the same manner as were the speed and delay studies and the total over-all travel time.

The travel on the north-south streets was found to be significantly lowest when the Freeway was operating normally. Travel on the north-south streets was significantly higher when a ramp was closed on the Freeway, and one ramp closure was not significantly different from another.

Travel on the east-west streets was not significantly different under any ramp condition.

Travel on the North Freeway was significantly highest when the Freeway was operating normally. The closing of any one ramp was not significantly different from the closing of any other ramp. The results obtained for the Northwest Freeway were similar to that of the North Freeway.

Travel on the Northeast Freeway was not significantly different under any ramp condition.

Table 10 gives the rank order of total over-all travel and significant differences of ramp conditions, and Table 11 gives a summary of the observed total over-all travel time.

Closing any off-ramp significantly increased the travel on the north-south streets while significantly decreasing the travel on the North and Northwest Freeways. Furthermore, the closing of an off-ramp had no significant effect on the travel on the east-west streets, Northwest Freeway, or the system as a whole. Apparently the decrease in travel on some streets balanced the increase on the other streets causing no significant differences in the vehicle-miles traveled in the entire study area.

SUMMARY OF RESULTS

A summary of the results of the analysis of the data collected in this study can be outlined as follows:

1. Averaged over-all positions, the highest volumes occurred on the North Freeway when the Freeway was operating normally, and these volumes were significantly dif-

ferent from those observed when any of the ramps were closed. The volumes obtained when any of the off-ramps were closed were not significantly different from each other.

2. The traffic volumes decreased as one moved southward along the North Freeway from 14th St. to North Ave. The volumes observed at 10th St. and 5th St. were not significantly different from each other.

3. The speed on the North Freeway increased significantly between 14th St. and 10th St., when considering all ramp conditions, but there were not any significant differences in the speeds observed at the other study positions.

4. Averaged over-all positions, speeds were significantly lowest when the 14th St. off-ramp was closed. The highest speeds occurred when the 10th St. off-ramp was closed and under normal operation, and were not significantly different from each other.

5. When averaged over-all of the ramp conditions, density was significantly highest at 14th St. The densities observed at each of the other study positions were not significantly different from each other.

6. The significantly highest densities occurred at all study positions when the 14th St. off-ramp was closed.

7. Closing any one of the off-ramps did not significantly affect the density on the North Freeway at a study location prior to ramp that was closed.

8. Closing any one of the off-ramps on the Freeway tended to reduce the volume on the Freeway, and closing the 14th St. off-ramp or the North Ave. off-ramp tended to decrease the speeds and thus increase the densities as well.

9. The total over-all travel time on the Freeway was significantly greatest when the North Ave. off-ramp was closed. The travel time which occurred under the other conditions were not significantly different.

10. The total over-all travel time for the Freeway and streets was significantly highest when the North Ave. off-ramp was closed. The total travel time for other ramp conditions was not significantly different from each other.

11. The total travel on the Freeway was lowest when the 14th St. off-ramp was closed, and was highest when the Freeway was operating normally.

12. The total travel on the Freeway and streets was not significantly different for any ramp condition.

CONCLUSIONS

The following conclusions were determined after an evaluation of the results of analysis of data:

1. Studying the volume, speed, and density on a freeway at several different locations simultaneously with variable ramp spacings will give a more reliable indication of the actual traffic flow characteristics which exist on the freeway than will a point study.

2. Closing any one of the southbound off-ramps during the morning peak hour caused little or no improvement in the operating characteristics of the North Freeway. When a significant change was noted, it was usually a reduction in the quality of traffic flow.

3. Of all the ramp closings, the closing of the 10th St. southbound off-ramp caused the least change in the operating characteristics of the North Freeway.

4. The total over-all vehicle-minutes of travel time used by all the vehicles traveling through the freeway system is an effective measure of the level of service existing in a freeway system.

5. The relatively high density which was observed at North Ave. when the North Ave. southbound off-ramp was closed was apparently caused by the close proximity of the next exit, Williams St.

RECOMMENDATIONS

The following recommendations were made as a result of this study:

1. All off-ramps in the study area should remain open during the morning peak hour to provide optimum operation of the freeway and city streets.

2. For a freeway to efficiently handle the morning peak hour traffic flow into the central city, the off-ramps should be spaced as close together as possible, consistent

with the factors of design of the freeway and the ability of the surface street system in the vicinity of the off-ramp to accommodate the traffic from the ramp.

ACKNOWLEDGMENT

The authors thank the Georgia State Highway Department and the U. S. Bureau of Public Roads who sponsored this research on the operation of the Atlanta Freeway System. The authors also thank the driving public in the City of Atlanta, who directly or indirectly participated in this research, for their tolerant cooperation during the ramp closing experiments.

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An Analysis of Random Freeway Traffic Accidents and Vehicle Disabilities

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•THE John C. Lodge Freeway Surveillance Project is being conducted on the John C. Lodge Freeway in Detroit, Mich., between the Davison and Edsel Ford Freeway Interchanges. The project is being conducted jointly by the Michigan State Highway Department, the City of Detroit Streets and Traffic Department, Wayne County Road Commission, in cooperation with the U. S. Bureau of Public Roads.

The study section is 3.2 mi, with television surveillance being accomplished by 14 remotely controlled television cameras. This section has such geometric features as portions of 6- and 8-lane divided, 9 on- and 9 off-ramps, a reverse curve and grades. It has carried as many as 160,000 vehicles per day for both directions. The lane and speed signal controls have been in operation for 1½ yr. and the television system for 3 yr.

Figure 1 indicates the study section's special design features, location of cameras, lane signals, speed signs, and ramp signals.

Trained observers are on duty for a 14-hr period from 6:00 AM to 8:00 PM daily, except weekends and holidays. A general log (see Appendix) is maintained as a permanent record of all vehicular incidents including accidents, vehicle disabilities, maintenance operations, and others such as motorists aiding distressed vehicles. This log contains all the data for this study and analysis of freeway incidents.

PURPOSE

The purpose of this study is to determine the frequency, duration, and character of random freeway traffic incidents, and also to investigate the factors influencing their occurrence.

STUDY DATA

The study data, taken from the general logs, covered the period from June 1, 1962 to June 1, 1963 (255 surveillance days).

Information contained in the general log includes type of accident, time of occurrence, duration, location, lanes affected, assistance, weather, pavement condition, and other pertinent remarks.

This study is limited to random vehicular incidents occurring on the freeway proper and categorized as (a) accidents, and (b) vehicle disabilities. Excluded are (a) maintenance operations, and (b) all other non-random incidents such as vehicles stopping to assist distressed vehicles. All shoulder uses were deleted as these constitute a separate study and the first report on this subject has been published ("Shoulder Usage on an Urban Freeway," January 1962).

ANALYSIS OF DATA

Data were reduced to a presentation of total incidents, accidents, and vehicle disabilities; and analyzed as to location (as indicated by camera field), weather conditions, pavement surface temperature, aid received, lane in which incident occurred, occur-

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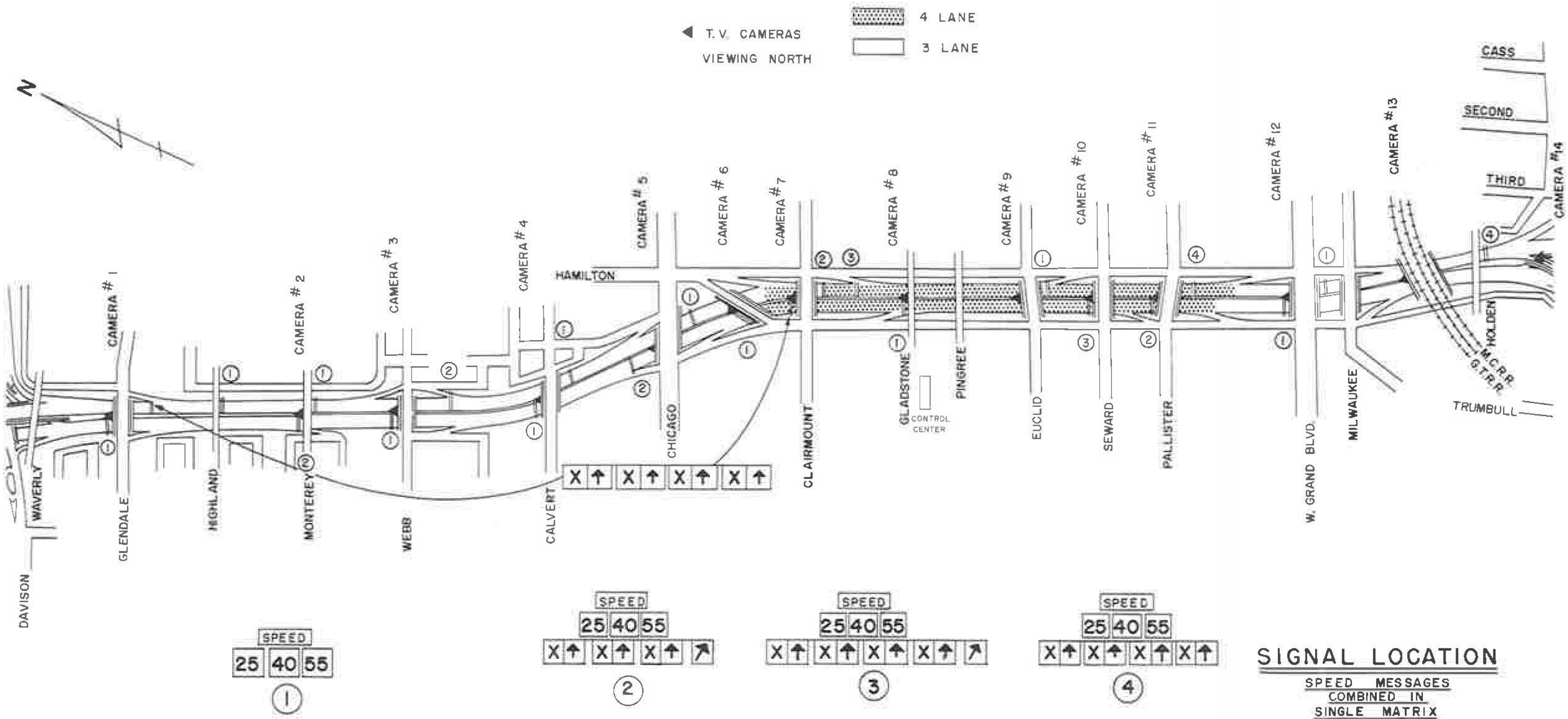


Figure 1. Freeway traffic surveillance and control research project, John C. Lodge Freeway, Detroit, Mich.

rence related to geometric design on 4-lane vs 3-lane sections, day of week, month of year, and time of day. Vehicle disabilities included stalls, flat tires, spin-outs (vehicle spinning on slippery pavement, no collision occurring), and others.

Vehicular Incidents by Location

Figure 2 shows the number of incidents by camera field location and by direction, respectively. Certain camera fields cover 4-lane and 3-lane, respectively, in both directions of travel. The camera fields 7, 8, 9, 10, 11, and 14 cover the 4-lane section inbound, whereas camera fields 8, 9, 10, 11, and 14 cover the 4-lane sections outbound. The greatest number of incidents occurred in camera field 9, which covers a 4-lane section. In the outbound direction, camera field 9 is located in advance of the point where the four lanes converge into three lanes. This factor is responsible for daily congestion and undoubtedly contributes to the occurrence of incidents in this area.

To compare incidents by camera fields, it was decided to determine the number of incidents per $\frac{1}{8}$ mi of camera field. This distance was selected as it is approximately the length of the shortest field. Figure 3 shows total incidents and incidents by direction per $\frac{1}{8}$ mi of camera field. Again, the 4-lane sections revealed a higher rate of incidents than the 3-lane section.

A more detailed analysis of the incidents in the 4- and 3-lane sections provides a better basis for comparison. Table 1 gives the number of vehicle incidents by lane on the 4- and 3-lane sections of the freeway. A greater number of incidents—both accidents and vehicle disabilities—occurred in the 3-lane section. For the 255 surveillance days, there was a daily average of 3.64 incidents (0.90 accidents and 2.74 vehicle disabilities). Of the 3.64 incidents, 2.11 occurred in the 3-lane section (0.54 accidents and 1.57 vehicle disabilities); 1.53 occurred in the 4-lane section (0.36 accidents and 1.17 vehicle disabilities).

To effect proper comparison of 4- vs 3-lane sections, the difference in length must be considered. Table 1 gives the incident rate per mile of section. The daily average of incidents for the 3-lane section was 0.51 (0.13 accidents and 0.38 vehicle disabilities) per mile; for the 4-lane section 0.69 (0.16 accidents and 0.53 vehicle disabilities) per mile. Again, the rate of incidents in the 4-lane section is higher. Comparison also can be made on a per lane basis in the respective sections.

Another basis of comparison for 4- vs 3-lane section can be made from Table 1 which gives the total incidents per million vehicle-miles.

On the basis of a daily volume of 116,000 vehicles for the 255 surveillance days, Table 1 indicates there were 3.2 more incidents per million vehicle-miles in the 4-lane section—an increase of 36 percent over the 3-lane section. The difference is reflected primarily in vehicle disabilities. For the total 3.2 mi of study section, the rates per million vehicle-miles are as follows: accidents = 2.4, vehicle disabilities = 7.4; total incidents = 9.8 per million vehicle-miles.

Vehicular Incidents by Time of Day, Direction of Travel, and Day of Week

Figures 4, presents total and directional occurrence of incidents by time of day and follows approximately the same pattern as do the daily volume distributions for this section of freeway.

Comparable to the volume pattern, the occurrence of incidents reached a peak inbound between 7:30 and 9:30 AM and reached an even higher peak outbound between 3:30 and 6 PM. There was a difference of only 11 incidents—inbound, 458 incidents; outbound, 469 incidents. It is evident, and expected, that the probability of an incident occurring increased as traffic exceeded the maximum density of the freeway.

Table 2 is a comparison of the daily averages of vehicular incidents by day of week. Analysis shows no significant influence of any particular day although Friday exhibited a deviation. The higher total rate may be attributable to the higher traffic volumes and prolonged periods of congestion on Fridays.

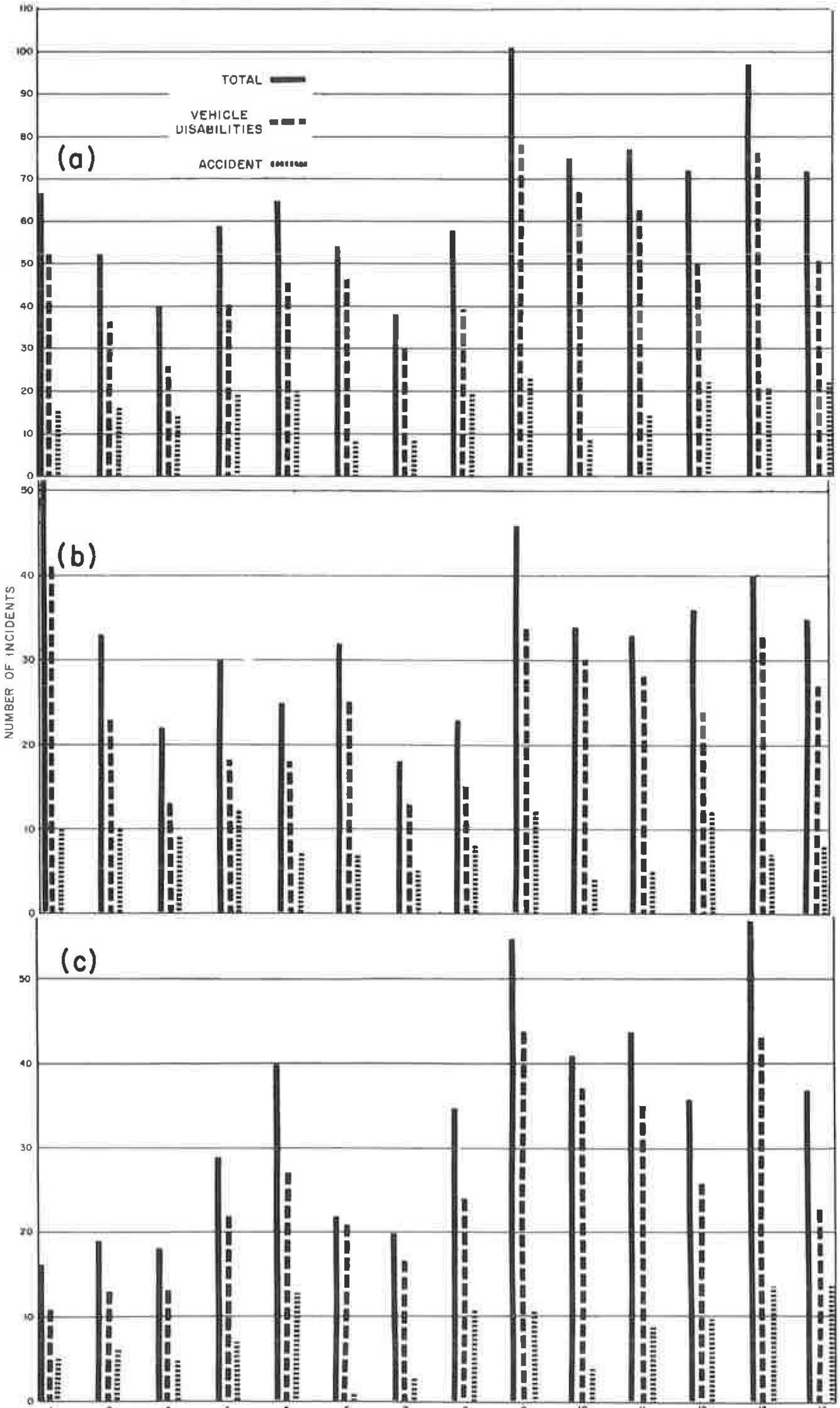


Figure 2. Incidents by camera field: (a) total, (b) inbound, and (c) outbound.

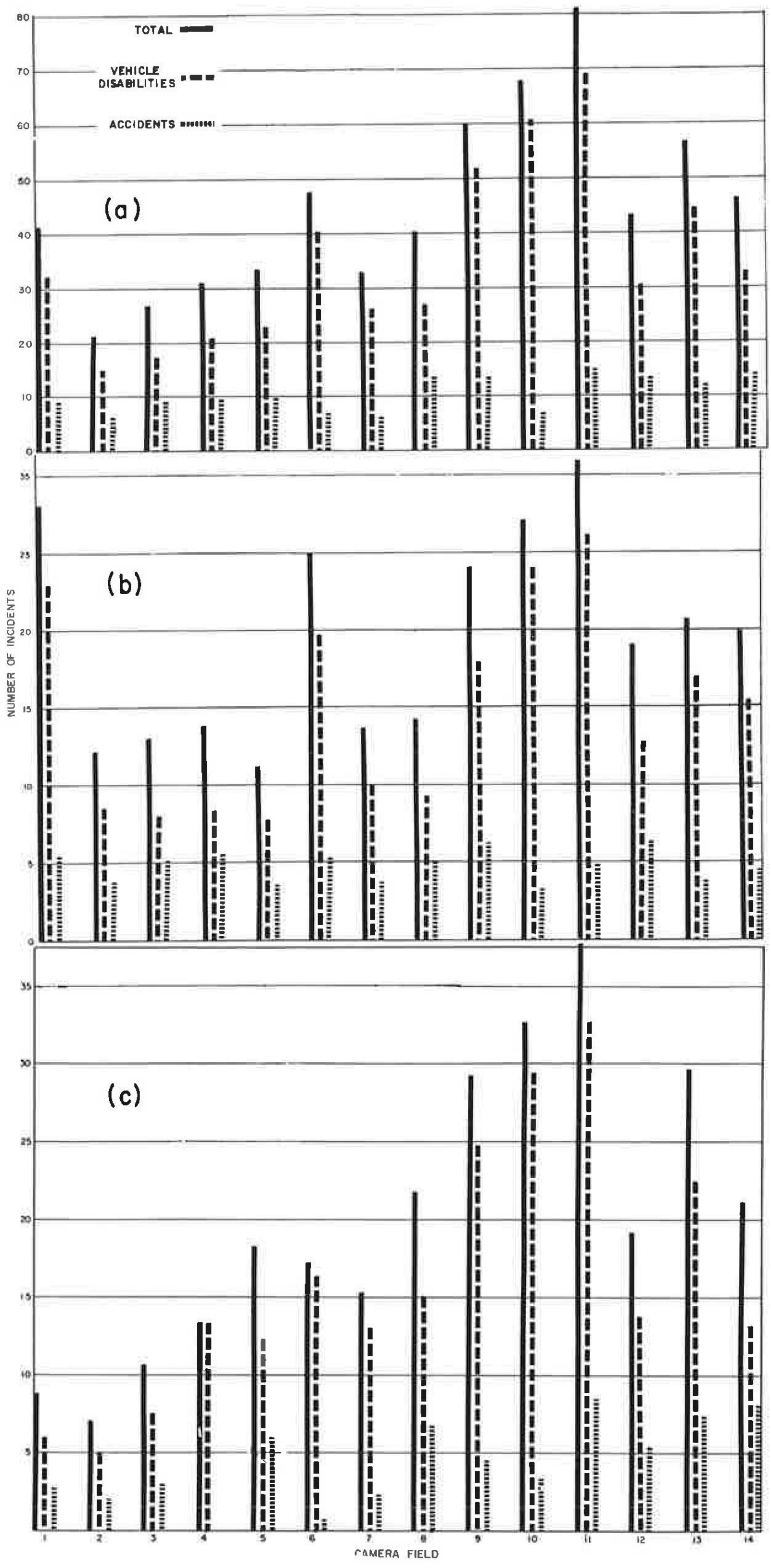


Figure 3. Incidents per $^1/_8$ mi of camera field: (a) total, (b) inbound, and (c) outbound.

Figure 4. By time of day: (a) total incidents,

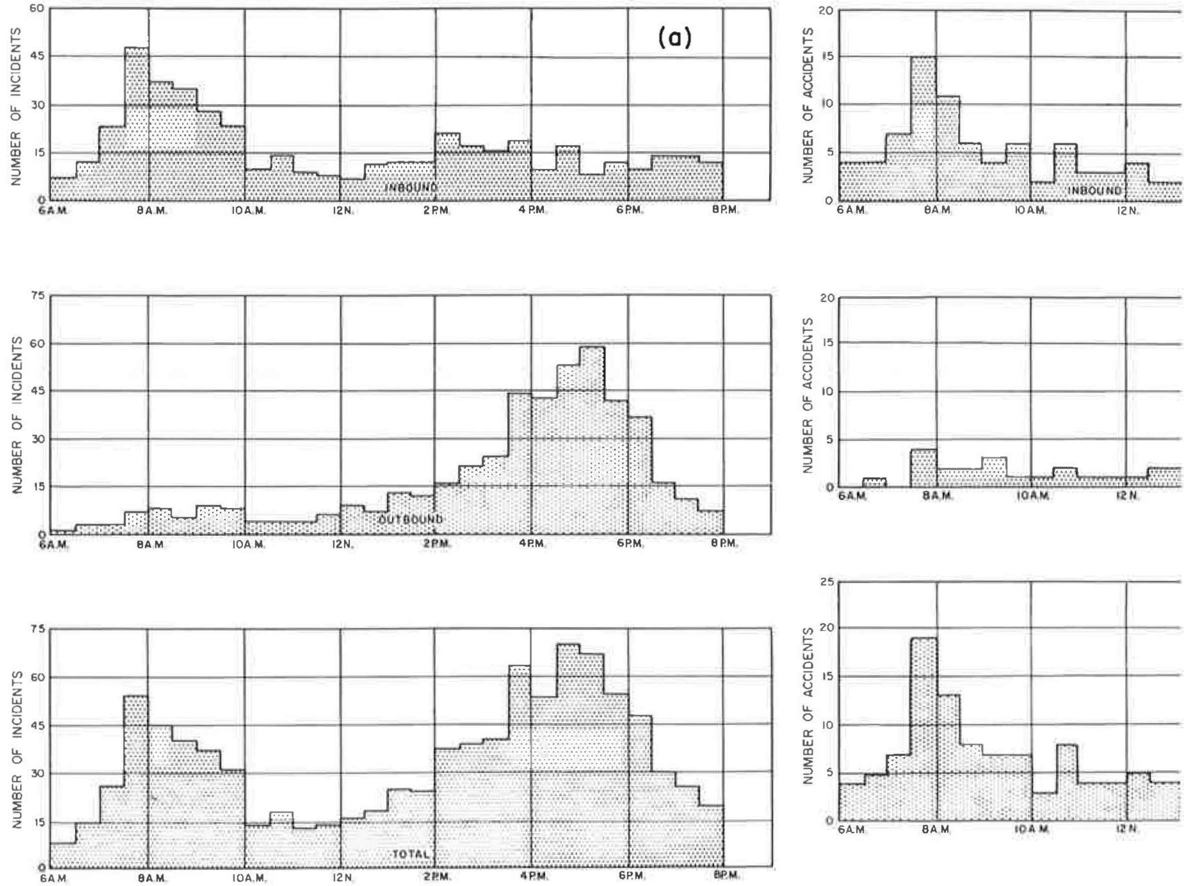


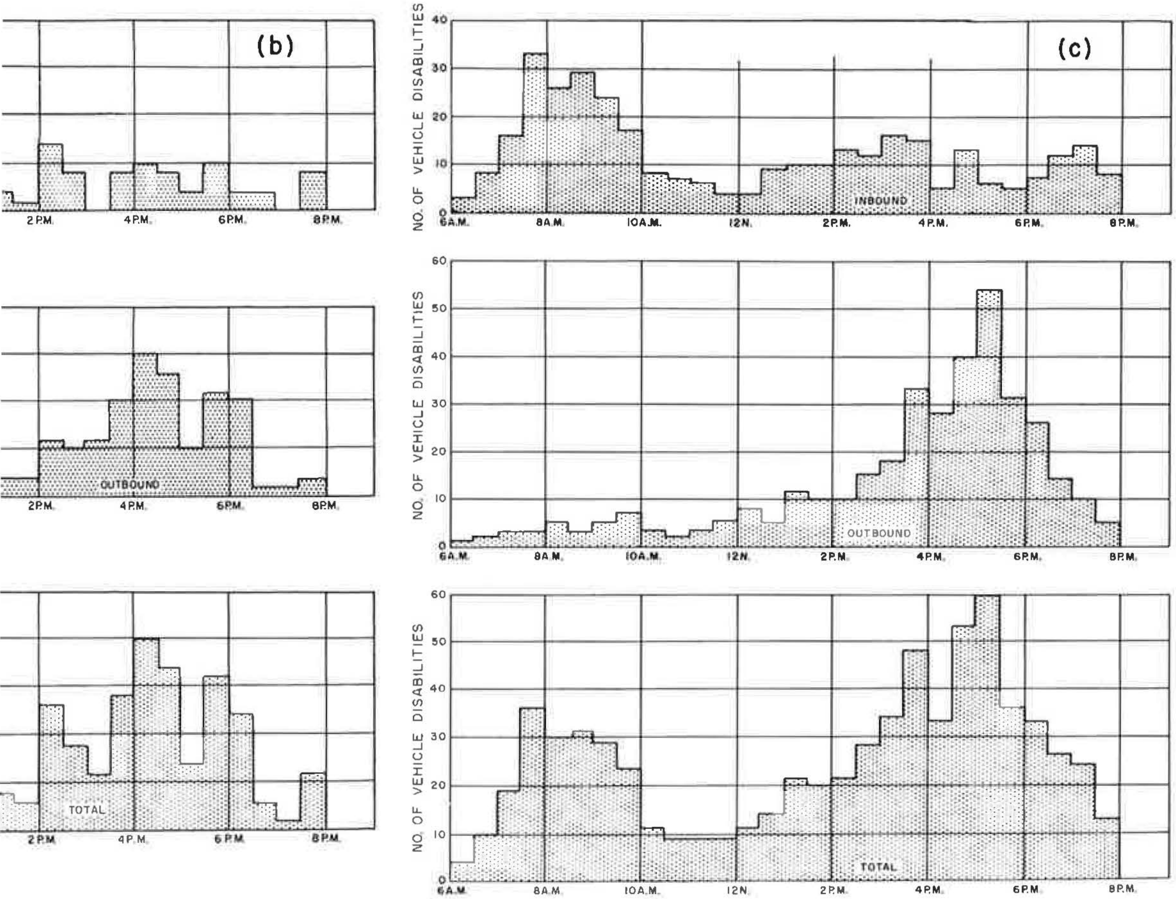
TABLE 1

VEHICULAR INCIDENTS[a]

Lane	Accidents			Vehicle Disabilities			Total Incidents		
	3-Lane Section[b]	4-Lane Section[c]	Total	3-Lane Section[b]	4-Lane Section[c]	Total	3-Lane Section[b]	4-Lane Section[c]	Total
(a) Total, by Lane									
1	45	36	81	209	134	343	254	170	424
2	35	23	58	89	61	150	124	84	208
3	53	18	71	96	45	141	149	63	212
4	—	6	6	—	52	52	—	58	58
Multiple	4	9	13	9	3	12	13	12	25
Total	137	92	229	403	295	698	540	387	927
(b) By Lane, per Mile									
1	10.7	16.4	12.7	49.8	60.9	53.6	60.5	77.3	66.3
2	8.3	10.5	9.1	21.7	27.7	23.4	29.5	38.2	32.5
3	12.6	8.2	11.1	22.9	20.5	22.0	35.5	28.6	33.1
4	—	2.7	2.7	—	23.6	23.6	—	26.4	26.4
Multiple	1.0	4.1	2.0	2.1	1.4	1.9	3.1	5.5	3.9
Total	32.6	41.9	35.8	96.0	134.1	109.1	128.6	176.0	144.8
(c) Total, per Million Vehicle-Miles									
—	2.2	2.8		6.5	9.1		0.7	11.9	

[a]Total for both directions. [b]21.2 mi both directions. [c]4.2 mi both directions.

(b) accidents, and (c) vehicle disabilities.



Vehicular Incidents Related to Climatic Conditions

Climatic conditions, especially seasonal variations, contribute to the cause of incidents and influence the frequency. Figure 5 shows the vehicular incidents by month of year. The greatest number of incidents occurred during the months when driving conditions were made more hazardous by low temperatures or sudden changes in the weather.

To compare frequency of incidents according to climatic and pavement conditions, and temperature, climatic data were obtained from the U. S. Weather Bureau publications "Local Climatological Data" for the study year. Applicable data are given in Table 3. Weather and temperature data were obtained from the U. S. Weather Bureau. Pavement conditions and durations were obtained from the general log.

Vehicular incidents, by types, were compared by pavement, weather, and temperature conditions. The daily average of vehicular incidents, by climatic condition, was computed by dividing the total number of incidents per climatic condition by the number of equivalent days of each condition. The day referred to is the 14-hr surveillance day. Table 4

TABLE 2

AVERAGE NUMBER OF INCIDENTS PER DAY BY DAY OF WEEK

Day	Accidents	Vehicle Disabilities	Total
Monday	0.74	2.37	3.11
Tuesday	0.90	2.66	3.56
Wednesday	0.82	2.77	3.59
Thursday	1.08	2.52	3.60
Friday	0.96	3.28	4.24
Daily Avg.	0.90	2.74	3.64

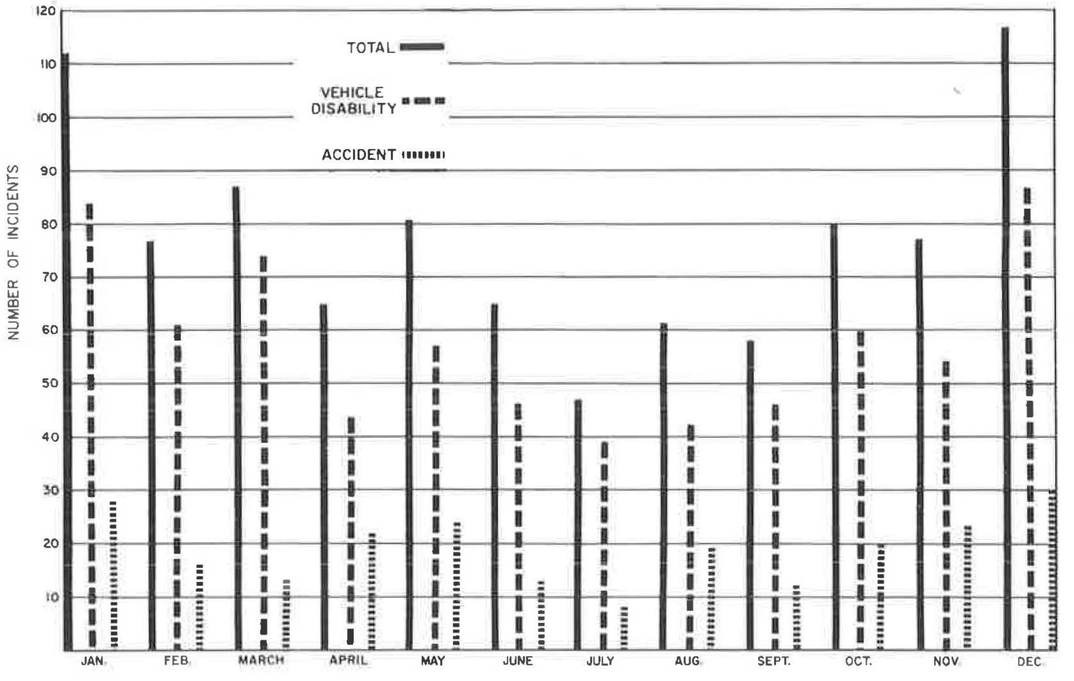


Figure 5. Incidents by month.

gives the comparison of total incidents by climatic conditions. Table 4 shows that the potential frequency of incidents is much greater during rain or snow, slushy and wet pavement conditions, as well as during colder temperature.

Table 5 gives the frequency of accidents under various conditions of pavement, weather, and temperature.

Table 6 gives the comparison of vehicle disabilities by climatic conditions. Vehicle disabilities include stalls, flats, spin-outs, etc.

In the tables, for purposes of comparison only, a rate of incidents per hour and per day was calculated for the entire study section and then determined for 1 mi of study section. As these incidents covered both directions of the freeway, the total length applicable to these incidents was 6.4 mi, thus considering these incidents as occurring on two roadways of 3.2 mi each.

The tables reveal the obvious effect of snow, wet and/or slushy pavement, and low temperatures by an increased frequency of stalled vehicles and accidents. These increases were all statistically significant. It was determined, however, that the frequency of flat tires was not affected significantly by climatic conditions.

TABLE 3

TOTAL TIME OF CLIMATIC CONDITIONS

Condition	Hours	14-Hr Days
Pavement:		
Dry	3,150 hours	225.000
Wet	373 hours	26.643
Slushy	47 hours	3.357
Weather:		
Clear or cloudy	3,306 hours	226.143
Rain	183 hours	13.071
Snow	81 hours	5.786
Temperature:		
Low (below 30)	672 hours	48.000
Medium (30-70)	2,366 hours	169.000
High (above 70)	532 hours	38.000

Duration of Incidents

Figure 6 shows the duration of the 927 incidents, based on an accumulated percentile, for the study year. From this data, average durations were determined to be 6.14 min for accidents, 4.94 min for vehicle disabilities; over-all average of 5.24 min. Median value of all incidents was 3 min. The duration of 50 percent of

TABLE 4

VEHICULAR INCIDENTS[a]
BY CLIMATIC CONDITIONS—TOTALS

Climatic Condition	Total Incidents	Total Time of Condition		Study Section Incidents			
		Hours	Days (14 hr)	Per Hour	Per Day (14 hr)	Per Hour per Mile	Per Day per Mile
Pavement:							
Dry	704	3,150	225.00	0.22	3.13	0.03	0.49
Wet	177	373	26.64	0.47	6.64	0.07	1.04
Slushy	46	47	3.36	0.98	13.70	0.15	2.14
Weather:							
Clear	720	3,306	236.14	0.22	3.05	0.03	0.48
Rain	106	183	13.07	0.58	8.11	0.09	1.27
Snow	101	81	5.79	1.25	17.46	0.19	2.73
Temperature:							
Below 30	253	672	48.00	0.38	5.27	0.06	0.82
30 to 70	561	2,366	169.00	0.24	3.32	0.04	0.52
Above 70	113	532	38.00	0.21	2.97	0.03	0.46

[a]For both directions—6.4 mi, total study area length.

TABLE 5

VEHICULAR INCIDENTS[a]
BY CLIMATIC CONDITIONS—ACCIDENTS

Climatic Condition	Total Accidents	Total Time of Condition		Study Section Accidents			
		Hours	Days (14 hr)	Per Hour	Per Day (14 hr)	Per Hour per Mile	Per Day per Mile
Pavement:							
Dry	165	3,150	225.00	0.05	0.73	0.01	0.11
Wet	53	373	26.64	0.14	1.99	0.02	0.31
Slushy	11	47	3.36	0.23	3.28	0.04	0.51
Weather:							
Clear	173	3,306	236.14	0.05	0.73	0.01	0.11
Rain	31	183	13.07	0.17	2.37	0.03	0.37
Snow	25	81	5.79	0.31	4.32	0.05	0.68
Temperature:							
Below 30	67	672	48.00	0.10	1.40	0.02	0.22
30 to 70	141	2,366	169.00	0.06	0.83	0.01	0.13
Above 70	21	532	38.00	0.04	0.55	0.01	0.09

[a]For both directions—6.4 mi, total study area length.

TABLE 6

VEHICULAR INCIDENTS[a]
BY CLIMATIC CONDITIONS—VEHICLE DISABILITIES

Climatic Condition	Total Vehicle Disabilities	Total Time of Condition		Study Section Vehicle Disabilities			
		Hours	Days (14 hr)	Per Hour	Per Day (14 hr)	Per Hour per Mile	Per Day per Mile
Pavement:							
Dry	539	3,150	225.00	0.17	2.40	0.03	0.37
Wet	124	373	26.64	0.33	4.65	0.05	0.73
Slushy	35	47	3.36	0.74	10.43	0.12	1.63
Weather:							
Clear	547	3,306	236.14	0.17	2.32	0.03	0.36
Rain	75	183	13.07	0.41	5.74	0.06	0.90
Snow	76	81	5.79	0.94	13.14	0.15	2.05
Temperature:							
Below 30	186	672	48.00	0.28	3.88	0.04	0.61
30 to 70	420	2,366	169.00	0.18	2.49	0.03	0.39
Above 70	92	532	38.00	0.17	2.42	0.03	0.38

[a]For both directions—6.4 mi, total study area length.

all incidents was 3 min or less and 12 percent lasted for 10 min or more. About 90 percent of the vehicle disabilities and 86 percent of the accidents were on the freeway for 10 min or less. The recorded duration of the incident was terminated when the

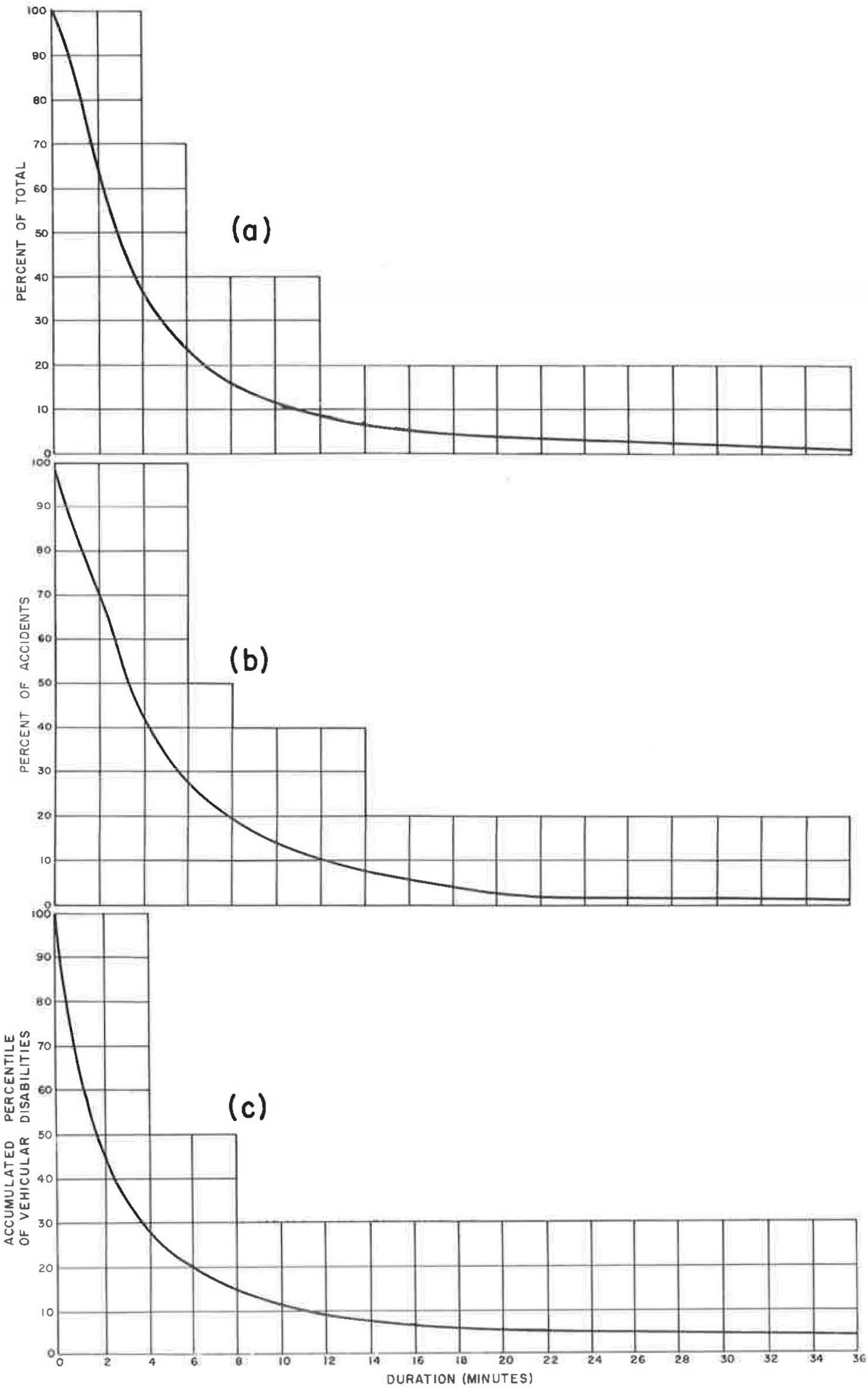


Figure 6. Duration of (a) incidents, (b) accidents, and (c) vehicular disabilities.

TABLE 7

TOTAL VEHICULAR INCIDENTS
BY TYPE OF AID RECEIVED

Type of Aid	Accidents		Veh. Disabilities		Total Incidents	
	No.	% of Total	No.	% of Total	No.	% of Total
Self to shoulder	108	46	113	16	221	24
Self to freeway	13	6	187	27	200	22
Pushed or pulled to shoulder	52	23	227	33	279	30
Pushed or pulled on freeway	2	1	126	18	128	13
Self to shoulder with official aid	54	24	45	6	99	11
Total	229	100	698	100	927	100

incident (vehicle) was removed to the shoulder or when the freeway traffic resumed movement.

Figure 6 also shows duration of accidents and vehicle disabilities. Similarity in the curves is apparent.

Vehicle Incidents and Type of Aid

From the general log, the freeway incidents were correlated by type of aid and by lane of occurrence. The "aid" was classified as follows:

1. Self to shoulder—motorist assisted self and moved vehicle to shoulder;
2. Self to freeway—motorist assisted self and continued on freeway;
3. Pushed or pulled to shoulder—vehicle disabled to the extent that it was necessary to be pushed or pulled to the shoulder;
4. Pushed or pulled on freeway—motorist, unable to repair disabled vehicle, received aid and was pushed or pulled on freeway; and
5. Self to shoulder with official aid—although the motorist was able to move vehicle, he waited for official aid to arrive before driving vehicle to the shoulder.

Table 7 gives the vehicular incidents for the total study section by types of aid.

Of the 927 total incidents, 46 percent of the motorists were able to move their vehicles to the shoulder or continue on the freeway, 43 percent needed assistance, and 11 percent waited for official aid before driving their vehicles to the shoulder. The total of 698 vehicle disabilities included 593 stalled vehicles. Forty-one percent of the motorists were able to restart their vehicles, 55 percent received aid from another motorist, and only 4 percent waited for official aid.

The record of accidents does not list the total vehicles involved but only the total number of accidents. Fifty-two percent of the accident vehicles were self-removed from the freeway, 24 percent needed a tow or push, and 24 percent waited for official assistance. Analysis of total incident data, to relate type of aid to the lane in which the incident occurred, has not been completed. Preliminary data reveal that 14 percent of the motorists, waiting for official aid, were involved in median lane (1) incidents as compared to 3 percent of the motorists stranded in the shoulder lane. Also, 32 percent of the accident vehicles in the median lane were movable but motorists waited for official aid. As expected, the need for assistance increased as the incidents occurred in lanes farther from the refuge shoulder.

SUMMARY

Incidents by Location

A comparison of incidents in the 4- and 3-lane sections reveals greater frequency, reflected in daily averages, in the 4-lane section, as follows:

1. 4-lane section = 0.69 incidents per mile = 0.16 accidents and 0.53 vehicle disabilities.
   3-lane section = 0.51 incidents per mile = 0.13 accidents and 0.38 vehicle disabilities.
2. 4-lane section = 11.9 total incidents per million vehicle-miles = 2.8 accidents and 9.1 vehicle disabilities.
   3-lane section = 8.7 total incidents per million vehicle-miles = 2.2 accidents and 6.5 vehicle disabilities.

Comparison of total incidents, by lane, revealed that 45.7 percent occurred in the median lane—more than twice those in lane 2 and lane 3; 47 percent of the accidents occurred in the median lane; and 44 percent of the vehicle disabilities occurred in the median lane.

Incidents by Time of Day, Direction of Travel, and Day of Week

As expected, the frequency of incidents was greater during the time of day when the heavy volumes occurred and the greatest number of incidents occurred during the periods when maximum density of the freeway was exceeded. More analysis of the volume and density relationship to incidents is planned for the future.

Neither the direction of travel nor the day of the week had any significant influence on vehicular incidents.

Vehicular Incidents Related to Climatic Conditions

The greatest number of incidents occurred during the months when driving conditions were made more hazardous by low temperatures or sudden changes in the weather.

Compared to dry conditions, there were twice as many incidents per day per mile when the pavement was wet and four times as many when the pavement was slushy. There were three times as many incidents per day per mile when it was raining and almost six times as many when it was snowing.

Comparison by temperature indicated the frequency of total incidents at below 30° was 60 percent more than above 30°. As compared to dry pavement, the number of vehicle disabilities occurring on wet pavement was almost doubled and on slushy pavement was 4½ times as frequent. Judging solely on weather conditions, the frequency of vehicle disabilities almost tripled during rainy weather and increased by almost 6 times when it was snowing. During temperatures below 30°, the vehicle disabilities were almost 1.6 times that above 30°.

The frequency of accidents when the pavement was wet was about 3 times greater than when it was dry and during slushy conditions it was almost 5 times as much.

There were more than three times as many accidents in rain and six times as many in snow as there were in clear weather. For below 30F the frequency of accidents was almost double that for above 30F.

Duration of Incidents

The average duration for the total 927 incidents was 5.24 min; the median value was 3 min. Thirty-eight percent of the total incidents had a duration of 3 to 10 min; 12 percent had a duration of 10 min or more.

The average duration of 229 accidents was 6.14 min; 86 percent of the accidents had a duration of less than 10 min.

The average duration of vehicle disabilities was 4.94 min; and 90 percent of the vehicle disabilities had a duration of less than 10 min.

Vehicle Incidents and Type of Aid

Examination of the type of assistance needed by the motorists involved in incidents showed that 46 percent of the motorists moved their vehicles to the shoulder by themselves or continued on the freeway, 43 percent needed a tow or push, and 11 percent waited in the lane for official aid even though their vehicles were in a condition to be moved. There was a gradual increase of motorists in the last category as the incident occurred farther from the shoulder (3 percent in the shoulder lane and 14 percent in the median lane).

The 698 vehicle disabilities included 593 stalled vehicles. Of these 593, 41 percent of the motorists were able to restart their vehicles and either continue on the freeway or move to the shoulder, 55 percent received aid from another motorist or wrecker, and only 4 percent waited for official aid before driving to the shoulder.

Out of a total of 229 accidents, 52 percent of the situations were resolved by assistance provided by the involved motorists, 24 percent needed a tow or push, and 24 percent waited for officials to arrive even though they were able to move the vehicles to the shoulder under their own power. Again, a high percentage of accidents occurring in the median lane were movable, but motorists waited for official aid. The relationship of lane of incident location and type of assistance required and received will be studied further to supplement this paper.

ACKNOWLEDGMENT

The author wishes to express his appreciation to Gordon Paesani of the Michigan State Highway Department, John C. Lodge Freeway Surveillance Project, for permitting use of data from a project study, entitled "Vehicular Incidents on an Urban Freeway," and for his generous cooperation and assistance in the preparation of this paper.

Appendix

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF ENGINEERING-TRAFFIC DIVISION

T. V. SURVEILLANCE DAILY LOG

PAGE 3 OF 4 Form 1578 B (Rev. 4/63)

DATE January 11 19 63 OPERATOR No. 3 Keryeski OBSERVER J. Taube

Incident Number	Time Start	Time End	Cause	Location Camera	Location Lane	Lanes Affected	Assistance Non Official	Assistance Official	Assistance Time Notified	Assistance Time Arrived	Assistance Action Taken	Pav't. Surface	Weather	Remarks
25	1413	1414	O	2o	S	S	S					4	2	Single axle—unknown
26	1550	1551	St	10o	2	All	PM			1551	SP	4	2	Standard—Stalled
26	1551	1551	Aid	10o	2	All						4	2	Standard—Pushed stalled veh.
27	1606	1606	St	5o	3	2 & 3	S				SS	2	2	Standard—Momentary stall
28	1627	1633	V	13i	S	S						2	2	Standard—Received ticket
28	1627	1633	V	13i	S	S						2	2	Police car—Issued ticket
29	1659	1701	St	12o	1	All	S				SS	5	4	Standard—Stalled
30	1701	1702	O	9o	S	S						5	4	Standard—unknown
31	1717	1810	St	11o	1	All		W/P	1717	1722	P	5	4	Standard—On shoulder at 1723
31	1722	1810	Aid	11o	1	All						5	4	Wrecker—On shoulder at 1723 Gave push to standard veh.
31	1722	1725	Aid	11o	S	S						5	4	Police car—Gave aid
32	1739	1846	St	14o	R	Ramp		W		1801	P	5	4	Standard—On shoulder at 1826
32	1801	1846	Aid	14o	R	Ramp						5	4	Wrecker—Pushed standard veh. On shoulder at 1826
33	1846	1848	O	7i	S	S						5	4	Standard—unknown
34	1905	1938	A	1o	1	All		W/P		1905	P	5	4	Standard—On shoulder at 1917
34	1905	1935	A	1o	1	All		W/P		1905		5	4	Standard—On shoulder at 1906
34	1905	1931	A	1o	1	All		W/P		1905		5	4	Standard—On shoulder at 1906
34	1905	1906	Aid	1o	1	All						5	4	Police car—On scene
34	1912	1938	Aid	1o	1	All						5	4	Wrecker—On shoulder at 1917 Pushed veh. shoulder

CODING FOR OBSERVER'S LOG

CAUSE

A - Accident
B - Vehicular breakdown
flat tire
motor trouble
C - Congestion

ST - Stall
V - Violation
W - Inclement weather
O - Other

PAVEMENT

1 - Dry
2 - Wet
3 - Icy
4 - Snowy
5 - Slushy

LOCATION

USE CAMERA NUMBER AND DIRECTION OF TRAVEL

o - Outbound
i - Inbound

ASSISTANCE

BY NON-OFFICIAL

S - Self help
M - Passing motorist

BY OFFICIAL

P - Police dept.
W - Wrecker
F - Fire dept.

LANES

1 - Median
2 - Center
3 - Right
4 - Extra right

M - Median
S - Shoulder
R - Ramp

ACTION TAKEN

P - Moved to shoulder
T - Moved off freeway
SP - Started (by push)
SS - Started (by self)
C - Changed tire

WEATHER

1 - Clear
2 - Overcast
3 - Rain
4 - Snow

5 - Fog
6 - Sleet
7 - Hail

Effect of Buses on Freeway Capacity

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•QUESTIONS frequently asked are, "How many buses can an exclusive bus lane on a freeway carry?" and "What is the effect of buses on mixed traffic flows on freeways?" Little published material has been released aimed toward providing an answer to these questions. Therefore, this study was initiated in the spring of 1962 in an effort to make a first step toward providing answers regarding bus equivalency in connection with mass transit operation on highways. It was limited to a study of buses on freeways and limited-access facilities.

In particular, the purpose of this study was to measure the speeds and the spacing between buses on freeways or other high-type facilities where several buses form a continuous group at relatively frequent intervals during the peak periods, to determine the passenger car equivalent of buses on such roads, thus permitting determination of the theoretical capacity of the separate all-bus lane on a freeway. It was felt that by studying a combination of speed-volume data and "cluster" data, it would be possible to develop capacity values as well as to determine the effect of buses on the traffic stream.

The study was designed to determine theoretical maximum volumes within a specified speed range. These volumes have been computed for level, tangent sections of freeway with 12-ft lane widths and no lateral restrictions. The effects of freeway interchanges, grades, ramps and ramp spacing have not been considered. The highest volume recorded of the several "maximum volumes within a specified speed range" for traffic streams consisting of 100 percent autos should, according to this criterion, approximate possible capacity under ideal conditions as defined in the Highway Capacity Manual (1). "Possible capacity," as determined by the study data and methods of analysis of this report, was calculated to be an average lane volume of 2,050 ± 50 autos/hr in both the 23- to 27-mph and the 28- to 32-mph speed ranges. This volume and these speed ranges agree quite closely with the possible capacity of 2,000 autos/hr per lane at approximately 30 mph presented in the Manual (1). The possible capacity of an exclusive bus lane, calculated by the same method, was determined to be 1,300 ± 100 buses/hr within the 28- to 32-mph speed range.

Locations Studied

Inquiries were sent to several agencies in an effort to find locations where there were a large number of buses in the traffic stream and/or where a separate lane was reserved for transit use. After studying the descriptions submitted of various locations the following areas were selected for study:

1. The Route 3 approaches to the Lincoln Tunnel in New Jersey;
2. The center tube of the Lincoln Tunnel;
3. Shoreway West in Cleveland, Ohio;
4. Lakeshore Drive in Chicago, Illinois;
5. The Mark Twain Expressway in St. Louis, Missouri;
6. The Bayshore Freeway in San Francisco, California; and
7. The lower deck of the San Francisco-Oakland Bay Bridge, including studies at three different locations on the exclusive bus lane in use at that time.

Paper sponsored by Committee on Highway Capacity.

Method of Field Observation

To examine the factors on which the determination of the capacity of a separate bus lane and the effect of buses on a typical freeway must depend, it was necessary to find a field method that would permit measurement of the following traffic characteristics: (a) lane volumes; (b) time headway in increments of hours (i. e., the spacing between vehicles measured in units of time); (c) vehicle classification (herein vehicles are classified as autos, buses, and trucks); and (d) vehicle speed.

The listed requirements suggested use of some kind of apparatus that would make a continuous record. Two relatively simple kinds of apparatus to do this are a motion picture camera and a graphic time recorder in which one or more pens record on a moving chart. The graphic time recorder was used in this study because of greater flexibility of the equipment in choosing site locations. For example, in using a motion picture camera it is necessary to have good light and a good view of the roadway at all times, whereas in using the graphic time recorder it was possible to study at any time, from the side of the road, on bridges, and in tunnels where lighting is marginal.

The graphic time recorder used in this study was a standard twenty-pen recorder, in which each pen, when actuated by its particular push button, makes a characteristic mark on moving paper. All observations were visual and recorded manually through the use of these push buttons.

In this study, only nine of the twenty pens were utilized. Chart paper moved through the recorder at the rate of 10 in./min. The paper was marked with 100 spaces/6 in. At the rate of 10 in./min, the time spacings could be read to the nearest 0.00005 hr (i.e., to the nearest 0.18 sec).

Pens 1, 2, and 3 were used to record automobiles, trucks, and buses, respectively, in lane 1. Similarly, pens 11, 12, and 13 were used to record automobiles, trucks and buses, respectively, in lane 2. Pens 4 and 14 were used to measure the time required for a vehicle to traverse a speed trap of predetermined length in lanes 1 and 2, respectively. The vehicle speeds were later calculated in the office. Pen 20 was actuated at 1-min intervals several times throughout the hour to check chart speed and at 5-min intervals to record the time. On multilane freeways only two lanes were studied at one time.

PRELIMINARY ANALYSIS

The preliminary analysis consisted of calculating and recording the time spacings, volumes, classifications, and speeds observed. These data were recorded on punch cards for use in computer analysis. It was found necessary to apply a paper speed factor to correct for day-to-day variation in chart speed on the spring-wound recorder.

Two breakdowns of these data were then made. The first breakdown was simply a calculation of the hourly volumes observed, the 15-min volumes observed, and the 5-min volumes observed for each lane at each study site. The volumes for the 5-min periods were further broken down into the different vehicle classifications and the average speed and speed ranges were calculated for the 5-min periods.

CLUSTER ANALYSIS

The basic method of analysis for this report is the cluster analysis. The theory behind the cluster analysis is that capacity flows are being approached on any actual highway when the driver of a vehicle begins to feel restricted by the vehicles around him.

The second breakdown of the field data, therefore, consisted of picking "clusters" from these data and calculating the cluster value. A cluster was defined as a group of three or more vehicles of the same type with a time headway between individual vehicles of 0.0020 hr (approximately 7 sec) or less. An average speed was calculated for each cluster and a location code was recorded so that the study site, lane number, date, and 5-min interval in which the cluster appeared could be readily determined. The cluster values were sorted into various speed ranges.

The cluster value is simply the expansion of a cluster volume to an hourly volume rate. This is calculated by

$$V_k = \frac{(N-1)\ (10{,}000)}{ft} \tag{1}$$

in which

V_k = cluster value in vehicles per hour;
N = number of vehicles per cluster;
f = factor to correct for variations in chart speed on the spring-wound recorder;
t = time spacing in 0.0001's hour (i.e., the time spacing from the front axle of the first vehicle in the cluster to the front axle of the last vehicle in the cluster); and
10,000 = factor to convert to a full hour.

For example, if a cluster consisted of three automobiles and the distance from the front axle of the first to the front axle of the second measured in units of time was 0.0011 hr; and if the time spacing between front axles of the second and third vehicles was 0.0009 hr; and if the chart paper was moving at exactly 10 in./min, the f-factor is 1.0.

In this example, then

N = 3
f = 1.0
t = 9 + 11 = 20 (i.e., a cumulative time spacing of 20 measured in units of ten-thousandths of an hour)

$$V_k = \frac{2\ (10{,}000)}{1.0\ (20)} = \frac{20{,}000}{20}$$

V_k = 1,000 vehicles per hour

The cut-off point of 0.0020 hr is that time headway frequently used in driver behavior studies as the point where the speed of a vehicle is affected by the speed of the vehicle preceding it. The Manual (1, p. 39) shows the critical spacing to be 9 sec or 0.0025 hr. For purposes of the cluster analysis, the use of 0.0020 hr instead of 0.0025 hr produced no significant differences in the results, and also was more conservative, establishing more certain clusters. Therefore, it was used throughout for greater ease in the manual calculations and checks.

It was assumed that the factors in Table 8 of the Manual (1, p. 54) could be applied to all types of vehicles. This table gives the means to correct for the effect of lateral restrictions and lane width; thus, the cluster data are adjusted to represent traffic on a 12-ft lane with no lateral restrictions. Once this adjustment is made, the main factors that are left to affect the driver of a vehicle are the vehicle ahead and the vehicle behind. The effect of these vehicles can be measured in terms of time headway.

It was previously mentioned that the cluster values were sorted into speed ranges. Within any particular speed range there was a distribution of cluster values. Therefore, if cluster values were averaged within any speed range (speed grouping), the average cluster value for that speed grouping would represent the maximum volume that could be expected to pass a point in one hour's time at the average speed represented by that speed grouping.

For example, if there were 49 bus clusters recorded traveling in the speed range of 28-32 mph; and if the average of the 49 cluster values was calculated to be 1,300 buses/hr it was assumed that a lane carrying 100 percent buses would have a maximum volume of 1,300 buses/hr when the average speed was approximately 30 mph.

It was felt that the average cluster values should be limited to an accuracy of ±150 vph within any speed range. To effect this the 95 percent confidence interval was calculated using the standard "t" distribution for each average cluster value and, if the confidence interval was >±150 vph, that average cluster value was considered to be "not statistically significant" and discarded. In other words, under capacity conditions on a 12-ft lane with no lateral restrictions, the volume rates presented in this paper ±150 vph should be actually observed at least 95 times out of 100.

Speed Range Volumes

The same correction factors (i.e., for lateral restrictions and lane widths) were applied to the speed volume groupings. There were three stages of these groupings: (a) all 5-min periods without commercial vehicles, except for buses (thereby eliminating the effect of trucks as a variable), were sorted into speed groups, and 5-min volumes were expanded to an hourly volume rate which was plotted according to speed grouping, the plots presenting bus vs auto volumes; and (b and c) similar graphs of the peak 5-min periods and the peak hour values were plotted except that trucks were included and converted to equivalent auto volumes by truck equivalency factors (1).

Results of Analysis

As previously mentioned, the cluster analysis was the basic method of analysis. It was assumed that the highest average cluster value at any given location would represent the values approaching possible capacity of the facility being studied. When these values are adjusted to compensate for the reduction in capacity due to lane width and lateral restrictions, the highest adjusted average cluster values represent possible capacity of an "ideal" roadway.

Possible capacity, as defined (1), represents the highest volume reached on a given facility, regardless of the speed at which this volume occurs. In this report separate capacity values are reported for each of the several speed ranges analyzed; each refers to the maximum hourly volume that can be expected within that speed range. Possible capacity would be the highest of these several maximums.

Although each study site initially was analyzed separately, only the analysis of all sites combined is discussed in this report.

Table 1 gives the unadjusted average cluster values for automobiles which were calculated for the seven locations studied, the 95 percent confidence limits, and whether or not these values were considered statistically significant. The values given in Table 1 are not adjusted to correct for lane width and lateral restrictions. The unadjusted average cluster values for automobiles are not significant above the 48- to 52-mph speed range.

Table 2 gives the unadjusted data calculated for bus clusters. There were no bus clusters recorded at speeds in excess of 52 mph. The unadjusted average cluster values for buses were not considered statistically significant above the 38- to 42-mph speed range. Tables 3 and 4 give the values presented in Tables 1 and 2, respectively, adjusted to 12-ft lane widths with no lateral restrictions.

The primary product of this study, the bus equivalent (or ratio of automobile capacities to bus capacities in terms of vehicles per hour) is given in Table 5. This ratio ranges from 1.52 to 1.64 and it indicates that a bus uses the same amount of space on one

TABLE 1

UNADJUSTED AVERAGE CLUSTER VALUES (AUTOS)

Speed Range (mph)	Unadjusted Average Cluster Value, Autos (vph)	95% Confidence Limits (vph)	Statistically Significant
13-17	1,448	± 38	Yes
18-22	1,690	± 36	Yes
23-27	1,844	± 36	Yes
28-32	1,885	± 45	Yes
33-37	1,858	± 48	Yes
38-42	1,756	± 43	Yes
43-47	1,726	± 53	Yes
48-52	1,734	± 88	Yes
53-57	1,705	± 175	No
58-62	1,570	± 406	No
63-67	1,426	± 1,003	No

TABLE 2

UNADJUSTED AVERAGE CLUSTER VALUES (BUSES)

Speed Range (mph)	Unadjusted Average Cluster Values, Buses (vph)	95% Confidence Limits (vph)	Statistically Significant
13-17	936	± 65	Yes
18-22	1,043	± 68	Yes
23-27	1,011	± 70	Yes
28-32	1,122	± 98	Yes
33-37	1,116	± 113	Yes
38-42	1,071	± 122	Yes
43-47	1,198	± 311	No
48-52	1,343	± 293	No
53-57	–[a]	–	No
58-62	–[a]	–	No
63-67	–[a]	–	No

[a]Not recorded.

TABLE 3

AVERAGE ADJUSTED CLUSTER VALUES (AUTOS)

Speed Range (mph)	Average Adjusted Cluster Values, Autos (vph)	95% Confidence Limits (vph)	Statistically Significant
13-17	1,737	± 43	Yes
18-22	1,932	± 38	Yes
23-27	2,045	± 36	Yes
28-32	2,039	± 47	Yes
33-37	1,940	± 50	Yes
38-42	1,832	± 45	Yes
43-47	1,797	± 56	Yes
48-52	1,817	± 95	Yes
53-57	1,770	± 184	No
58-62	1,628	± 403	No
63-67	1,561	± 1,157	No

TABLE 4

AVERAGE ADJUSTED CLUSTER VALUES (BUSES)

Speed Range (mph)	Average Adjusted Cluster Values, Buses (vph)	95% Confidence Limits (vph)	Statistically Significant
13-17	1,146	± 69	Yes
18-22	1,245	± 76	Yes
23-27	1,227	± 77	Yes
28-32	1,280	± 104	Yes
33-37	1,238	± 123	Yes
38-42	1,177	± 131	Yes
43-47	1,311	± 317	No
48-52	1,463	± 333	No
53-57	_[a]	–	–
58-62	_[a]	–	–
63-67	_[a]	–	–

[a]Not recorded.

TABLE 5

SPEED RANGE CAPACITIES

Speed Range (mph)	Speed Range Capacity (vph/lane)		Bus Equiv. Factor
	Autos	Buses	
13-17	1,750	1,150	1.52
18-22	1,950	1,250	1.56
23-27	2,050	1,250	1.64
28-32	2,050	1,300	1.58
33-37	1,950	1,250	1.56
38-42	1,850	1,200	1.55
43-47	1,800	_[a]	–
48-52	1,800	_[a]	–

[a]Not statistically significant.

TABLE 6

ROUNDED SPEED RANGE CAPACITIES

Speed (mph)	Highest Hourly Volume Expected, by Lane	
	100% Autos	100% Buses
15	1,750	1,100
20	1,950	1,200
25	2,050	1,300
30	2,050	1,300
35	1,950	1,200
40	1,850	1,150
45	1,800	1,100
50	1,800	1,100

lane of a highway as 1.52 to 1.64 automobiles would use. From this range, a bus equivalency factor of 1.6 was selected as a reasonably conservative general value. Because the sample sizes of the automobile clusters were much larger than the sample sizes of the bus clusters, the 95 percent confidence intervals for the automobile clusters were less than the 95 percent confidence intervals for the bus clusters; therefore, the automobile cluster values were used as a base and the equivalency factor of 1.6 was applied to this base. Table 6 gives the result of these calculations. The bus equivalency factor of approximately 1.6 was found to be constant for all facilities and for breakdown by lanes.

In Table 6, the possible capacity of a 12-ft lane carrying buses only is found to be 1,300 buses/hr. Similarly, the capacity of a 12-ft lane carrying automobiles only is found to be 2,050 auto/hr.

Originally ± 150 vph was established as the maximum allowable confidence interval; examination of Tables 3 and 4 shows that the maximum 95 percent confidence interval used for automobiles was actually ± 95 vph and the maximum interval for buses was ± 131 vph.

Therefore, it is safe to assume that at capacity flow, on a 12-ft lane with no lateral restrictions, the volume rates presented in Table 6, ± 150 vph, should be observed at least 95 times out of 100. It is also valid to assume that the volumes given in Table 6 will be observed much more often than the outer limit volumes, and the average volumes of any random nationwide sampling of volumes will be very close to those given in Table 6.

Incidentally, the cluster analysis further verifies many previous studies which have reported that the true possible capacity of most highways is achieved somewhere between 25 and 30 mph. The highest volumes that can be expected to be found within each of the different speed ranges, as determined by the cluster analysis, are given in Table 5. These volumes have been rounded to the nearest 50 vph.

Using the bus equivalency factor of 1.6, it is possible to calculate the effect of buses on the capacity of a lane on a free-

TABLE 7

SPEED RANGE CAPACITIES — AVERAGE LANE VOLUMES
MIXED AUTO-BUS TRAFFIC

Percentage of Buses	Capacity Factor	Volume Rate of Auto-Bus Mixed Traffic (vph) at						
		15 Mph	20 Mph	25 Mph	30 Mph	35 Mph	40 Mph	45 Mph
0	1.000	1,750	1,950	2,050	2,050	1,950	1,850	1,800
5	0.971	1,700	1,890	1,990	1,990	1,890	1,800	1,750
10	0.943	1,650	1,840	1,930	1,930	1,840	1,750	1,700
15	0.917	1,600	1,790	1,880	1,880	1,790	1,700	1,650
20	0.893	1,560	1,740	1,830	1,830	1,740	1,650	1,600
25	0.870	1,520	1,700	1,780	1,780	1,700	1,610	1,570
30	0.847	1,480	1,650	1,740	1,740	1,650	1,570	1,530
35	0.826	1,460	1,610	1,690	1,690	1,610	1,530	1,490
40	0.806	1,410	1,570	1,650	1,650	1,570	1,490	1,450
45	0.787	1,380	1,540	1,610	1,610	1,540	1,460	1,420
50	0.769	1,350	1,500	1,580	1,580	1,500	1,420	1,380
55	0.752	1,320	1,470	1,540	1,540	1,470	1,390	1,350
60	0.735	1,290	1,430	1,510	1,510	1,430	1,350	1,320
65	0.719	1,260	1,400	1,470	1,470	1,400	1,330	1,290
70	0.704	1,230	1,370	1,440	1,440	1,370	1,300	1,270
75	0.690	1,210	1,340	1,420	1,420	1,340	1,280	1,240
80	0.676	1,180	1,320	1,390	1,390	1,320	1,250	1,220
85	0.662	1,160	1,290	1,360	1,360	1,290	1,230	1,190
90	0.649	1,140	1,270	1,330	1,330	1,270	1,200	1,170
95	0.637	1,120	1,240	1,310	1,310	1,240	1,180	1,150
100	0.625	1,100	1,200	1,300	1,300	1,200	1,150	1,100

way. This is given in two ways in Table 7: The first is the calculation of a capacity factor for any percentage of buses (this is shown in increments of five percent in the first two columns); the second is the calculation of volume rates of mixed traffic consisting of buses and autos.

The speed range capacity figures given in Tables 6 and 7 are average lane values. When the cluster analysis is made by lanes, different capacity values are found for each lane. Speed range capacities for individual lanes are given in Table 8. The volumes given are for a traffic flow consisting of 100 percent automobiles. These volumes can be divided by the bus equivalency factor of 1.6 to calculate the capacity of a lane carrying 100 percent buses. This can be done because the equivalency factor remained at approximately 1.6 when the cluster values for individual lanes were computed.

The four-lane-divided highway is represented by the two lanes of the center tube of the Lincoln Tunnel inasmuch as this is the only site studied with only two lanes in one direction; therefore, although these values have been adjusted to the standard 12-ft lane with no lateral restrictions, they may not be considered representative of the nation as a whole. However, the highest statistically significant adjusted cluster volume rate recorded in the tunnel is 2,000 equivalent passenger cars in lane 2 at 25 mph, which is the value given as possible capacity in the Manual (1).

From the available data, it appears that the addition of a third lane actually increases the capacity of the median lane. The possible capacity of lane 3 (the median lane) appears to be 2,350 vph at 35 mph, while the possible capacity of both lanes 1 and 2 is 2,100 vph at 25 mph. Although insufficient data were collected in this study to substantiate the fact, it seems reasonable to assume that on an eight-lane-divided highway, lanes 1 and 4 would carry volumes calculated for lanes 1 and 3 from Table 8, and volumes for lanes 2 and 3 would be the same as those calculated for lane 2 in the same table.

TABLE 8

SPEED RANGE CAPACITIES BY LANE
(ADJUSTED CLUSTER VALUES-VPH)

Speed (mph)	Divided Highway				
	Two Lanes One Direction		Three Lanes One Direction		
	Lane 1	Lane 2	Lane 1	Lane 2	Lane 3
15	1,900	1,650	1,650	1,700	1,800
20	1,950	1,850	1,900	1,900	2,050
25	1,900	2,000	2,100	2,100	2,050
30	–	–	1,950	2,000	2,200
35	–	–	1,900	1,850	2,350
40	–	–	1,600	1,850	2,150

OPERATIVE VOLUME

The preceding analysis has described the volume limits for the several speed ranges; these values have been defined as the speed range capacities. The speed range capacities may be considered as the maximum or "possible" free-flow capacity at a certain speed. Because the speed range capacity is the maximum hourly volume that can be obtained at a certain speed, it may be considered the volume rate of impending congestion.

It appeared desirable to investigate also operation under more clearly free-flow conditions. By definition and through use of ogive curves showing the distribution of volume rates within a speed range, it was possible to develop an "operative volume" for a speed range.

The operative volume is that cluster volume rate within a speed range which was exceeded by 50 percent of the observed clusters. The 50th percentile volume was arbitrarily chosen. It was assumed that if 50 percent of the drivers are willing to travel at a spacing closer than that required to attain the operative volume, then the operative volume is a volume rate at which a freeway lane should operate indefinitely without danger of congestion setting in. Operative volumes can be calculated for every speed range. However, it was assumed that operating speeds below 30 mph are not desirable on multilane highways; therefore, the operative volumes were calculated for speeds ranging from 30 to 45 mph at 5-mph increments. Figures 1 and 2 show the ogive curves for the various speed ranges. The fiftieth percentile of each curve is the operative volume for the speed range it represents. These values are shown in Table 9.

Table 9 gives the average cluster values divided by the 50th percentile volumes resulting in a ratio ranging from 1.1 to 1.3 for both buses and autos. The ratio of 1.25 was selected for calculating the operative volumes. The speed range volumes previously calculated were divided by 1.25 to arrive at the operative volumes given in Table 9.

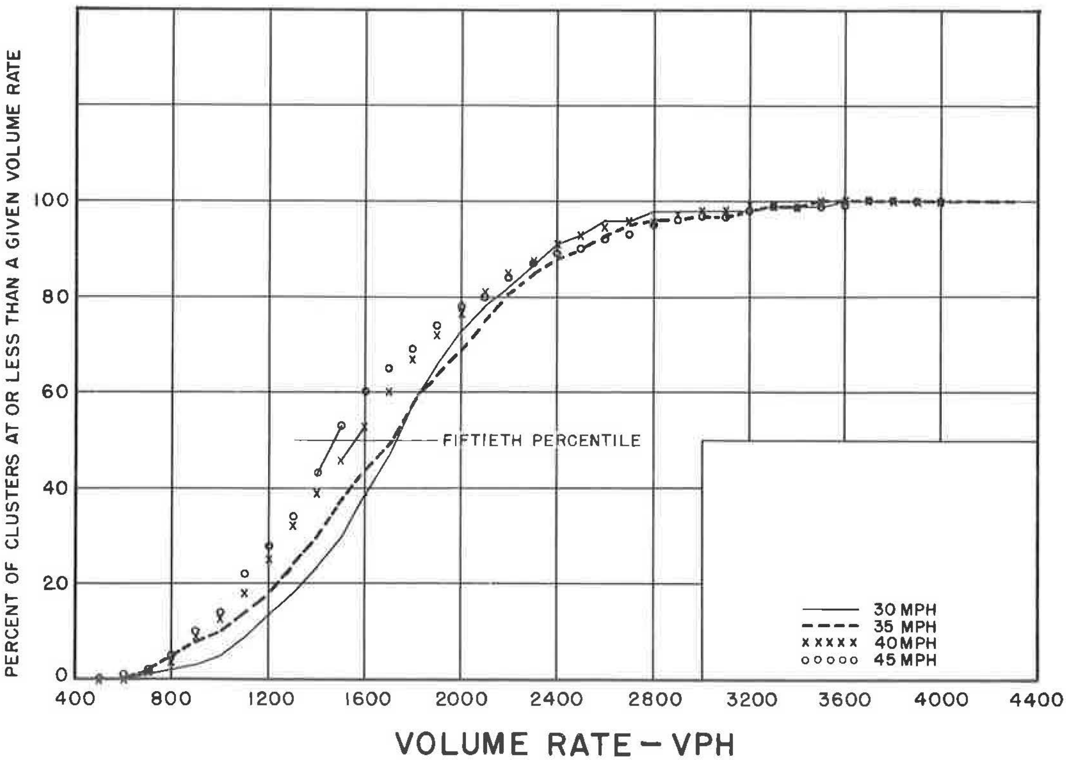

Figure 1. Cumulative distribution, auto clusters.

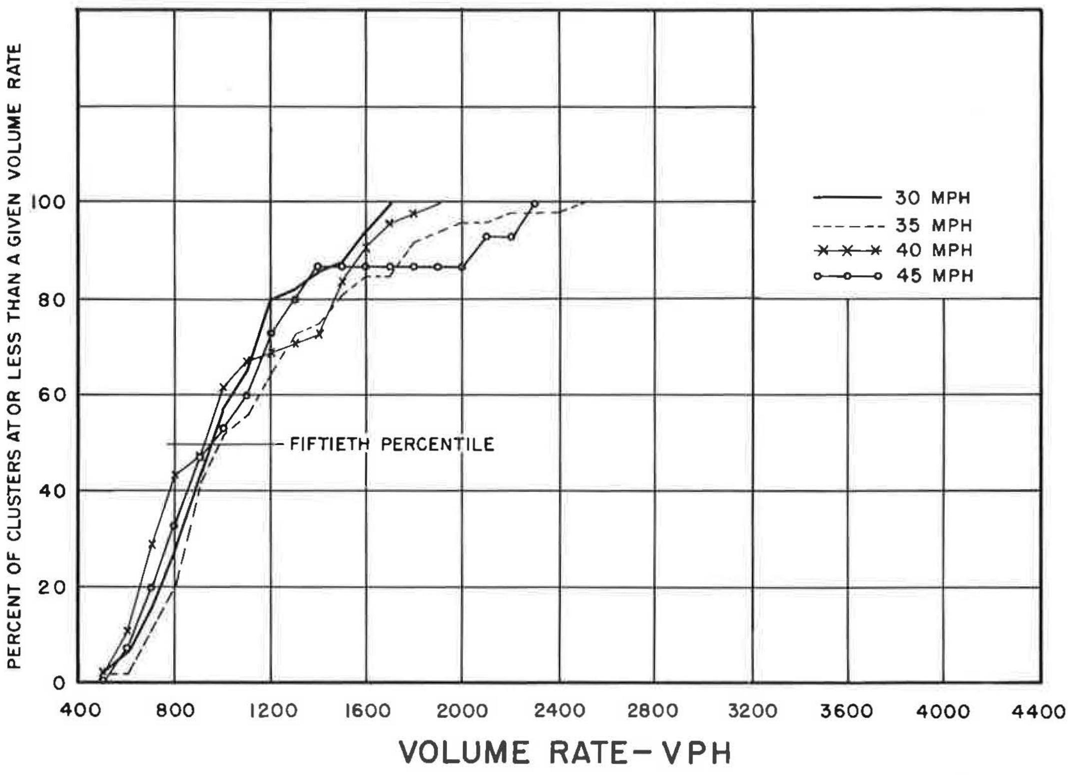

Figure 2. Cumulative distribution, bus clusters.

TABLE 9

COMPARISON OF OPERATIVE VOLUMES WITH SPEED RANGE VOLUMES

Speed Range (mph)	Volume from Ogive Curve, A (vph)	Average Cluster Value, B (vph)	Ratio B/A	Speed (mph)	Average Speed Range Capacity (vph)	Calculated Operative Volume (vph)
			(a) Autos			
28-32	1,720	2,050	1.2	30	2,050	1,640
33-37	1,710	1,950	1.1	35	1,950	1,560
38-42	1,560	1,850	1.2	40	1,850	1,480
43-47	1,470	1,800	1.2	45	1,800	1,440
			(b) Buses			
28-32	950	1,300	1.3	30	1,300	1,040
33-37	980	1,250	1.3	35	1,200	960
38-42	920	1,250	1.3	40	1,150	920
43-47	—	—	—	45	1,100	880

Additional calculations, somewhat incidental to the main purpose of this report, were made to determine the average densities, the standard deviations of the cluster volume rates, and the spacings in each of the speed ranges. Figures 3, 4 and 5 show the relationship of each of these values with speed. It should be noted that the spacings shown in Figure 5 include the length of the vehicle, thus the spacing shown is actually the average distance in feet from the front axle of a following vehicle to the front axle of the vehicle preceding it.

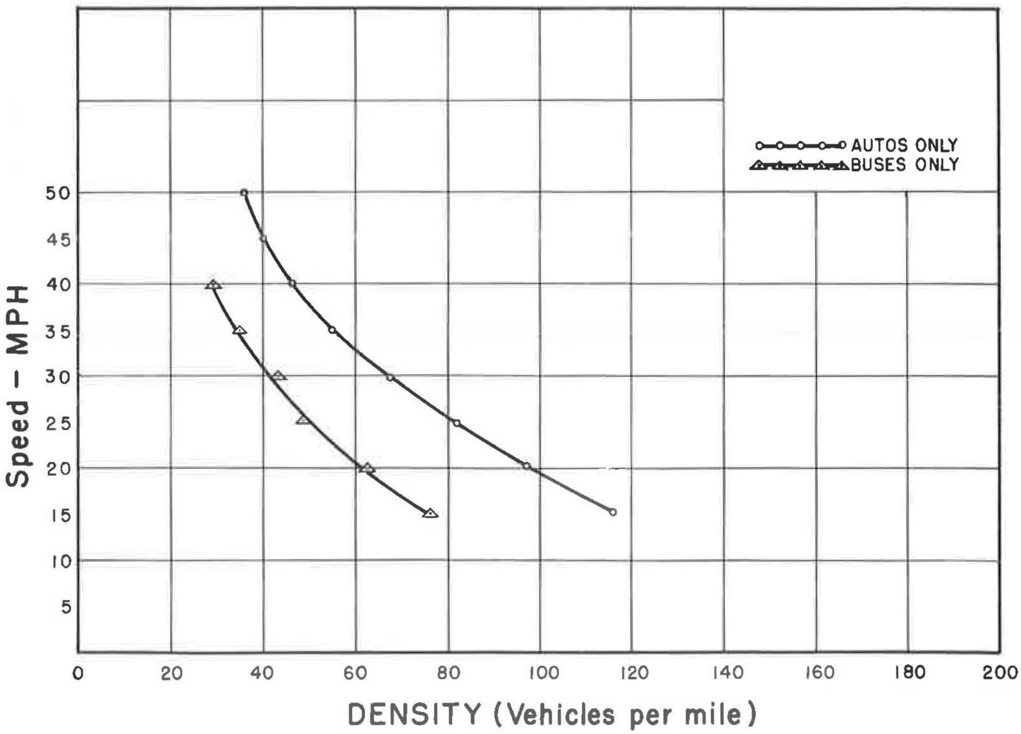

Figure 3. Speed-density relationships, cluster analysis.

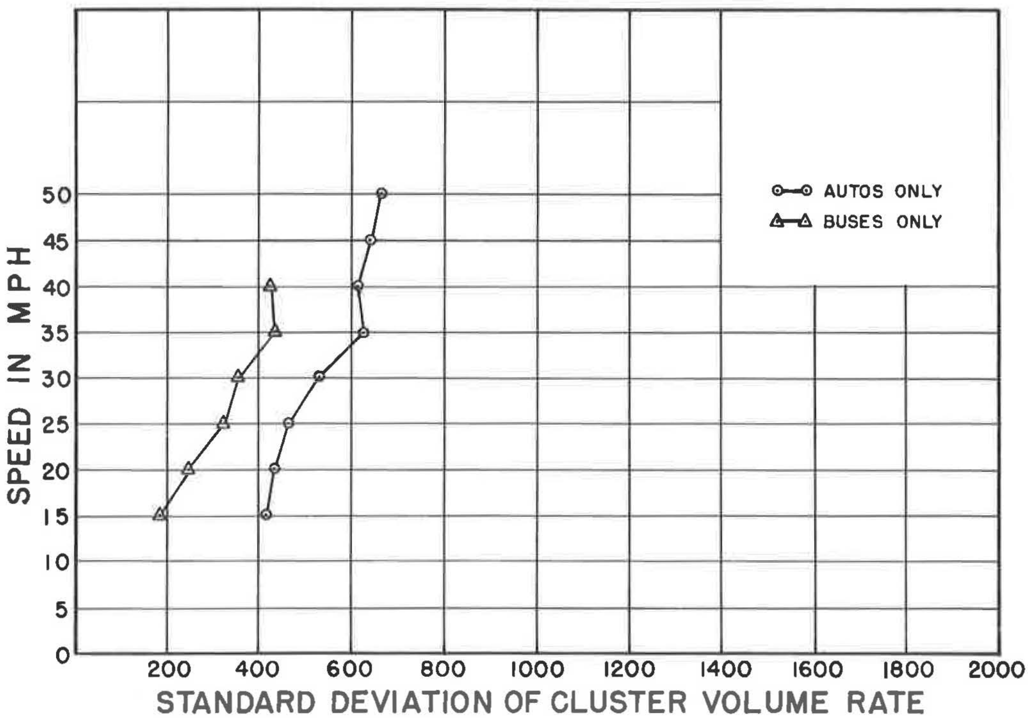

Figure 4. Speed-standard deviation relationship, cluster analysis.

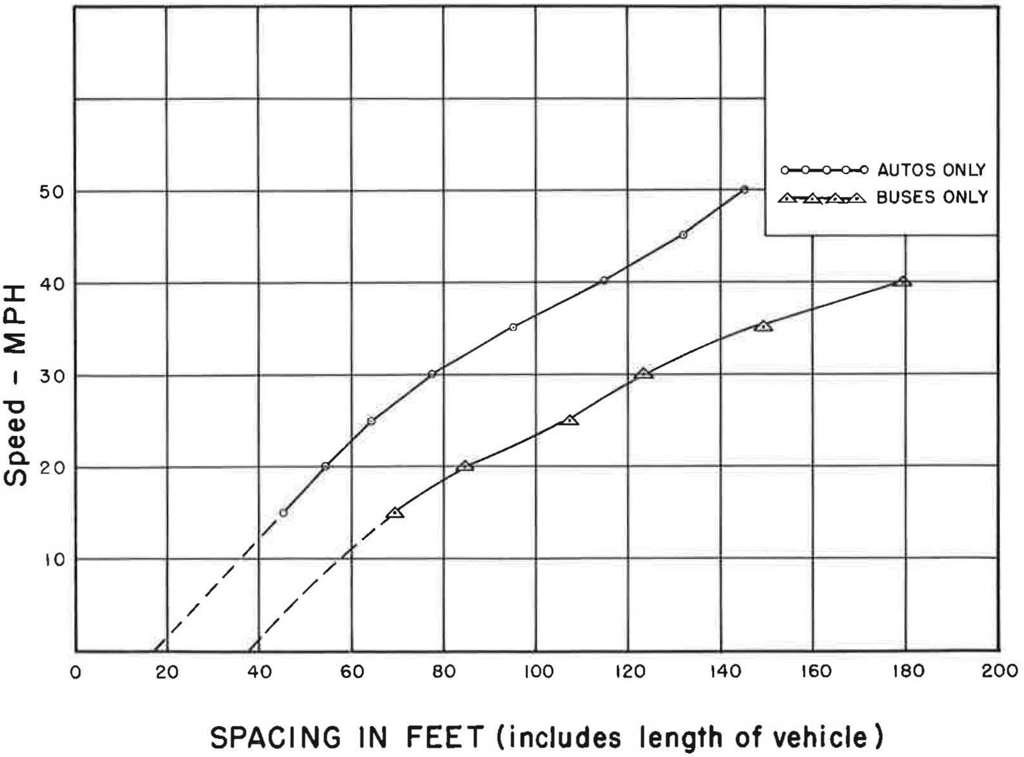

Figure 5. Speed-spacing relationships, cluster analysis.

SPEED RANGE VOLUME ANALYSIS

It was previously mentioned that the speed range volume analysis consisted of sorting the peak hour volumes, the peak 5-min period volumes, and the volumes for every 5-min period studied (omitting those 5-min periods which contained trucks) into speed ranges; each speed grouping was then plotted separately in a series of graphs in an effort to determine a bus equivalency factor independently from the cluster analysis and, in so doing, to check the validity of the cluster analysis.

The results of this analysis indicate that the bus equivalency factor may be taken as 1.6 and comparison of these results with those of the cluster analysis indicates that the cluster analysis is indeed a valid method of computing capacities.

The comparison of the results of the two different methods of computation has been made in two ways. The first method consists of converting the observed volumes to equivalent auto volumes using the bus equivalency factor of 1.6 which has been derived in this study and the truck equivalency factors which are given in the Highway Capacity Manual.

For example, if a lane in level terrain carried 1,000 autos, 250 buses, and 200 trucks, the equivalent auto volume would be 1,000, 400, and 400 autos/hr, respectively. The total equivalent auto volume carried is the sum of these volumes or 1,800 equivalent autos/hr.

The second method of comparison is the plotting of bus volume vs auto volume on graphs for each speed range as described earlier in this report.

The highest peak hour values, in terms of equivalent autos, are given in Table 10 and the average cluster values are given for purposes of comparison. If the limit of ± 150 vph is used as the 95 percent confidence interval, it can be seen (Table 10) that for the highest peak hour volumes recorded, only two exceeded these limits. These were the values for lane 2 at 25 mph and 40 mph on the 6-lane highway. The 2,300 vph

TABLE 10

HIGHEST PEAK HOUR VALUES PER SPEED RANGE COMPARED WITH AVERAGE CLUSTER VALUES

Speed (mph)	Equivalent Auto Volume			Volume as Computed by the Cluster Analysis		
	Lane 1	Lane 2	Lane 3	Lane 1	Lane 2	Lane 3
4-Lane-Divided Highway						
15	1,810	1,754	–	1,900	1,650	–
20	1,797	1,491	–	1,950	1,850	–
25	1,904	N. R.	–	1,900	2,000	–
6-Lane-Divided Highway						
15	–[a]	–[a]	–[a]	1,650	1,700	1,800
20	1,311	–[a]	–[a]	1,900	1,900	2,050
25	1,768	2,300	1,954	2,100	2,100	2,050
30	1,469	1,508[b]	1,629	1,950	2,000	2,200
35	1,875	1,943	N. R.	1,900	1,850	2,350
40	1,408	2,278	2,124	1,600	1,850	2,150

[a] Not recorded.
[b] Accident occurred during this hour.

TABLE 11

TWENTY HIGHEST BUS PEAK HOURS COMPARED WITH CLUSTER VALUES

Bus Vol.	Auto Vol.	Truck Vol.	Truck Vol. in Equiv. Autos	Auto Vol. plus Equiv. Truck Vol.	Adjusted Bus Vol.	Adjusted Auto plus Equiv. Truck Vol.	Avg. of Avg. Speeds	Speed Group	Bus Vol. in Speed Group Equiv. Autos	Total Equiv. Autos	Vol. as Calculated by the Cluster Analysis
213	1,012	130	520	1,532	236	1,700	23.0	25	378	2,078	2,100
273	1,251	8	32	1,283	303	1,424	25.5	25	485	1,727	2,050
337	835	1	4	839	438	1,091	18.3	20	701	1,792	1,950
169	860	4	16	876	220	1,139	17.6	20	352	1,491	1,850
351	815	5	10	826	456	1,094	16.6	15	730	1,804	1,900
176	1,066	0	0	1,066	229	1,386	14.6	15	366	1,752	1,650
209	963	137	548	1,511	232	1,677	22.8	25	371	2,048	2,100
260	1,170	7	28	1,198	289	1,330	25.5	25	462	1,792	2,050
122	910	96	192	1,102	135	1,223	23.8	25	216	1,439[a]	2,100
224	917	20	40	957	249	1,062	20.6	20	398	1,460[a]	2,050
223	1,007	129	516	1,523	250	1,691	24.2	25	413	2,104	2,100
251	1,316	4	16	1,332	279	1,479	24.2	25	446	1,925	2,050
141	1,408	64	128	1,536	157	1,705	41.5	40	251	1,956	1,850
291	1,400	17	34	1,434	323	1,592	40.8	40	517	2,109	2,150
117	1,121	0	0	1,121	118	1,132	37.8	40	189	1,321	1,850
247	8	0	0	8	274	9	26.9	25	438	447	1,300[b]
226	12	0	0	12	251	13	39.0	40	402	415	1,150[b]
196	478	227	454	932	218	1,035	31.9	35	349	1,384	1,900
243	10	0	0	10	270	11	39.3	40	432	443	1,150[b]
163	580	170	340	920	181	1,021	19.0	20	290	1,311	1,900

[a] Major stoppage occurred during this hour.
[b] Values calculated for separate bus lane at speeds given.

recorded at 25 mph is 200 vph more than the average cluster value for that speed range and the 2,278 vph recorded at 40 mph is 428 vph more than the average cluster value for that range. The remainder of the volumes fall either reasonably close to the average cluster value (i.e., within the ± 150 vph confidence interval) or the values are so low that the roadway cannot be considered to have been handling close to possible capacity at the time of the study.

Peak hour volumes using buses per hour per lane as the controlling factor rather than vehicles per hour per lane were also calculated. The 20 highest bus peak hours are given in Table 11.

The highest observed hourly volume for buses per lane occurred in the center tube of the Lincoln Tunnel with volumes of 351 buses/hr, 816 autos/hr and 5 trucks/hr. When these values are adjusted to compensate for lane width and lateral restrictions, they amount to 456 buses/hr, 1,074 autos/hr, and 7 trucks/hr or an equivalent of 1,804 autos/hr. During the same hour, the adjoining lane carried 176 buses/hr, 1,066 autos/hr and no trucks. When these values are adjusted for lane width and lateral restrictions, the volumes become 229 buses/hr and 1,386 autos/hr or an equivalent automobile volume of 1,752 vph. The average speeds within this hour were in the 15-mph range. Thus the adjusted hourly volumes are 1,804 vph and 1,752 vph compared to 1,750 vph, the average cluster value calculated for the combined seven study locations by the cluster analysis, or to 1,900 vph and 1,650 vph calculated for the by-lane breakdown for the tunnel.

The value of 527 buses/hr is also the highest hourly bus volume that was observed on any roadway in the country in this study.

The separate bus lane on the San Francisco-Oakland Bay Bridge, which did not operate at close to capacity volumes, had peaks of 247 buses/hr at an average speed of 25 mph, and of 243 buses/hr at 40 mph. Three different locations on the exclusive bus lane were studied: one near the beginning of the lane, one near the middle, and one close to the end of the lane. The average speed of 25 mph was recorded at the site close to the beginning of the exclusive lane. At this site the buses had not yet accelerated to the 40-mph average speed observed at the other two locations.

It was difficult to get accurate hourly comparisons mainly because of the method of calculating the average speeds over a full hour. The average speeds used for purposes of comparison were simply the average of all speed samples taken during the hour. It was not possible to get a speed for each individual vehicle during the course of the study. Also, the average speeds do not reflect the effect of stoppages because speed samples were not taken when the cars were stopped; thus there are no 0-mph speeds included in the averages. Even with these built-in inaccuracies, most of the peak hour volumes compare favorably with the average cluster values that would be achieved if the speeds retained a fair degree of homogeneity throughout the hour.

The peak 5-min periods in terms of vehicles per hour were also calculated and compared with the cluster values. These peak 5-min periods were converted to equivalent auto volumes as the hourly volumes were. All 5-min values were expanded to hourly rates, as previously mentioned. Table 12 gives the highest peak 5-min periods for each speed range.

In Table 12 the 21 peak periods shown are the highest peak 5-min period shown in each speed range. Six of the values are higher than the upper limit of the 95 percent confidence intervals. Because 1,022 different 5-min periods were included in this analysis, it can be expected that many of them should fall outside of the 95 percent confidence intervals.

All 5-min periods which did not contain any trucks were plotted with auto volume vs bus volume and these values were compared with the limiting volumes calculated with the bus equivalency factor derived from the cluster analysis. Figures 6 through 10 show samples of these graphs. Separate graphs have been plotted for each lane in every speed range for both 4-lane and 6-lane-divided highways and are fairly good examples of how the observed volumes compared with the calculated cluster values. The solid line in each graph represents the speed range capacity in terms of mixed bus and auto traffic as calculated from the cluster analysis. The

TABLE 12

HIGHEST PEAK 5-MIN PERIOD IN A SPEED RANGE

Speed (mph)	Equivalent Auto Vol.			Vol. as Computed by Cluster Analysis		
	Lane 1	Lane 2	Lane 3	Lane 1	Lane 2	Lane 3
(a) 4-Lane-Divided Highway						
15	—[a]	1,875	—	1,900	1,650	—
20	1,901	—[a]	—	1,950	1,850	—
25	1,856	1,938	—	1,900	2,000	—
(b) 6-Lane-Divided Highway						
15	—[a]	1,899	—[a]	1,650	1,700	1,800
20	1,676	2,510	2,097	1,900	1,900	2,050
25	1,958	2,105	2,105	2,100	2,100	2,050
30	2,031	1,869	—[a]	1,950	2,000	2,200
35	2,013	2,421	2,323	1,900	1,850	2,350
40	1,444	1,977	2,321	1,600	1,850	2,150
45	1,549	2,229	—[a]	1,600	1,850	2,100

[a]Not recorded.

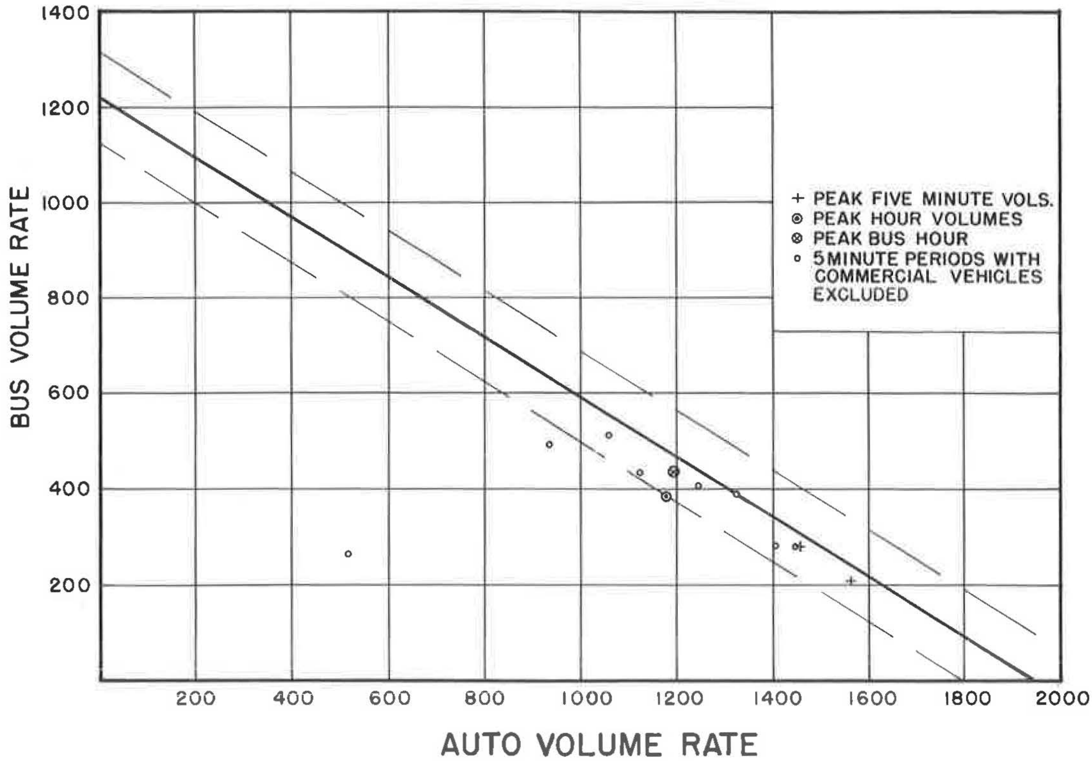

Figure 6. Auto vs bus volume, 4-lane divided, lane 1—20 mph.

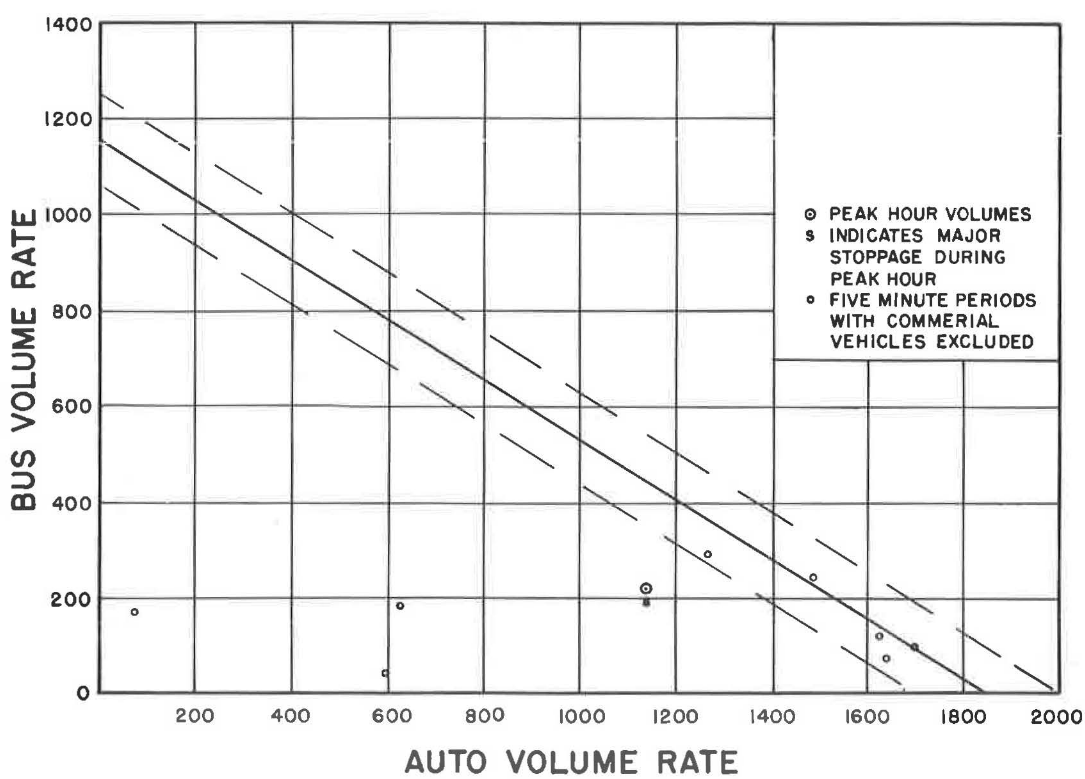

Figure 7. Auto vs bus volume, 4-lane divided, lane 2—20 mph.

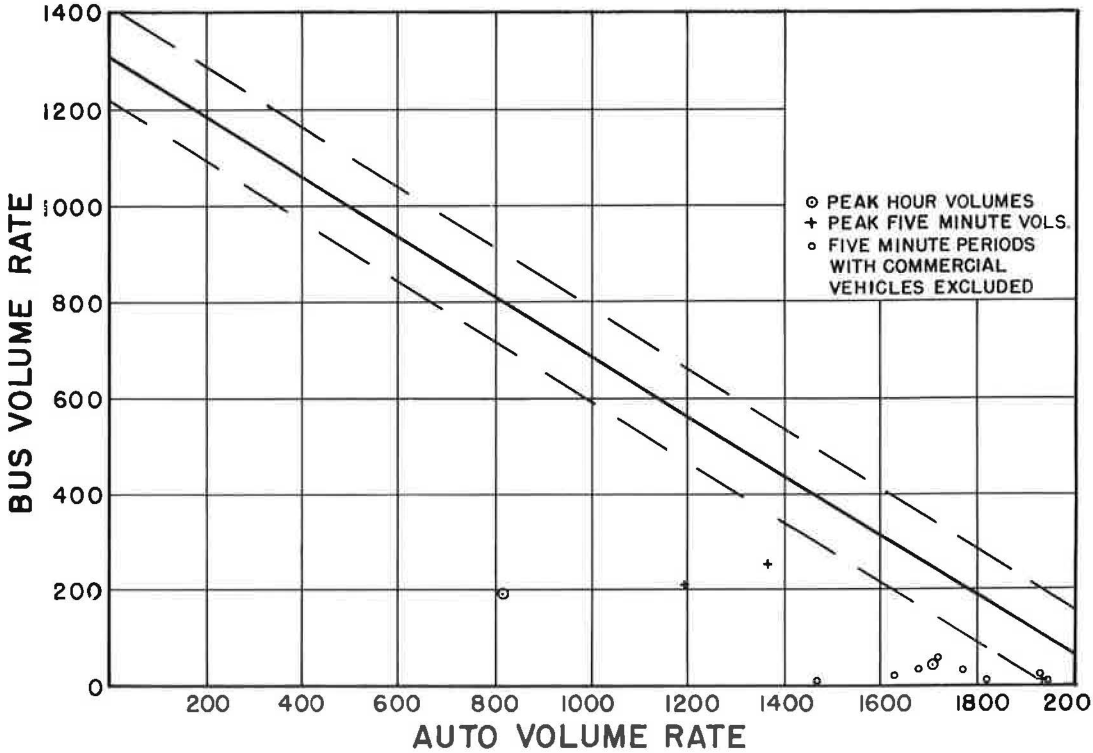

Figure 8. Auto vs bus volume, 6-lane divided, lane 1—25 mph.

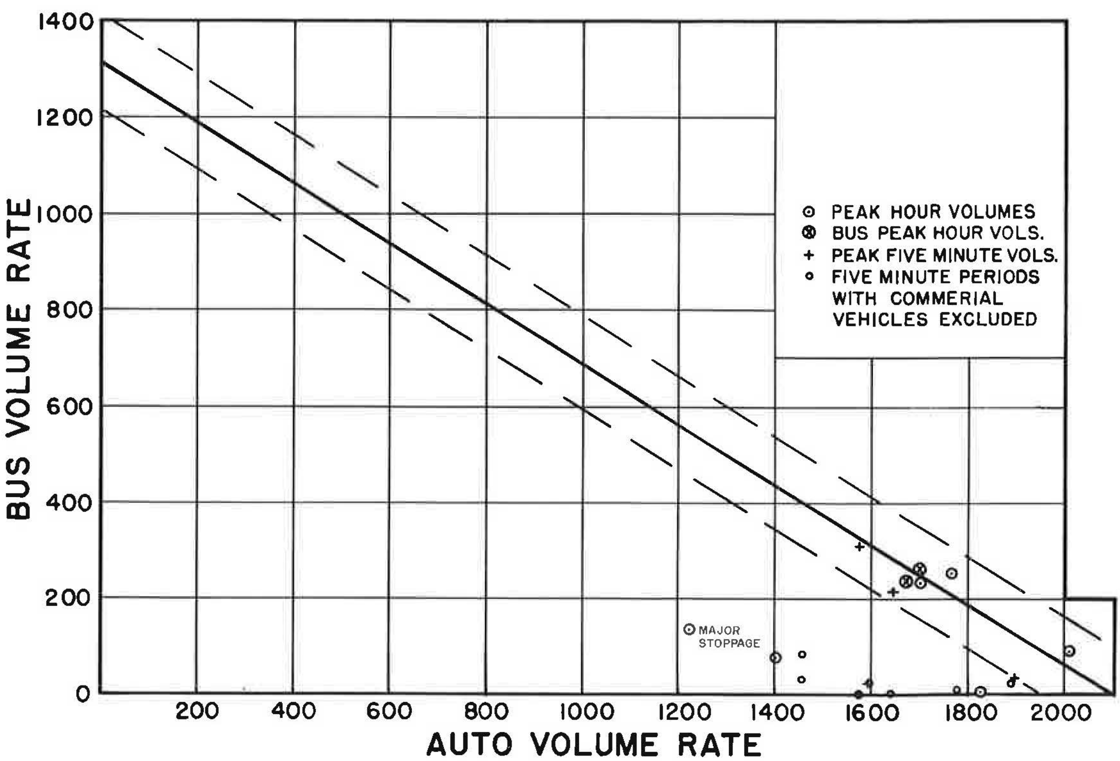

Figure 9. Auto vs bus volume, 6-lane divided, lane 2—25 mph.

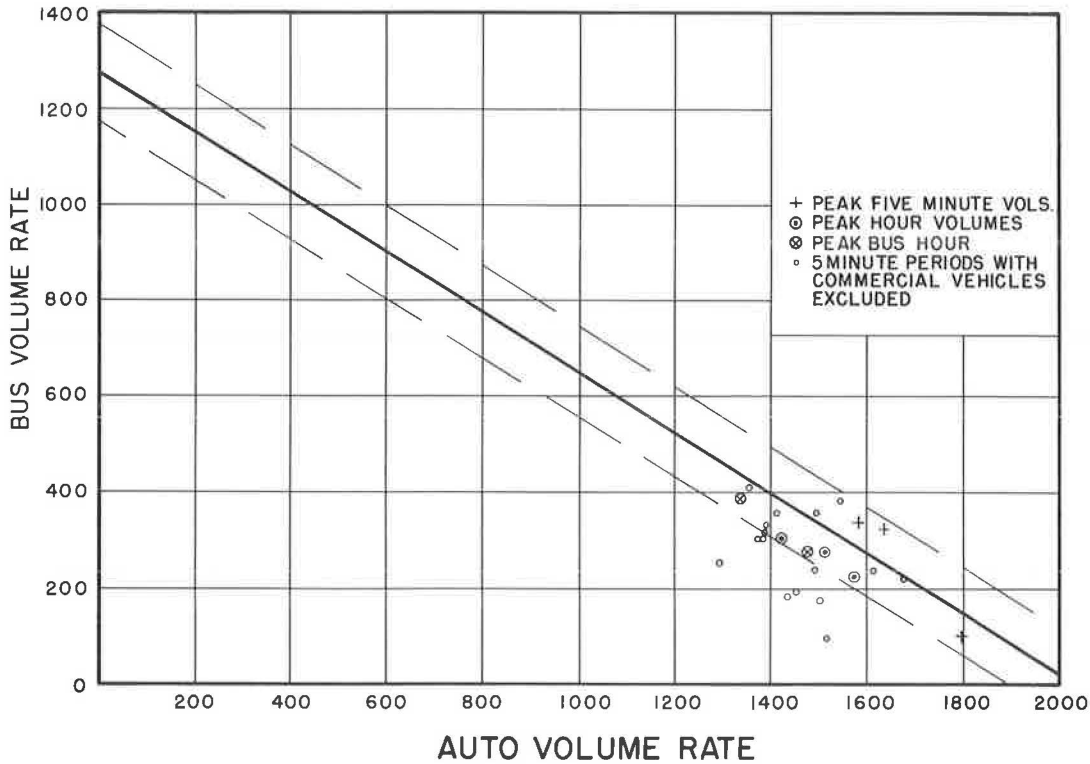

Figure 10. Auto vs bus volume, 6-lane divided, lane 3—25 mph.

dotted lines represent the ±150-vph limit set for the 95 percent confidence interval. The peak period values have also been added to these graphs for purposes of comparison.

For example, in Figure 10, the graph for lane 3 of a 6-lane-divided highway in the 25-mph average speed category (i.e., the 23- to 27-mph speed range), the solid line indicates that when the lane is operating at capacity it can carry 1,200 autos/hr plus about 530 buses/hr for a total of 1,730 vph. This can be calculated using the average cluster value previously derived for autos (2,050 autos/hr) and the bus equivalency factor of 1.6 for a traffic flow consisting of 30.7 percent buses and 69.3 percent automobiles. Similarly, every auto volume shown on the abscissa has a complementary bus volume shown on the ordinate axis. The two volumes added together give the total vehicles per hour (autos and buses) that can be carried by lane 3 of a 6-lane-divided highway at 25 mph. The points plotted on Figure 10 include two observed bus peak hour volumes, 3 peak hour volumes (where the peak was determined by the total number of vehicles—i.e., buses, autos, and trucks—and not by the bus peak hour), and several 5-min expanded volumes. It will be noted that all of the observed peak period volumes fall within the 95 percent confidence limits; none of the observed volumes exceeds the upper confidence limit; a scattering of 5-min periods fall below the lower confidence limit indicating that these particular 5-min periods were not carrying volumes approaching possible capacity. It should be noted that the slope of the theoretical capacity line appears to be correct, although for this graph the observed peak hour periods seem to indicate that possible capacity of this lane should be reduced from 2,050 autos/hr to 2,000 autos/hr at 25 mph. All observed volumes have been adjusted to 12-ft lanes with no lateral restrictions; the observed peak period volumes include some trucks which have been converted to equivalent autos as described previously.

Figures 6 through 10 are fairly representative of the speed range volume graphs which were plotted. However, in the higher speed ranges a large number of the random non-peak 5-min volumes which were plotted were below the lower limit of the confidence interval. In almost all of the cases the volumes were so low that it was obvious the roadway was not carrying close to possible capacity at that particular time. The number of observed volumes higher than the upper limit of the 95 percent confidence interval was no more than could be reasonably expected.

SUMMARY AND CONCLUSIONS

The following results have been derived from this study:

1. The bus equivalency factor for a bus on a reasonably level freeway may be taken as 1.6 regardless of which lane it is in or the speed at which it is traveling.
2. It is possible to determine the capacity of a lane through use of cluster analysis. The term capacity is used here as the maximum volume within a speed range.
3. The possible capacity of an exclusive bus lane is 1,300 buses/hr per lane at 25 and 30 mph. Actually, on a multilane highway, if lane 1 (the right shoulder lane) is designated as the separate bus lane, the possible capacity would be 1,300 buses/hr at 25 mph, if lane 2 is chosen, possible capacity is 1,300 buses/hr at 25 mph, and, if lane 3 is selected, possible capacity is 1,450 buses/hr at 35 mph.
4. Operative volumes may be taken as approximately 80 percent of the speed range capacities as derived from the cluster analysis.
5. The maximum number of buses actually observed on any of the roadways studied in a 1-hr period was 527 buses/hr in the center, two-lane, tube of the Lincoln Tunnel. At the same time, the same roadway carried 1,882 automobiles and 5 tractor-semi-trailer trucks.

This study indicated that, in some instances, it may prove desirable to designate exclusive bus lanes on freeways. However, even if the maximum number of buses observed in this study on any roadway in the country (i.e., 527 buses/hr in the center tube of the Lincoln Tunnel) were put on a single lane, that lane would still appear capable of carrying at least 800 more automobiles with relatively little speed reduction in level terrain. In the author's opinion, it would seem, therefore, that an exclusive, continuous bus lane on a freeway would not prove practical unless:

1. The freeway was already operating beyond its practical capacity to the point of congestion.
2. There were at least 200 buses/hr using the lane during peak periods.

Where adverse grades exist, of course, bus climbing lanes might be desirable under other conditions.

This volume of 200 buses/hr appears to be a reasonable, lower-volume limit because on the typical 6-lane-divided freeway 200 buses/hr can carry at least 10,000 people/hr, more than equal to the capabilities of automobiles in two adjacent lanes, which would in all likelihood carry less than 8,000 people/hr. Such criteria, which would apply only to peak periods, would insure that a majority of the people using the freeway would not be affected by the congestion. (Vehicle occupancy counts were not made during this study, but it was noted that most of the buses observed carried a large percentage of standing passengers; with such loading, the percentage of commuters avoiding congestion would be even greater.) Strict enforcement, or physical separation to keep autos from traveling in the bus lane, would be required.

ACKNOWLEDGMENTS

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Improving Traffic Flow at Transfer Roadways On Collector-Distributor Type Expressways

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•THE Dan Ryan Expressway on Chicago's south side features a 5-mi section of four depressed traveled ways, two in each direction, with a total cross-section of either 12 or 14 traffic lanes. The two inner roadways function as express routes for through traffic; the two outer roadways serve as continuous collector-distributor roads for more localized traffic. Crossover connections from express lanes to local lanes, or vice versa, are provided by 2-lane transfer roadways at three locations for each direction of travel.

Inasmuch as the design feature of continous collector-distributor roadways is a relatively recent development, an analysis of traffic operations at transfer roadways seemed appropriate. An initial operational study at two transfer roadway locations (1) recommended several control measures for improving the efficiency of traffic flow at transfer roadways. This presentation concerns the study of operational characteristics at an express-to-local transfer roadway and compares "before" and "after" pavement markings.

Definitions

Certain terms used throughout this paper are defined as follows:

Transfer roadway—a high-type crossover connection between express lanes and collector-distributor lanes, or vice versa.
Express vehicles—vehicles traveling on the express roadway.
Transfer vehicles—vehicles using a transfer roadway.
Local vehicles—vehicles traveling on the collector-distributor roadway.
63rd St. vehicles—vehicles traveling on the expressway destined for 63rd St.
Through vehicles—vehicles destined for points beyond the Chicago Skyway exit.
Local lanes—collector-distributor lanes.

EXPRESSWAY CHARACTERISTICS

Route Description

The Dan Ryan Expressway links the Eisenhower Expressway (I-90) and the Kennedy Expressway (I-94) with the Chicago Skyway (I-94) and the Calumet Expressway (I-90) (Fig. 1). The north terminal of the Dan Ryan Expressway is the Halsted Street interchange, the junction of Dan Ryan, Kennedy and Eisenhower Expressways, located about 1 mi west of the Chicago downtown area. This interchange accommodates an average daily traffic of approximately 300,000 vehicles. The Dan Ryan Expressway carries I-90 between this interchange and the Calumet Expressway, 11.5 mi south, and is designated both I-90 and I-94 between this interchange and the Chicago Skyway, 8.0 mi south. The latter section was opened to traffic on December 15, 1962, and includes 5 mi of continuous collector-distributor roadways at the southern end.

As shown in Figure 2, the continuous collector-distributor (C-D) roadways are used between 27th St. and 65th St. Although through traffic may use either the express roadways or the C-D roadways, weaving is minimized and flow is expedited for vehicles on the express roadways. At the north end of this section local (C-D) and express road-

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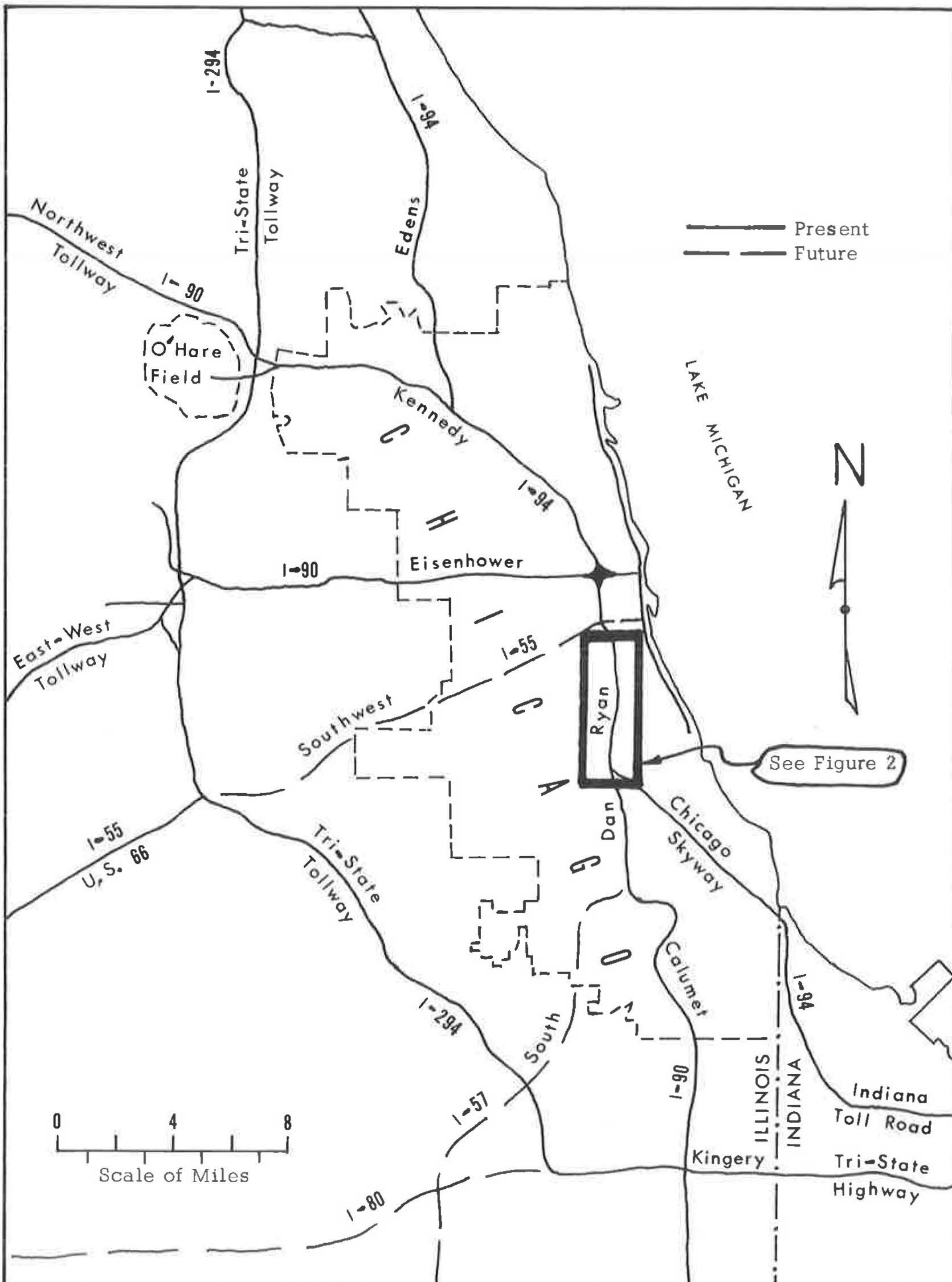

Figure 1. Chicago area expressways.

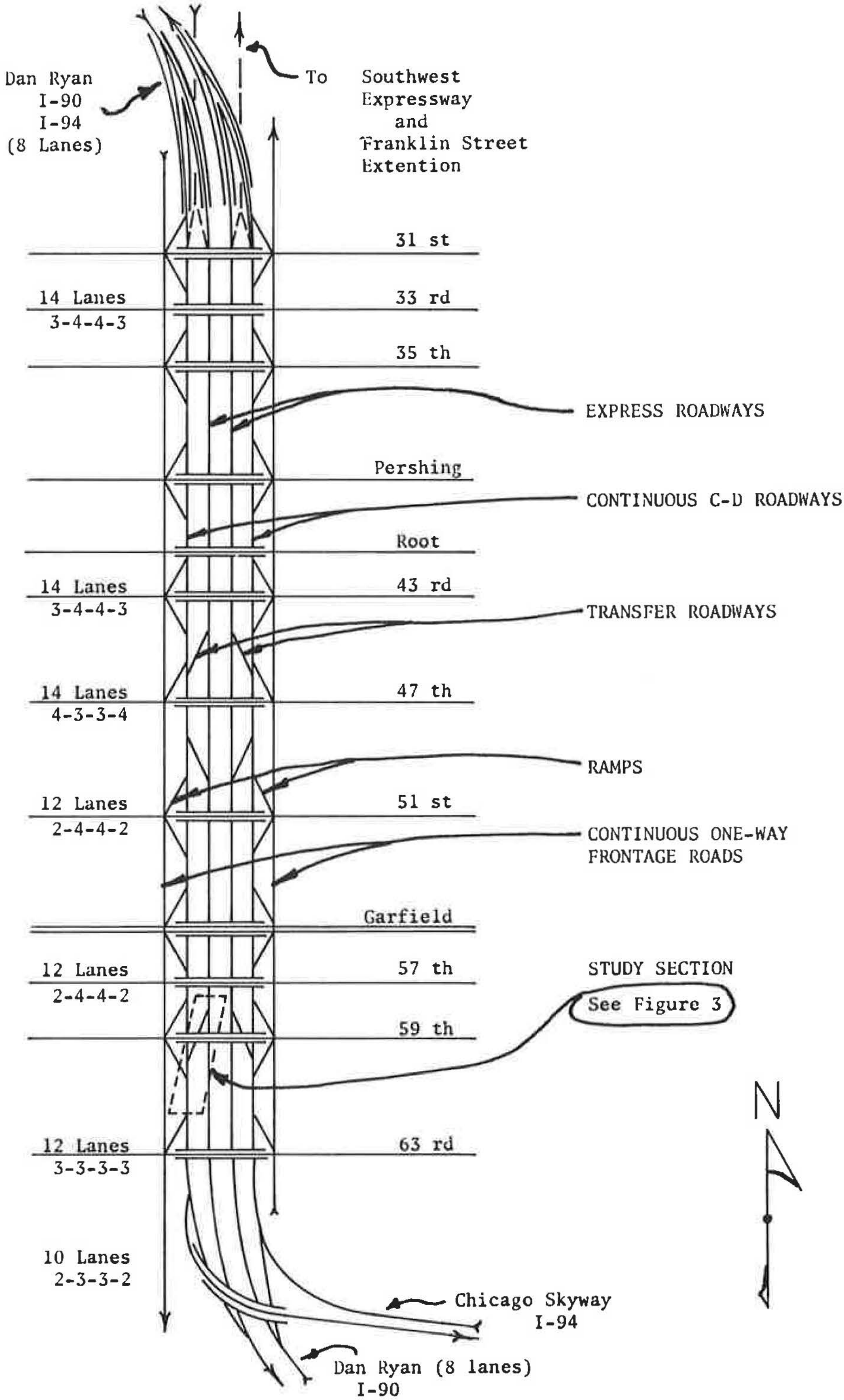

Figure 2. Continuous collector-distributor section, Dan Ryan Expressway.

ways combine on an 8-lane elevated structure. Connection to the future Southwest Expressway and the Franklin Street Extension is provided near 31st St. At the south end of the 5-mi route connection to the Chicago Skyway is provided near 65th St., beyond which local and express lanes combine to form an 8-lane cross-section with only one traveled way for each direction. Thus, the 5-mi continuous C-D roadway section has either 14 or 12 through traffic lanes and connects 8-lane sections at each end.

Route Geometrics

High-type design standards were applied throughout the construction of the four depressed roadways of the Dan Ryan Expressway. Continuous one-way frontage roads feed and collect ramp traffic to and from the continuous C-D roadways at diamond interchanges spaced approximately at 0.5-mi intervals. Continuity prevails along the route because all ramps adjoin the right-hand side of the C-D roadways. A common acceleration-deceleration lane is used where adjacent ramps handle entering and exiting C-D traffic. The only diverging and merging movements on the expressway not requiring right-hand operations occur at the six transfer roadways, which either leave or enter the local roadway on the left side.

Transfer roadways are located for both directions of travel near 45th St., 49th St. and 59th St. All six sites differ either in lane configuration or type of transfer. Figure 2 shows that the cross-section lane configuration changes at each transfer point, inasmuch as one traffic lane is physically "transferred" between roadways. The second lane of a transfer roadway permits optional use of an additional express or local lane at each terminal. An exception occurs at 49th St., however, where the normal cross-section changes from 14 lanes on the north to 12 lanes on the south via the transfer roadways.

All traffic lanes are 12 ft wide and are constructed of continuously reinforced concrete pavement. Contrasting shoulders of bituminous material, generally 8 to 10 ft wide, are furnished on both sides of each roadway. A combination gutter and mountable curb separates shoulders from through pavements. Because most of the route has been designated to accommodate a future mass transit median facility, an unusually wide center median, protected by guardrail along both edges, separates directional flow along the route. Express and local roadways are separated by a 20-ft shoulder-median bisected by continuous, double-faced, steel, barrier-type guardrail. Throughout the whole expressway, mercury vapor luminaires are provided and directional signs are externally illuminated.

Operational Controls

Soon after the Dan Ryan Expressway was opened to traffic, operational problems resulted in the imposition of commercial vehicle lane-use restrictions. Trucks are confined to the collector-distributor roadways, where speed limits are 45 mph for cars and buses and 40 mph for trucks. Passenger vehicles on the express roadways are limited to 50 mph, with a posted minimum of 40 mph.

STUDY PROCEDURE

Although the initial operational study (1) was conducted for both transfer roadways at 59th St., pavement marking improvements were fully installed only at the southbound site. This presentation, therefore, contains "before" and "after" findings for the southbound express-to-local transfer roadway at 59th St.

Study Section Details

Figure 3 shows the study area, located approximately 7 mi south of downtown Chicago and 1 mi north of the Chicago Skyway connection. The total cross-section at this site is 12 through traffic lanes and the average daily traffic flow is approximately 180,000 vehicles. This transfer roadway "transfers" one lane of pavement from the express roadway to the local roadway. Thus, the express roadway is reduced from 4 lanes to 3 lanes and the local roadway is increased from 2 lanes to 3 lanes. Vehicles approach-

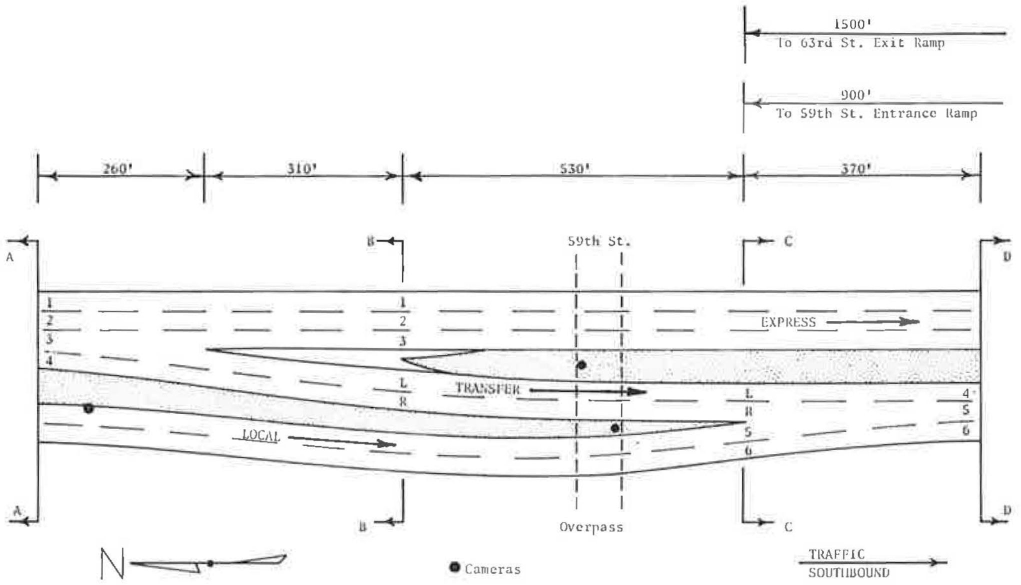

Figure 3. Study section, southbound express-to-local transfer roadway at 59th St.

ing in the right express lane, therefore, are required to transfer; vehicles approaching in the adjacent express lane (second from the right) may either transfer or remain on the express roadway.

The 1,470-ft study section begins at a normal 48-ft express pavement width and ends at a normal 36-ft local cross-section. Observation was confined within these extremes for transfer vehicles (Secs. A-A and D-D); express and local traffic were studied in the diverging (Secs. A-A and B-B) and merging (Secs. C-C and D-D) areas terminating at

Figure 4. Diverging Terminal of study section with "after" pavement markings.

the nose and commencing at the merging end, respectively. Data were obtained from time-lapse films taken simultaneously from both sides of the 59th St. overpass, as shown in Figure 3. Figures 4 and 5 show typical camera views from the overpass positions.

For notation purposes, lanes were numbered from left to right at the four check points (A-A, B-B, C-C, and D-D) indicated in Figure 3. L and R denote left or right lane of the transfer roadway. All vehicles were counted in and out of the study section. Examples of typical vehicle traces through the study limits for express, transfer, and local vehicles are 3-2, 4-R-R-5, and 5-6, respectively.

Individual transfer vehicles were checked outside the study section to determine if they exited at 63rd St. Such a maneuver requires a left-to-right weave within 1,500 ft across local traffic. Although this particular maneuver is not illegal, advance signing advises use of the 45th St. transfer roadway for express traffic desiring to exit at the 63rd St. ramp.

Signing on the express roadway for the study section directs only "CHICAGO SKY-WAY (I-94 EAST)" traffic into the transfer roadway from express lane 4 and optionally from express lane 3. "THRU TRAFFIC (I-90 EAST)" is advised to remain on the express roadway in express lanes 1, 2, or 3, although transfer to the C-D roadway also allows through travel. Overhead directional signing is provided at the nose, and 0.5 mi and 1.8 mi in advance of the nose. Although the study site is situated in a level tangent location, a slight shift due to extra pavement and median introduces some horizontal curvature.

The analysis of the study section was based on the hypothesis that all vehicles in the express roadway, on entering the study section, should be located in the proper lane for executing the desired movements. All transfer vehicles should originate only from express lanes 3 and 4 and terminate only in local lanes 4 and 5. No through express traffic should enter in express lane 4 and no local traffic should encroach on local lane 4. It was further assumed that any violation of the neutral approach nose pavement area constituted a hazardous maneuver, reflecting the quality of traffic operations at the diverging terminal.

Figure 5. Merging terminal of study section with "after" pavement markings.

Field Methods

Two time-lapse cameras equipped with 16-mm color film were used for data collection at the study site. Approximately 65 min of continuous observation per roll of film was produced by exposing one frame per second. Power for the initial study was supplied by a 110-v AC inverter connected to a standard 12-v automobile battery. The expressway lighting circuit was tapped for the "after" study power supply.

Before actual filming operations, reference lines were whitewashed at 50-ft intervals along the shoulder nearest the transfer roadway at the diverging and merging areas of the study section, thus delineating the extremes of the study site and providing intermediate speed traps. A radar speed meter was also used to check speeds in the "after" study.

For the initial study, data films were taken on Wednesday, May 1, 1963, for three complete hours of traffic—1:55-2:55 PM; 3:15-4:15 PM, and 4:30-5:30 PM. For the "after" study, data films were taken on Tuesday, September 10, 1963, for two complete hours of traffic —1:55-2:55 PM and 4:30-5:30 PM. Weather conditions were ideal and traffic conditions were normal throughout all study periods.

Film Analysis

Ten films covering 5 hr of traffic were analyzed for the study section. All laboratory work was performed with a modified commercial projector featuring variable controls and a daylight screen.

Each matching pair of films was coordinated by identifying every transfer vehicle and tracing its path through the four check points of the study section: lane of entry (Sec. A-A); lane at nose (Sec. B-B); lane at merging end (Sec. C-C); and lane of exit (Sec. D-D). Each transfer vehicle was also classified as a "63rd" vehicle if the 63rd St. exit ramp was used.

Both matching films were then analyzed for each 5-min interval; lane volume counts were made of all vehicles at the nose and merging end check stations. These lane volume figures were combined with lane change movements in the terminal areas to produce lane volumes at the extremes of the study section. Using this method, flow values for all movements within the study section were determined for all coverage periods.

Vehicle types were classified only at the 2-lane C-D section located at the merging end (Sec. C-C). Truck volumes were negligible on express and transfer roadways due to the prohibition of such operations. Vehicles having more than four tires were considered heavy trucks; four-tired single-unit trucks of the panel and pickup variety were classified as light trucks.

Hazardous maneuvers were summarized for each hour of data based on violation of the neutral approach nose pavement at the diverging terminal. Two degrees of severity were recorded: crossing the neutral zone with all wheels and straddling this area with either side of wheels.

In the initial study, speeds were checked through a 500-ft trap terminating at the nose and a 400-ft trap commencing at the merging end. Because present traffic demands seldom produce congestion during normal peak flows through the study section, speeds usually are quite uniform. Thus, instead of checking every vehicle, individual speed samples were obtained from each lane throughout each initial study period. The radar speed meter used in the "after" study provided comparable data.

INITIAL OPERATIONAL STUDY

The initial study findings at the express-to-local transfer roadway are based on worn pavement markings, which were limited to painted lane lines that had been applied as a minimum control to allow use of the expressway immediately following pavement construction. Although similar in color and texture to the traveled way, the neutral approach nose pavement lacked delineation other than visible construction joints along the edges of contiguous traffic lanes. Therefore, the "before" markings could not be considered as an effective traffic control measure because of their deteriorated condition.

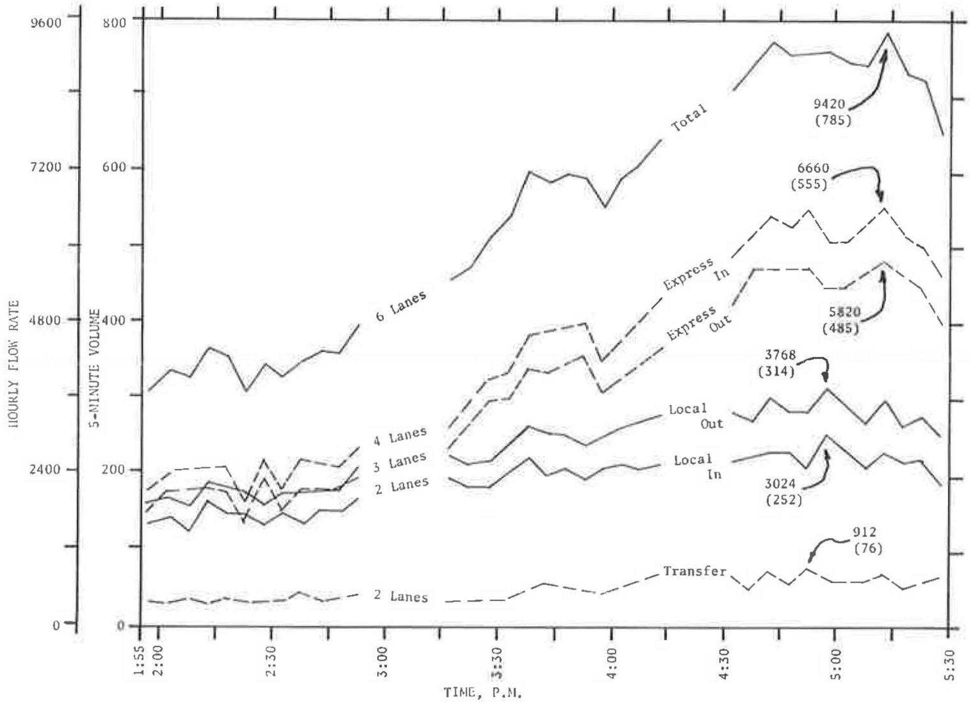

Figure 6. Traffic pattern in study section.

TABLE 1

AVERAGE SPEED BY LANE AND PERIOD

Lane	Average Speed (mph)		
	Off-Peak 1:55-2:55 PM	Transition 3:15-4:15 PM	Peak 4:30-5:30 PM
	(a) Section B-B, Approach Nose		
1	52	54	48
2	51	53	47
3	49	53	48
L	48	51	48
R	48	51	47
(Sample size)	(44)	(53)	(57)
	(b) Section C-C, Merging End		
L	50	50	48
R	48	46	44
5	45	44	42
6	41	42	40
(Sample size)	(44)	(55)	(72)

Traffic Pattern

Figure 6 shows actual 5-min roadway volumes plotted against clock time for the three initial coverage periods. The peak 5-min flow rates are also depicted; the peak 5-min lane volume of 180 (2,160 vph) occurred in express lane 1.

From the total trend displayed, the hour periods are classified as: off-peak (1:55-2:55 PM); transition (3:15-4:15 PM); and peak (4:30-5:30 PM). Roadway volumes at the various check points indicate that the bulk of the volume increase occurs in the express lanes, whereas transfer traffic values are relatively low.

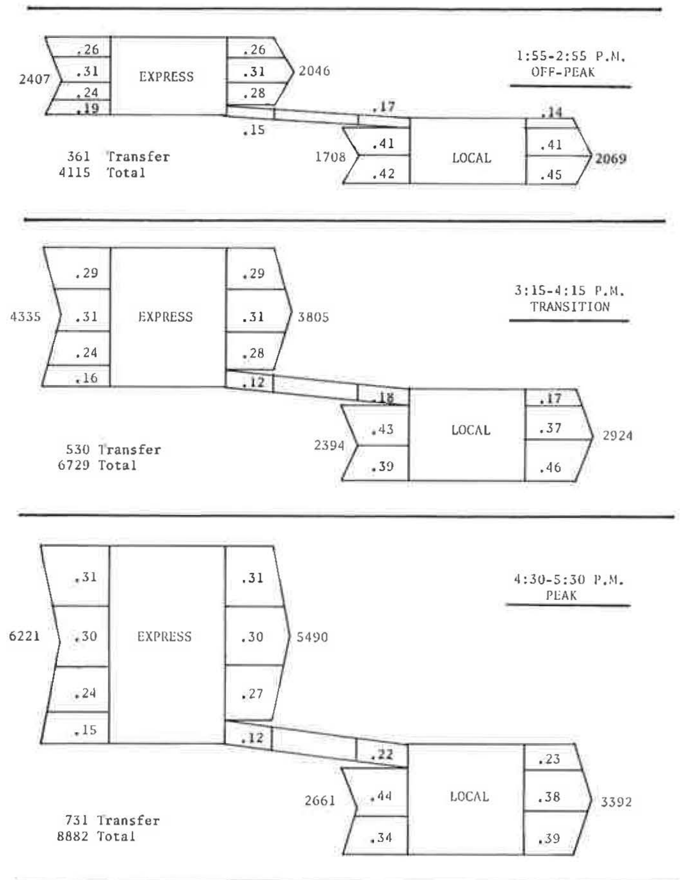

Figure 7. Roadway volumes and lane distributions, by period.

Speeds

Table 1 gives the general range of lane speeds for each hour of analysis, merely verifying operations as "free-flowing." These values seem reasonable in view of the differential speed limits. Only a few individual vehicles deviated greatly from these averages, usually for maneuvers requiring speed reductions.

Flow Diagrams

Figure 7 summarizes the data of Figure 6 into hourly roadway volumes and also presents lane volume distributions. Figures 8, 9 and 10 show volumes for all movements observed in the 3 hr of initial analysis, as well as vehicle classifications.

These four figures indicate that express lanes 1 and 2 are not apparently influenced by transfer operations and express lane 4 feeds many non-transfer vehicles into the diverging terminal. Optional transfers from express lane 3 are minor; transfers from express lanes 1 and 2 are relatively insignificant. The movement of vehicles avoiding compulsory transfer from express lane 4 imposes weaving conditions with optional transfers and merging conditions with through express vehicles.

At the merging end, a substantial number of local vehicles shift into local lane 4 and many transfer vehicles leave the study section via local lane 6. These weaves were performed despite local traffic of high truck concentration.

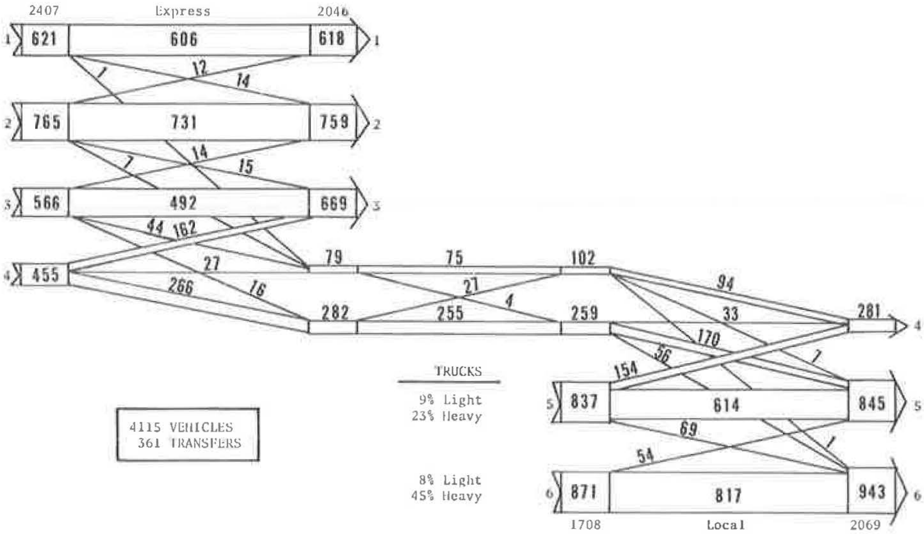

Figure 8. Flow diagram for off-peak hour, 1:55-2:55 PM.

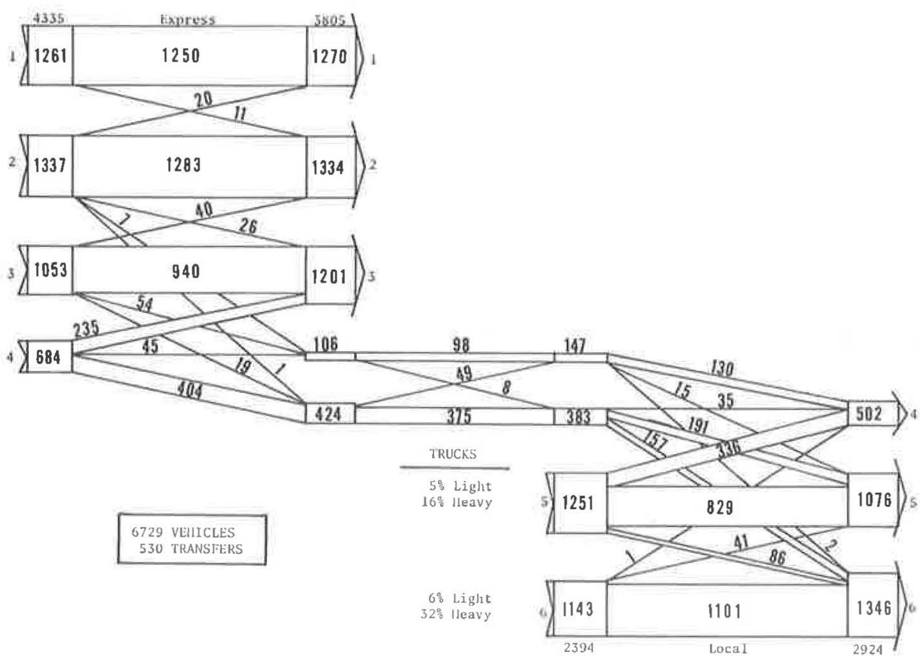

Figure 9. Flow diagram for transitional hour, 3:15-4:15 PM.

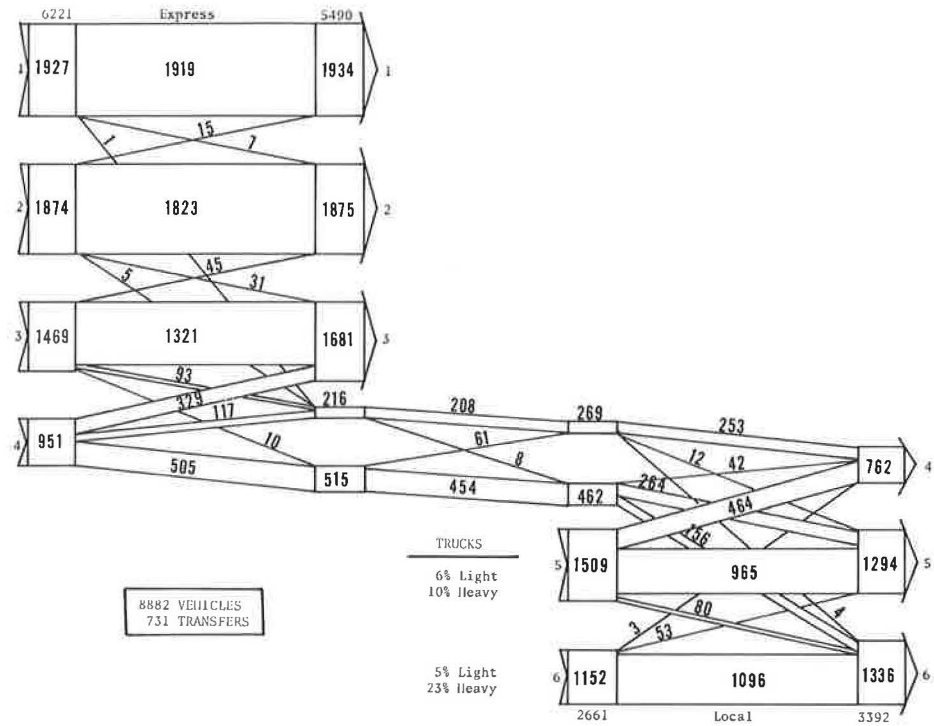

Figure 10. Flow diagram for peak hour, 4:30-5:30 PM.

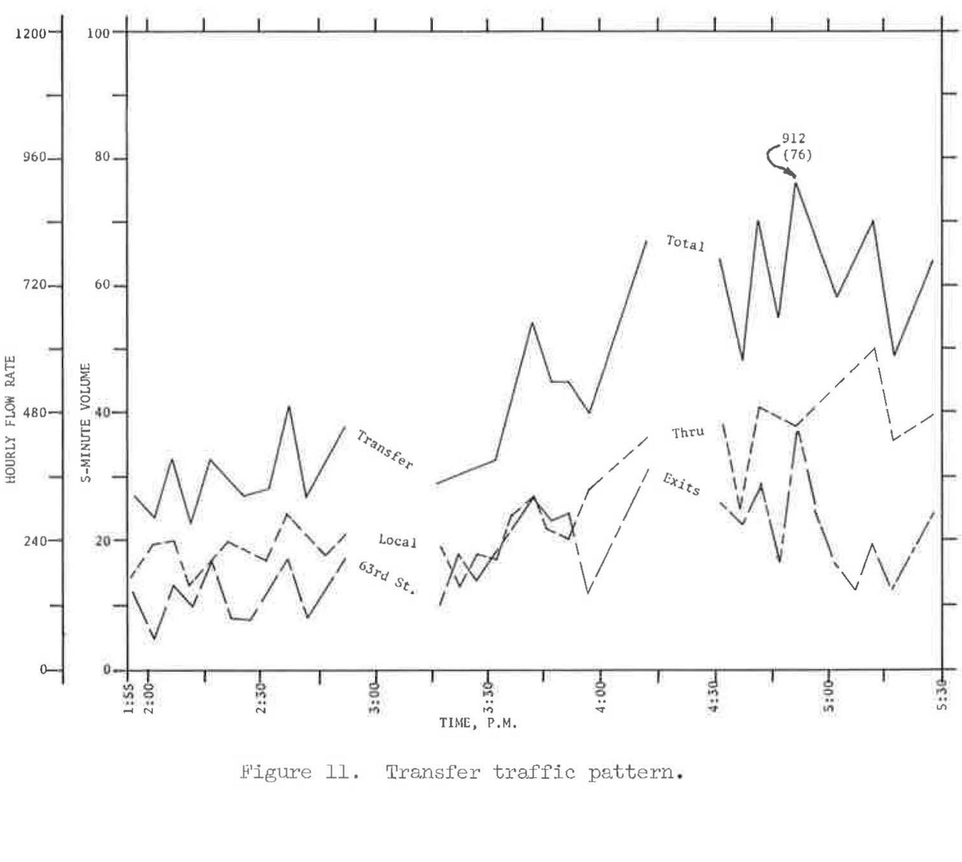

Figure 11. Transfer traffic pattern.

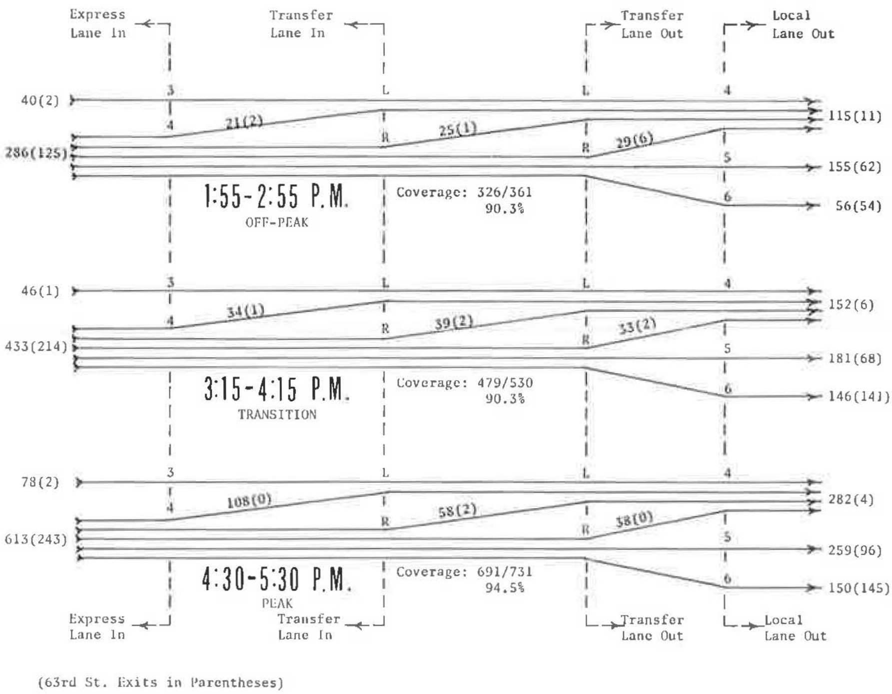

Figure 12. Transfer paths.

Transfer Traffic

Figure 11 shows the traffic pattern of transfer vehicles during the initial study. A large number of transfer vehicles do not conform to the advisory signing, but exiting for 63rd St. via this transfer roadway. For the three periods 63rd St. exits accounted for 39, 45 and 36 percent of all transfer vehicles as total volumes increased. The maximum flow rate for total transfer traffic during a 5-min period was 912 vph. Only three heavy trucks were observed on the transfer roadway during three hours of study.

Figure 12 presents volumes for vehicle paths through the transfer roadway. The coverage figures indicate the proportion of total transfer vehicles represented by paths of at least 20 vph, as shown. Although express lane 4 originates most of the transfer traffic, many vehicles shift to the left lane of the transfer roadway, particularly in the peak hour, so as to enter local lane 4 freely. Vehicles destined for 63rd St. generally keep to the right, thus accounting for nearly all shifts from the transfer roadway into local lane 6.

Hazardous Maneuvers

Figure 13 summarizes the hazardous maneuvers observed at the diverging terminal, as based on the criterion of neutral approach nose pavement violation. Although hazardous maneuvers increase with increasing total volumes, the percentage of these undesirable movements decreases. The magnitude of hazardous maneuvers, mostly of the more severe type in which all wheels cross the neutral zone, reflects the major problem of vehicles not transferring from express lane 4. Total violations for this movement comprised a nearly constant 35 percent of all express lane 4 traffic for the whole initial study period. On the other hand, fewer than 5 percent of all transfer vehicles executed hazardous maneuvers at the diverging terminal in each hour. An average of only 12 percent of the transfer hazardous maneuvers were destined for 63rd Street.

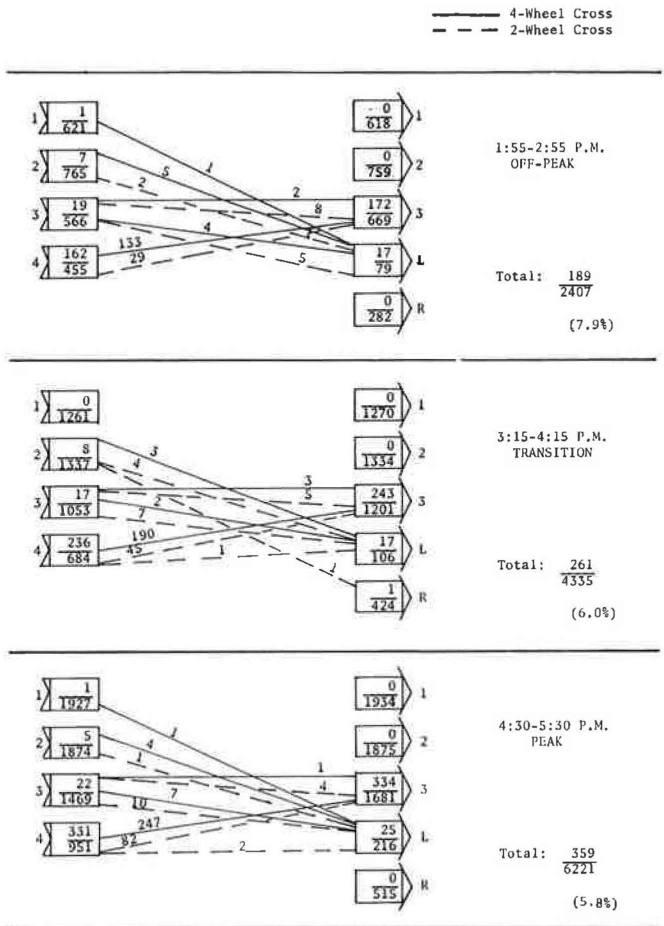

Figure 13. Hazardous maneuvers at diverging terminal.

Many hazardous maneuvers from express lane 4 into express lane 3 required forced entry into normally unacceptable gaps, especially during peak flow. Lane 3 vehicles affected by these movements usually shifted to lane 2 or reduced speed. These events occurred beyond the study limits in the area contiguous to the elongated recovery taper and did not affect more than one or two individual vehicles at a time. Hence, these "merges" did not cause speed reductions and lane changes within the actual study boundaries.

DISCUSSION OF INITIAL FINDINGS

The initial operational study of the southbound express-to-local transfer roadway at 59th St. indicated that a satisfactory level of service was provided for all lanes for the range of volumes normally encountered. Closer study of other factors, however, suggested that the efficiency of traffic operations could be improved by eliminating certain undesirable lane changes at each transfer roadway terminal. Although accident records do not reflect particular problems at transfer roadway locations, the observed maneuvers reveal a definite accident potential.

The movements of major concern, as shown in Figure 14, are discussed hereafter: transfer vehicles shifting into local lane 6 and local vehicles encroaching into local lane 4 at the merging end; and non-transfer vehicles originating from express lane 4

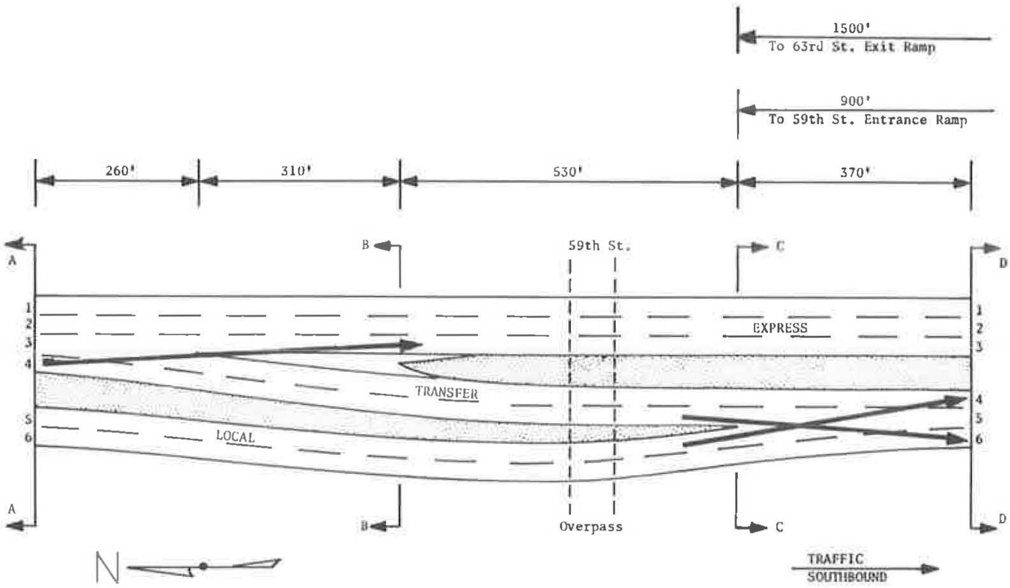

Figure 14. Operational problems at study area.

in the diverging area. It is highly conceivable that these particular maneuvers would have a greater influence on the level of service if the expressway demand were increased at this location.

The Merging Terminal

The effect of the 63rd St. exit ramp on traffic operations is reflected by the magnitude of transfer vehicles weaving for this destination. Disregard for advisory signing suggests that familiar drivers probably perform this maneuver, which is often characterized by an urgent shift immediately from the transfer roadway into local lane 6, despite relatively high volumes of heavy truck composition on the local lanes. The AASHO Urban Policy (2) recommends for this type of design: "Weaving distances... should be 2,000 ft or more and not less than 1,500 ft." The physical weaving distance at this location is 1,500 ft, merging end to nose, but this length includes merging, weaving, and ramp diverging areas, as well as auxiliary weaving pavement shared by the 63rd St. exit ramp as a common acceleration-deceleration lane with the 59th St. entrance ramp.

Evidence exists to warrant prohibition of 63rd St. exits by vehicles using the transfer roadway at 59th St. Although such enforcement would help to minimize conflicts at the merging terminal of the transfer roadway, transfer volumes would decrease at the study site and increase at the 45th St. express-to-local transfer roadway. Further research would be needed at upstream transfer roadways and C-D sections to assure no problems occur from shifting all 63rd express vehicles into the C-D roadway at 45th St.

Due to the lane balance problem at each terminal from physically "transferring" one complete traffic lane, the low transfer demand imposes correspondingly low volumes in the "free" lane at each extreme of the transfer roadway. This relatively empty lane attracts traffic from adjacent lanes at each terminal, particularly at the merging end where local vehicles seek relief from heavy truck concentrations by shifting into local lane 4. Restricting transfer exit to one lane at this terminal would alter the encroaching movement to a normal lane change beyond the study section and eliminate the merging operations now present. Obviously one-lane channelization would increase the difficulty of transfer vehicles exiting at 63rd St. and could not be safely justified without the prohibition of this weave.

The Diverging Terminal

The "attraction theory" also explains in part why many non-transfer vehicles enter the study area in express lane 4. The magnitude of this movement indicates a definite need to emphatically forewarn drivers of mandatory transfer from that particular lane. The flexibility of design at the nose of the transfer roadway permits, or perhaps even encourages, many hazardous maneuvers. Undoubtedly contributing to the number performed by express lane 4 vehicles is the fear of "exit." Signing advises "CHICAGO SKYWAY" traffic to transfer and "THRU TRAFFIC" to remain on the express roadway. Thus the diverging terminal appears to be an exit, although transfer to the C-D roadway allows through traffic to continue on the expressway via the local lanes.

It is quite likely that the increase of right-to-left lane shifts within the transfer roadway in the peak hour can be attributed to familiar drivers utilizing the relatively vacant transfer roadway for through travel. Most vehicles performing this operation remained in local lane 4 for a considerable distance beyond the study section; Chicago Skyway and 63rd St. traffic must perform a left-to-right weave to exit from the expressway. Although actual counts were unobtainable from the films, very little weaving was noticed for transfer vehicles suspected as through traffic.

If some express traffic were encouraged by signing to transfer from express lane 4 to local lane 4 as through vehicles, transfer roadway volumes would increase and undesirable movements at the nose would decrease. Under heavy express flows it might be feasible to relieve the pressure of four lanes reducing to three lanes by using the transfer roadway and local lane 4 for express travel (3). Although this scheme of through access via the transfer roadway would improve traffic operations in the vicinity of the transfer facility, further research would be needed at downstream locations to determine the ability of the C-D roadway to handle increased volumes, especially in the 2-lane section just beyond the Chicago Skyway direct exit.

PAVEMENT MARKING STUDY

Because the lack of sufficient pavement markings obviously contributed to the undesirable movements in the transfer roadway diverging area, new pavement markings were installed and the transfer roadway was restudied to evaluate the effectiveness of the markings as a traffic control measure.

Figure 15 shows the locations of thermoplastic pavement marking improvements in the diverging area. As part of the standard State of Illinois expressway striping program (4), traffic lanes were delineated by broken, white, 5-in. lane lines having 15-ft line segments and 25-ft gaps. In addition, an 8-in. solid, white, channelizing line was installed to isolate the neutral approach nose pavement. These standard markings can be seen in Figures 4 and 5.

Based on the initial operational findings, special markings were applied to further improve operations in the diverging area. A solid, white, 5-in. channelizing line was placed to indicate compulsory transfer from express lane 4. Advance warning in this lane was provided by white, right-arrows elongated to 8 ft and accompanied by "ONLY" legends in 8-ft elongated letters. Express lane 3 was furnished with white, straight-through-and-right-arrows elongated to 8 ft, thus showing optional movements permitted from this lane.

These pavement marking improvements were the only physical changes made in the study area between the initial operational analysis in May 1963 and the "after" study in September 1963. Inasmuch as the improved pavement markings were installed in August 1963, the short time of service which the new markings experienced prior to the "after" study adds further impact to the results obtained. The procedures of the initial study were repeated in the "after" study for the same off-peak and peak time periods.

Traffic Pattern

Traffic volumes in the "before" and "after" study periods are fairly comparable (Fig. 16). The rate of vehicle entry into the study section, however, is slightly higher during the "before" peak hour, due to higher express roadway inputs.

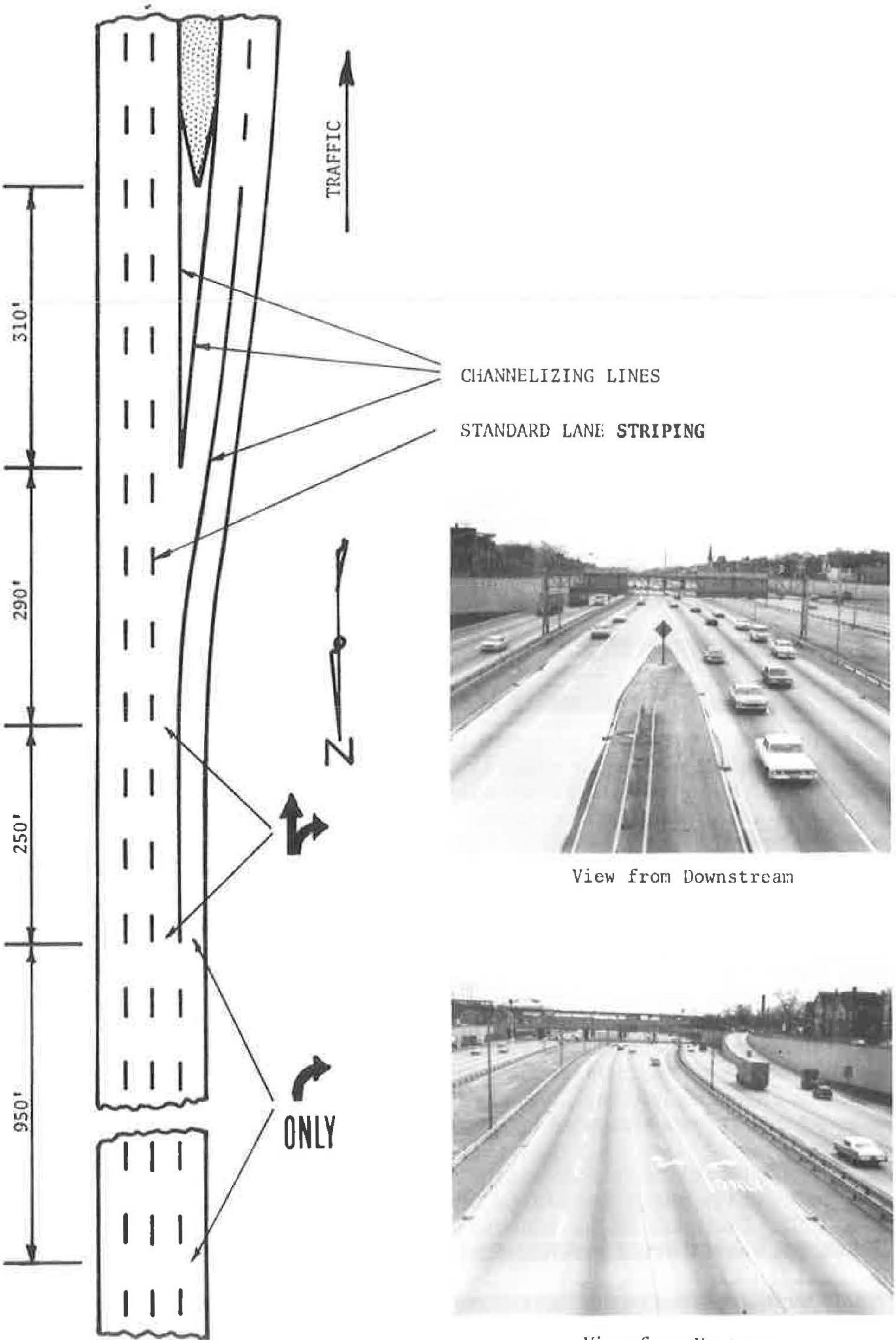

Figure 15. Pavement markings at transfer roadway approach and diverging terminal.

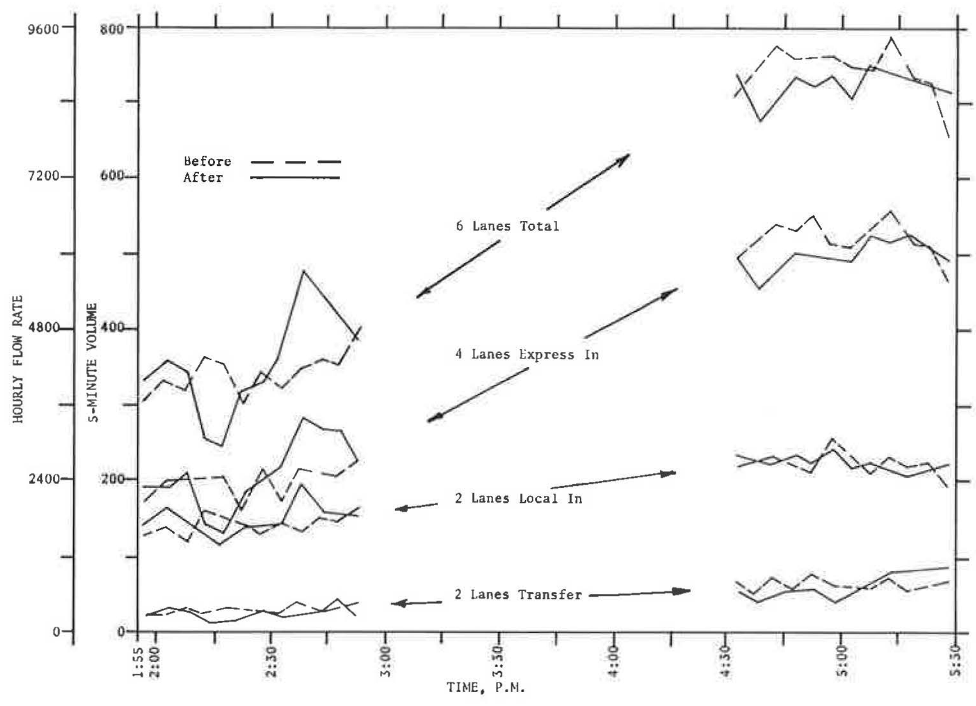

Figure 16. Comparative before and after traffic patterns.

Speeds

In general, the radar speed meter indicated that speeds approximated the "before" averages. There was a decrease in average speeds in the express lanes after 5:00 PM, but the average speeds for each hour remained over 40 mph for all lanes.

Flow Diagrams

Figure 17 shows hourly roadway volumes and lane volume distributions, with "before" results indicated in parentheses. Figures 18 and 19 display "after" movements and vehicle classifications for the off-peak and peak hours, respectively. Significant changes between data in these three figures and the "before" findings are summarized in Table 2.

The operational movements in the "after" periods were basically comparable with the "before" movements except for the two right express lanes affected by the improved pavement markings. Another obvious difference was the over-all reduction in lane changes at both transfer roadway terminals.

Table 2 indicates that the combined inputs of express lanes 3 and 4 were practically identical "before" and "after" for both time periods. Essentially, express lane 4 approach volumes were decreased and express lane 3 volumes increased an equivalent amount.

For the off-peak period the 46 percent reduction (162 to 88) of express lane 4 vehicles avoiding transfer can be attributed mostly to vehicles shifting to the through roadway in advance of the study section. The decrease in transfer roadway volume with an increase in express roadway input, however, suggests that some transfer traffic may have been similarly shifted to the through express lanes. Perhaps the "before" transfer volume included some unfamiliar motorists who desired through travel but became trapped on the transfer roadway under the former transfer approach pattern.

For the peak period, the 56 percent reduction (329 to 144) of express lane 4 vehicles avoiding transfer can be attributed partly to vehicles shifting to the through express

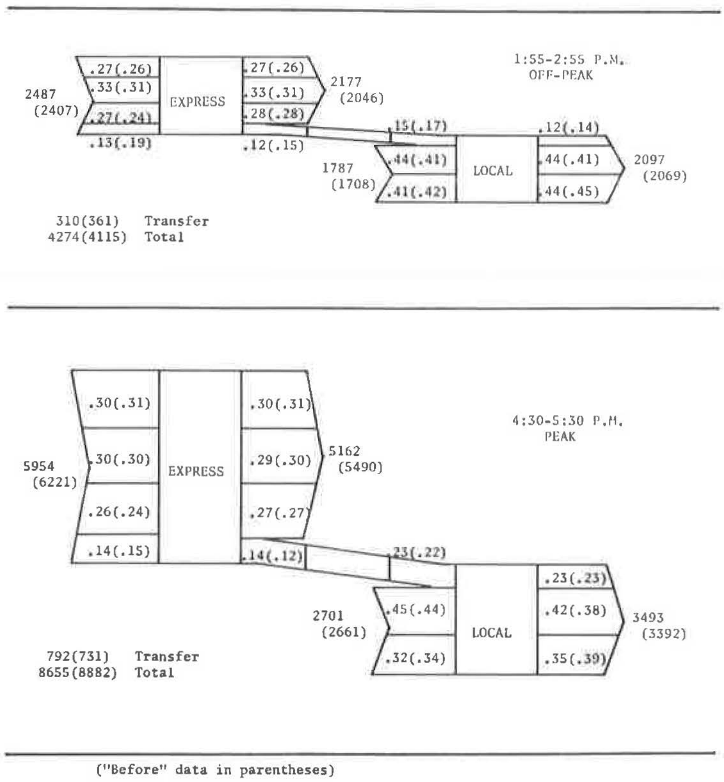

Figure 17. Comparative after and before roadway volumes and lane distributions.

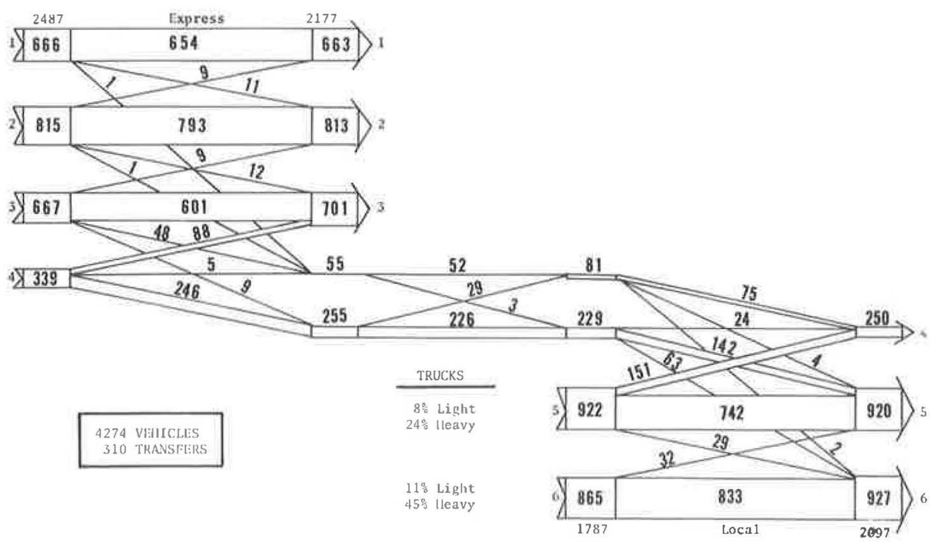

Figure 18. "After" flow diagram for off-peak hour, 1:55-2:55 PM.

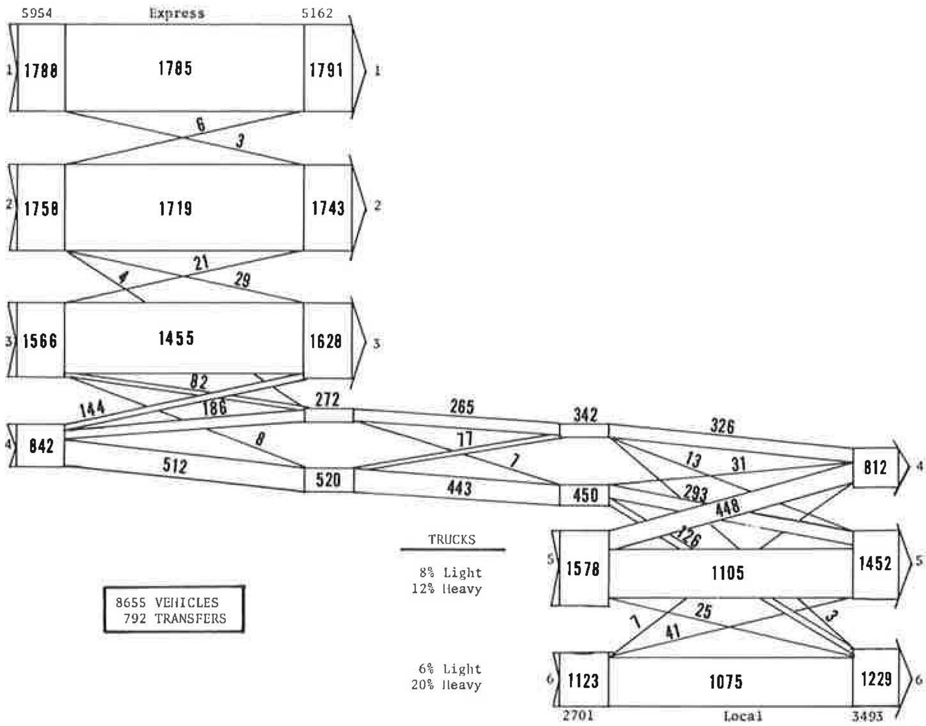

Figure 19. "After" flow diagram for peak hour, 4:30-5:30 PM.

TABLE 2
LANE TRAFFIC VOLUME COMPARISONS

Lanes	Traffic Volume (vph)					
	Off-Peak, 1:55-2:55 PM			Off-Peak, 4:30-5:30 PM		
	Before	After	Change	Before	After	Change
6-Lane total	4115	4274	+159	8882	8655	- 227
Express in	2407	2487	+ 80	6221	5954	- 267
3 and 4 in	1021	1006	- 15	2420	2408	- 12
3 In	566	667	+101	1469	1566	+ 97
4 In	455	339	- 116	951	842	- 109
Transfer	361	310	- 51	731	792	+ 61
4 In 3 out	162	88	- 74	329	144	- 185

roadway in advance of the study section and partly to vehicles shifting to the transfer roadway as through traffic. This latter movement, probably performed by familiar users, is suspected because transfer roadway volumes increased, although the express roadway input decreased.

Transfer Traffic

Figure 20 depicts the pattern of "before" and "after" transfer traffic volumes for off-peak and peak periods. The higher maximum 5-min flow rate of the "after" data (1,080 vph) is probably due to an increase in transfer roadway use for through travel by familiar drivers. The transfer volume increase occurred after 5:00 PM and coincides with average speed decreases on the express roadway. Thus it is likely that many through vehicles were attracted to the faster moving collector-distributor roadway via the transfer roadway.

Observations in both study periods revealed continued motorist lack of response to the advisory signing which designates the express-to-local transfer roadway at 45th St. for traffic exiting from the expressway at 63rd St. The posted regulation restricting commercial vehicles to the local lanes was again well observed, as only four trucks were express roadway travelers in the two "after" hours.

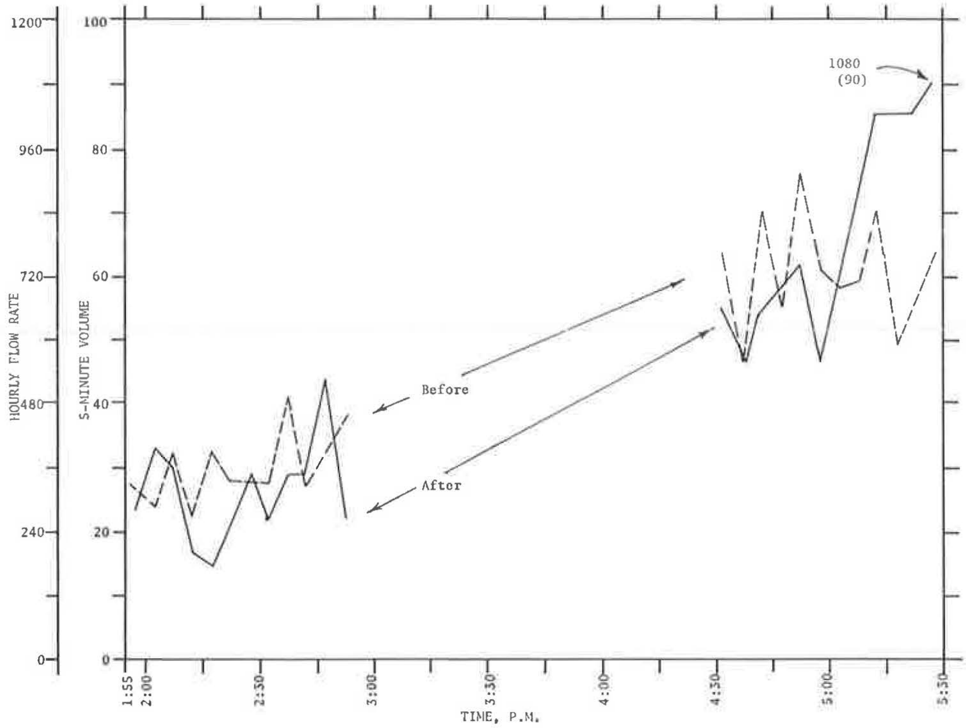

Figure 20. "After" transfer traffic pattern.

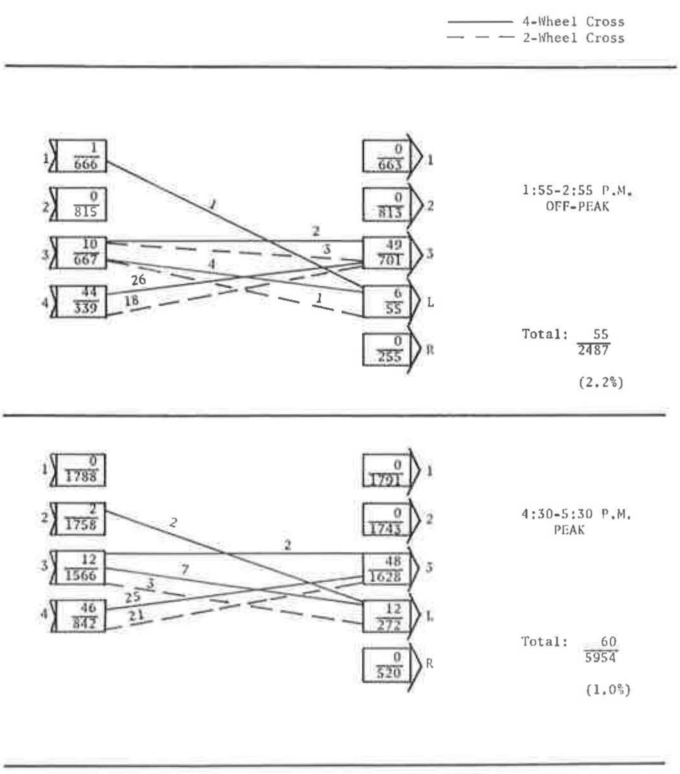

Figure 21. "After" hazardous maneuvers, diverging terminal.

Hazardous Maneuvers

The considerable reduction in hazardous maneuvers at the diverging terminal is shown in Figure 21. The neutral approach nose pavement encroachments were reduced 71 percent (189 to 55) in the off-peak hour and 83 percent (359 to 60) in the peak hour. Hazardous maneuvers of the more severe type, in which all wheels cross the neutral zone, were reduced 77 percent (145 to 33) in the off-peak hour and 86 percent (260 to 36) in the peak hour. Once again the movement from express lane 4 to express lane 3 across the neutral approach nose pavement accounted for the majority of the hazardous maneuvers in the diverging area.

CONCLUSIONS

The operational studies at the express-to-local transfer roadway pointed out various possibilities for improving the efficiency and safety of traffic flow by the elimination or reduction of certain undesirable movements at the merging and diverging terminals.

The effect of pavement markings in reducing hazardous lane changes in the transfer roadway diverging area demonstrated the usefulness of this type of traffic control measure.

Other possibilities for further improving the quality of traffic operations at this transfer roadway location warrant investigation. These include:

1. Geometric changes to allow one-lane operations on the transfer roadway.
2. Advance warning signs for vehicles approaching in the compulsory transfer lane.
3. Signing to inform express traffic of through access via the transfer roadway.
4. Methods to eliminate or reduce weaving at the merging terminal by transfer vehicles exiting at the next downstream ramp.

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Optimization of Freeway Traffic by Ramp Control

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•THE JOHN C. LODGE Freeway Traffic Surveillance Research Project in Detroit, Mich., has as one of its principal objectives the development of a freeway variable traffic control system which will permit a more efficient utilization of this high-type road facility. The intent of the research is to develop and create a control system which will reduce congestion, improve capacity, and provide a safer travel environment. If these benefits are realized, costs savings of considerable proportions will result. These savings could then be compared to the costs of construction which would produce equivalent benefits. If the control system proves to be the most efficient, the money should be placed in this area rather than in expensive construction.

The project has developed three distinct types of variable traffic control on the freeway. One is the lane signal system telling the motorist when, or when not, to use a driving lane; the second is a variable speed control advising the motorist on the proper speed to drive the freeway; and the third, ramp entrance control permitting the opening or closing of a ramp, is the subject of this paper.

RAMP CONTROL SIGNAL

There are nine entrance ramps in the 3-mi study area on the John C. Lodge Freeway. Each of these entrance ramps was placed under the control of a ramp signal bearing the message "Dont Enter" (Fig. 1). This signal is a blankout type signal device that shows the message only when the signal is illuminated.

Preliminary tests, measuring the effectiveness of the ramp signals, were conducted in Detroit in the spring of 1960. An experimental ramp signal installation was made at the Trumbull Avenue entrance ramp to the westbound direction of the Edsel Ford Freeway. The installation consisted of two back-to-back signals mounted on a mast arm so they were visible to both directions of oncoming Trumbull Avenue traffic. The results of this experiment were documented by Richard D. Belprez and Conrad L. Dudek.

In general, the experiment proved that motorists read a signal placed in a visible location. However, this does not imply that they always obey. Several motorists were observed to look intently at the signal and then look around to determine whether the ramp was still usable. If clear, they would then enter the freeway.

The "sheep" effect of people was also noted during this experiment. If one motorist would commit himself to enter the ramp against the signal message, several others would follow. However, if the lead motorist in a platoon would obey the signal, the rest of the motorist did likewise. The fact that the motorist did not always obey the signal was accepted as being quite natural due to the fact that there had been no publicity about the test and no enforcement efforts were made. Unless it can be proved that a ramp can be controlled only by a signal, there is little evidence to support the expense of obtaining devices such as automatic gates, which would cost considerably more than a signal system but yet be cheaper than law enforcement.

Proper mounting of ramp signals plays a very important part in the success of ramp control. Each approach direction to the ramp must have a signal plainly visible to the

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motorist and the ramp must be identified with the signal. An overhead mount is most appropriate for the majority of locations; however, pedestal mounts were used in certain areas for better visibility.

Figure 1. Ramp signal.

DESCRIPTION OF EXPERIMENT

The installation of signals at each entrance ramp provided an opportunity to study the effects of ramp control on freeway traffic by a total closure of a ramp during selected periods of time. No attempt was made to meter ramp traffic by any signal system during this experiment.

A continuing study is being made on the John C. Lodge Freeway, showing volume-speed relationships, by lanes, for various locations. Results indicate that single point measurements of freeway traffic do not permit an accurate prediction of impending breakdown. It is only by considering the downstream nature of the traffic, and comparing it to the traffic being discharged into the area, that a reasonable prediction can be made. For example, a high-volume high-speed platoon of traffic moving into an area of lower speed limit which cannot be raised before the arrival of the higher speed traffic can result in a complete traffic breakdown. This same traffic, moving into an area that is discharging traffic as fast as it is received, will cause no difficulty. If this knowledge is added to the fact that the highest traffic volumes on a freeway are recorded while traffic is moving in a fluid condition, the benefits to be derived from ramp control become apparent.

Experience has proved that fluid levels of traffic can be maintained if no interference occurs in the traffic stream to cause this large volume to break speed and, consequently, reduce capacity. When large volumes of traffic are using the freeway, the average spatial headway between vehicles becomes reduced to a point that, whenever an interference establishes itself in the traffic path, there is not enough elasticity in the traffic stream to permit any alteration in the spatial headway between vehicles without creating a speed reduction on the traffic following. This influence creates the well-known "shock wave."

One of the most prevalent conditions creating this disturbance in the traffic stream, thus breaking vehicle speed, is entrance ramp traffic. A vehicle entering the freeway will merge into a gap between vehicles but will not continue to maintain this close distance while driving on the freeway. As a result, the vehicle will slow down to acquire a greater distance from the preceding vehicle. When the gaps between vehicles reduce to an average of approximately three car lengths, lowering of speeds will result because vehicles back off to obtain more comfortable driving space (1). The purpose of the ramp experiment was to prove that, when headways between vehicles were of the order permitting high traffic volumes at good speeds on the freeway but with no additional capacity to permit ramp traffic to enter, for reasons mentioned, closing ramps during such periods of time would preserve good freeway operation. The theory was that, if higher volumes of traffic could use the freeway in a short period of time, there would be less backlog of traffic in the central business area. This would not only benefit freeway traffic operation but would actually minimize congestion on the arterial streets.

It generally works out, in practice, that the freeway starts loading with traffic from the various entrance ramps in the central business area. As traffic keeps accumulating on the freeway, it reaches the condition where high traffic volumes cannot be maintained because of the lowering of speed. The study area of the John C. Lodge Research Project is in an area adjacent to the central business district, thereby permitting evaluation of the practical application of this reasoning.

Closing freeway ramps to entering traffic created a problem of public reaction which had to be handled carefully to avoid discontinuance of experiments because of lack of public support. It was necessary to carefully select and sign alternate routes by which motorists could travel in a path parallel to the freeway either to their destinations or to a point on the freeway farther downstream where reduction in traffic volumes, due to discharge at exit ramps, permitted the addition of more traffic without creating traffic stoppages. Trail blazer-type signs were used to mark the alternate routes for the motorists not permitted to enter the entrance ramps. These signs advised motorists how to proceed along the alternate route and reach the next entrance to the freeway.

This left the problem of those cases where the second entrance ramp also was closed. A motorist may tolerate finding his first choice of an entrance ramp closed and may follow an alternate route; but, if he is guided into a second closed entrance ramp, he may become quite impatient. If this were repeated often enough, it could create a serious public relations problem. To prevent this difficulty, the trail blazer sign, located at the point where a motorist could be guided to a closed entrance ramp, was designed so the arrows on the sign could be changed manually to direct the motorist farther along the bypass route and eventually to an open entrance ramp.

DESIGN OF STUDY

The initial ramp closure study was conducted during the week of March 4, 1963. There are nine entrance ramps in the study area, as shown in Figure 2. Various combinations of ramps and periods of time at which they would be closed were selected to determine not only the effect of number of vehicles entering the freeway from entrance ramps but also the effects of individual ramps.

The selection of the ramps and combination of ramps to be closed was based on several considerations. Each entrance ramp was scheduled for at least one closure period to gain the most complete experience possible on the effects of various types of entrance ramps on freeway traffic behavior. Peak traffic periods were chosen because these were the periods when ramp traffic would have its most marked effects. Previous studies had indicated locations where trouble was likely to develop during peak travel periods. Ramps at or near these locations were selected for special attention. By noting the effects of the closure of individual ramps, combinations of ramp closures were selected during the course of the experiment.

It was important to the results of the experiment to show the changing traffic characteristics created by ramp closure; therefore, three weeks in advance of the ramp closing experiment, various traffic measurements were taken both on the freeway proper and on surrounding streets that would be used by bypass traffic. Forty-eight locations were studied so that a pattern of traffic movements could be obtained. During these 3 wk, freeway lane stoppages caused by traffic volume congestion were recorded for the peak traffic directions from 7:00 to 9:00 AM and from 3:00 to 6:00 PM. Stoppages due to a stalled vehicle or other incident were recorded separately so they could be properly interpreted in the analysis of the experiment. The data gathered in advance of the experiment were used also to determine the proper placement of directional signs throughout the area in which motorists would be guided along alternate routes to the freeway whenever entrance ramps were closed.

Approximately two-thirds of the length of the freeway in the study area is paralleled by service drives which offer very convenient alternate paths to the next available entrance point to the freeway. In the areas without service drives, motorists were directed to the most accessible arterial street which would carry them to the open entrance ramps. Figure 2 shows these routes. Dispersal of ramp traffic after departing from a closed entrance ramp was carefully noted to determine their new travel paths and the ability of the surface streets to handle the additional travel load.

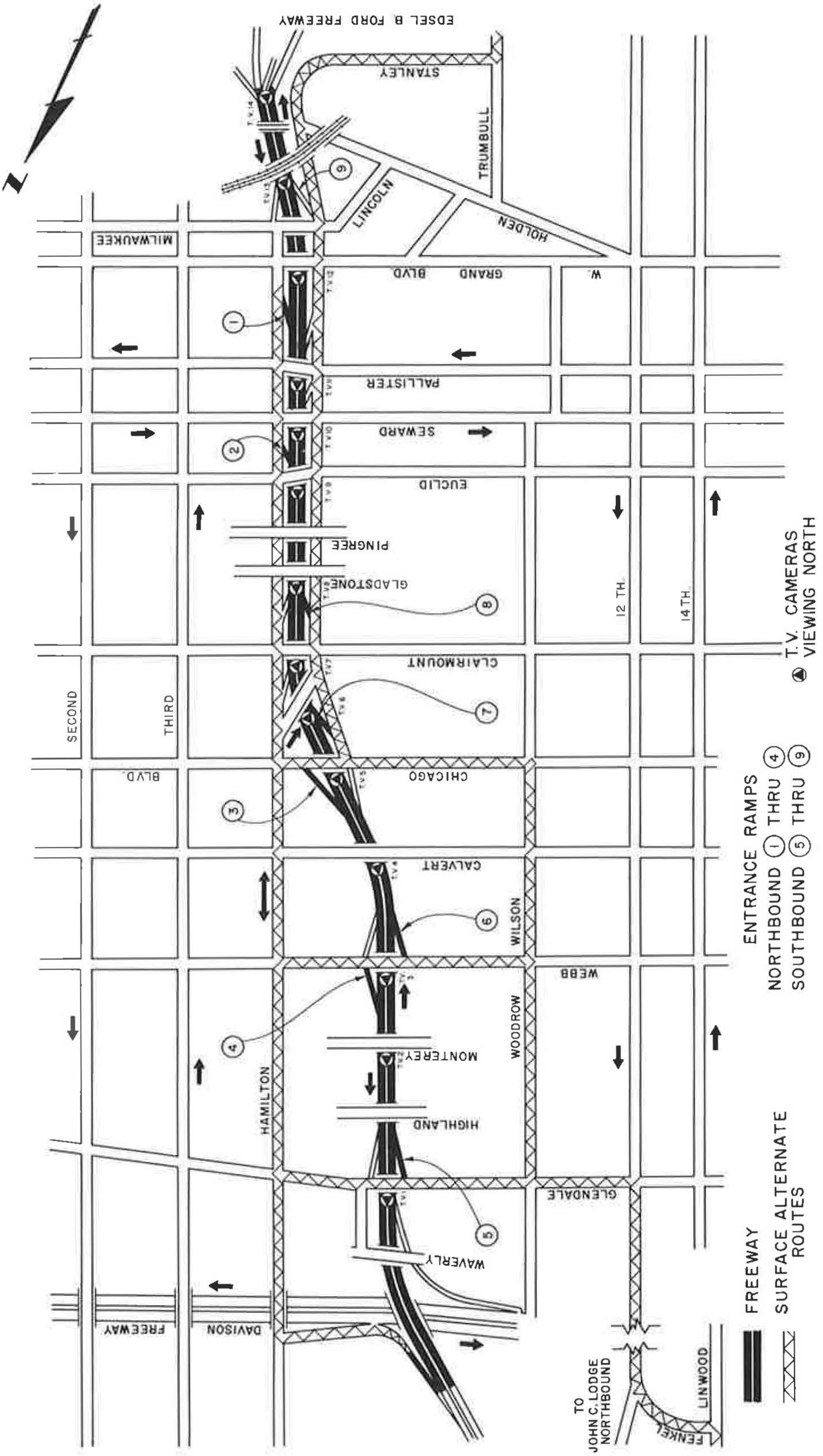

Figure 2. John C. Lodge Freeway on-ramp study area plan showing alternate routes.

To improve obedience to the "Dont Enter" ramp signals, police officers were stationed at the head of the entrance ramps. This permitted measurement of the full effect of positive closure at these various points and the full impact on diverted ramp traffic to the surface street.

Vehicle stoppages on the freeway, along with the travel times of numerous vehicles, were recorded by lane throughout the study area. This information was acquired by stationing observers at television monitors in the TV control center of the Project. Reference lines were placed on the TV monitors to identify the location of the vehicle with respect to the pavement, for the purpose of obtaining travel times. Lane changes were recorded, in several camera views in each direction, covering the areas most critical in their relationship to the ramps being closed.

The first day of the study week was devoted to familiarizing the numerous new men with the performance of various duties toward gathering data for the study. For this reason, data for the first day are not included with the four other days of the study week.

FREEWAY TRAFFIC FLOW ANALYSIS

During the study week, closing of ramps during peak traffic periods substantially benefited freeway traffic movement. The comparison of freeway traffic volumes for the peak travel periods of 7:00 to 9:00 AM and 3:00 to 6:00 PM is very important to the results of the ramp closure evaluation study. Table 1 shows the comparison of 2-hr southbound volumes and 3-hr northbound volumes on the John C. Lodge Freeway in the 3-wk period before ramp closure and the 1-wk period of ramp closure. Southbound traffic was inbound to the central business area of Detroit in the morning and northbound traffic was outbound in the afternoon. These represented the peak travel directions and were used in the comparisons shown in Table 1. Instrumentation limitations prevented comparisons of both travel directions simultaneously but had no influence on the validity of the results. Any unusual incidents occurring in the direction of lighter travel, which could have influenced traffic behavior in the peak travel direction, would have been noted by television surveillance.

An examination of the volumes shows that freeway volumes were, in all instances, higher during the ramp closure week than during the nonclosure periods. These figures are distorted to the disadvantage of ramp closure since, in certain of the closures, some traffic normally using the freeway was bypassing the traffic sensors. A more perfect experiment would have been obtained if the freeway sensors could be placed at the beginning and end of the ramp closure area. Again, equipment limitations did not permit this but it will definitely be considered in future experiments. Southbound traffic volumes increased from 4. 8 to 13. 7 percent, whereas northbound traffic volumes increased from 3. 5 to 5 percent.

Since stoppages on a freeway are definite indications of poor operating conditions, Table 2 was designed to compare stoppages on the freeway, with and without ramp control. For the southbound direction, there was a 22 to 54 percent reduction in stoppages between 7:00 and 9:00 AM. Considering the ramp closure periods only, the reduction in stoppages were even more marked: 26 to 65 percent. The northbound direction has a reduction in lane stoppages ranging from 51. 5 to 92. 5 percent in a comparison of 3-hr periods from 3:00 to 6:00 PM. Comparing ramp closing periods only, the reduction in stoppages, with ramp closure, is 65 to 92. 5 percent.

The stoppages on the freeway were tabulated for the entire length of the freeway in the study area. Under these circumstances, not only the number of stoppages but also the extent of the stoppage is important. One stoppage extending a short distance can be far less critical than one extending a long distance. In Table 3, the average length of travel of stoppage waves in the

TABLE 1

FREEWAY VOLUME COMPARISONS

Day	2-Hr Southbound Volumes 7:00-9:00 AM			3-Hr Northbound Volumes 3:00-6:00 PM		
	Previous 3-Week Avg.	Study Week 3-5-63	% Increase	Previous 3-Week Avg.	Study Week 3-5-63	% Increase
Tues.	10,998	11,521	4. 8	16,490	17,320	5
Wed.	10,807	11,456	6	15,378	15,954	4
Thurs.	9,998	11,365	13. 7	16,248	17,067	5
Fri.	10,845	11,542	6. 5	16,575	17,155	3. 5

TABLE 2

FREEWAY LANE STOPPAGE COMPARISONS[a]

Day	Southbound				Northbound			
	Lane Stoppages 7:00-9:00 AM		% Difference		Lane Stoppages 3:00-6:00 PM		% Difference	
	Previous 3-Week Avg.	Study Week 3-5-63	2-Hr Study Period	Ramp Closure Periods Only	Previous 3-Week Avg.	Study Week 3-5-63	3-Hr Study Period	Ramp Closure Periods Only
Tues.	43.5	34	Down 22	Down 36	67	33	Down 51	Down 65
Wed.	67	31	Down 54	Down 65	114	26	Down 77	Down 82
Thurs.	58	28	Down 52	Down 54	90	9	Down 90	Down 90
Fri.	40.5	30	Down 26	Down 26	94	7	Down 92.5	Down 92.5

[a]Stoppages due to congestion only.

period before ramp closure is compared with that in the period after ramp closure. These reductions were very significant, ranging from 28 to 86 percent with most of the reductions nearer the higher percentage figure.

An interpretation of Tables 2 and 3 indicates that ramp control not only reduced the number of stoppages but also reduced the area of the freeway over which they had influence. This has a twofold benefit producing a traffic stream of much greater fluidity.

An analysis of travel-time data gathered on the freeway gives substance to this last conclusion. Travel time of traffic moving through the study area was taken by observers viewing the television monitors. The results are shown in Table 4. This table shows average travel speed through the study area by time of day and identifies the period during which designated ramps were closed. Although closure of all ramps did not produce positive benefits, the results showed higher average speeds during most ramp-closing periods. To properly interpret Table 4, it is necessary to take a period during which a ramp is closed and compare to the time period best matching it.

TABLE 3

LENGTH OF TRAVEL OF STOPPAGE WAVES COMPARISON

Day	Time Period	Average Length of Travel (ft)		% Reduction
		Before Period	Study Week	
(a) Southbound				
Tues.	7:00-9:00 AM	5,980	1,500	74
Wed.	7:00-9:00 AM	580	350	40
Thurs.	7:00-9:00 AM	2,880	520	82
Fri.	7:00-9:00 AM	4,600	2,880	37
(b) Northbound				
Tues.	3:00-6:00 PM	4,450	3,190	28
	Ramp closure periods only	5,130	690	86
Wed.	3:00-6:00 PM	5,130	3,190	38
	Ramp closure periods only	3,880	1,030	73
Thurs.	3:00-6:00 PM	4,790	1,140	76
	Ramp closure periods only	3,190	570	82
Fri.	3:00-6:00 PM	2,280	800	65
	Ramp closure periods only	2,280	800	65

During the critical period from 7:30 to 8:45 AM, the average travel speed without ramp control was 27 mph; yet in this same period, average travel speeds upward to 42 mph could be obtained. Although direct comparisons of this type are open to question, there can be no doubt that the average travel speed is significantly higher under most choices of ramp closures.

The afternoon period for northbound traffic produces similar results. Here it was possible to obtain even greater improvements in average travel speeds with ramp control. Table 5 gives lane changing data for the northbound direction with (a) all ramps open, (b) West Grand Boulevard ramp closed, and (c) Seward Avenue ramp closed. The data show a definite relationship between entering ramp traffic and lane changing. In making the study of the effects of lane changing by ramp control, all camera fields were not covered due to limitations of personnel and the location of some cameras in

TABLE 4
SPEED COMPARISONS

Ramps Closed				All Ramps Open	
Time and Day	Ramp	Avg. Travel Time Speeds (lanes 1 & 2) (mph)	Speed Range (mph)	Avg. Weekday Periods	Avg. Travel Time Speeds (lanes 1 & 2) (mph)
(a) Southbound (AM)					
7:30-8:00 T	Glendale & Webb	35.4	32.3-42.7	7:00-7:15	39.1
8:15-8:30 T	Glendale	42.0	36.1-45.6	7:15-7:30	33.4
7:30-8:30 W	Glendale & Webb	37.0	31.3-43.1	7:30-7:45	24.4
7:35-7:50 Th	Davison	28.4	27.5-28.9	7:45-8:00	27.6
8:00-8:30 Th	Glendale & Webb	39.6	30.5-48.2	8:00-8:15	26.6
8:30-8:45 Th	Davison	40.7	39.5-42.7	8:15-8:30	28.3
7:15-7:50 F	Glendale	33.8	28.1-43.7	8:30-8:45	28.0
7:50-8:20 F	Glendale & Webb	37.2	34.3-39.5	8:45-9:00	22.9
8:20-8:40 F	Glendale	40.3	38.0-43.7	9:00-9:15	34.3
				9:15-9:30	40.1
		Avg. 37.2			Avg.[a] 25.5
(b) Northbound (PM)					
3:45-4:15 T	W. Grand Blvd.	33.3	29.0-41.0	3:00-3:15	44.2
4:30-5:00 T	Seward & Chicago	41.0	34.0-46.1	3:15-3:30	49.2
5:15-5:30 T	Seward & W. Grand Blvd.	37.5	34.0-41.0	3:30-3:45	36.0
5:30-5:40 T	W. Grand Blvd.	38.9	37.0-41.0	3:45-4:00	27.0
3:30-3:45 W	W. Grand Blvd.	41.2	40.0-42.3	4:00-4:15	28.0
4:00-4:15 W	Chicago & Seward	30.6	28.5-33.9	4:15-4:30	30.7
4:15-5:00 W	Webb, Chicago, & Seward	28.2	26.0-31.7	4:30-4:45	29.0
5:00-5:35 W	Chicago & Seward	36.6	31.7-50.5	4:45-5:00	22.7
3:45-4:00 Th	Chicago & Seward	45.0	38.7-49.7	5:00-5:15	20.5
4:00-5:34 Th	W. Grand Blvd.	44.7	37.0-52.2	5:15-5:30	21.4
3:30-3:56 F	W. Grand Blvd.	38.0	33.7-47.5	5:30-5:45	24.9
3:56-5:11 F	Chicago & W. Grand Blvd.	43.5	37.0-50.8	5:45-6:00	28.8
5:11-5:35 F	W. Grand Blvd.	42.0	36.6-46.8	6:00-6:15	37.2
				6:15-6:30	43.4
		Avg. 38.5			Avg.[a] 25.5

[a]Values in brackets.

TABLE 5
LANE CHANGE RATE COMPARISONS
(Seward and West Grand Blvd. Ramps Closed vs All Ramps Open[a])

Condition (Northbound Direction Only)	Camera Fields								
	Webb 3	Calvert 4	Chicago 5	Total	Gladstone 8	Euclid 9	Seward 10	Pallister 11	Total
All ramps open, lane changes per min	0.4	0.4	0.15	0.95	0.7	1.6	1.7	2.9	6.9
W. Grand Blvd. closed (camera field 12) lane changes per min	0.6	0.8	0.1	1.5	1.2	2.8	1.2	1.3	6.5
Seward Ave. ramp closed (camera field 10) lane changes per min	0.6	1.0	0.4	2.0	0.4	1.2	1.2	3.1	5.9

[a]One week average, 3:00-6:00 PM.

areas of the freeway least likely to be influenced by ramp control. The lane changes per minute, observed in the Webb (No. 1), Calvert (No. 2), and Chicago (No. 3) camera fields, show that fewer lane changes occur in the north area than in the south portion covered by the Gladstone (No. 8), Euclid (No. 9), Seward (No. 10), and Pallister (No. 11) camera fields. A logical explanation is that, in the north area, only the low-volume Chicago ramp feeds traffic to the freeway, whereas in the south portion are the very-high-volume West Grand Boulevard entrance ramp and the above-average-volume Seward entrance ramp. The results show significant changes in lane changing under the influence of ramp control. Interpretation of results will require more work in the future due to two factors which apparently work against each other.

Ramp control produced more fluidity of traffic movement in the freeway permitting more lane changes than would be possible under conditions of congestion. Contrary to this is the reduction of entering traffic in certain areas due to ramp control. This should reduce the number of lane changes that might otherwise occur. The phenomena of lane changing will require much more detailed study in the future because findings in this study show definite merit to such investigation.

TABLE 6

DIRECTION OF LANE CHANGING COMPARISON
(Northbound Direction Only, PM)

Camera Fields	Lane Changes and Left Movements																	
	Seward Closed, Wed. (4:00-5:35)			W. Grand Blvd. Closed, Thurs. (4:00-5:35)			W. Grand Blvd. Closed, Fri. (3:30-6:00)			No Ramps Closed								
										Tues. (4:15-4:30)			Wed. (3:45-4:00)			Thurs. (3:10-3:45)		
	Total	Left	%	Total	Left	%	Total	Left	%	Total	Left	%	Total	Left	%	Total	Left	%
8	42	28	67	70	63	90	187	160	86	—	—	—	7	6	85	30	22	73
9	133	84	63	164	140	85	573	531	93	55	46	84	22	18	82	20	13	65
10	116	81	70	67	35	52	215	109	51	8	5	63	9	4	44	16	14	88
11	320	218	68	155	52	34	159	70	44	56	38	68	70	52	74	90	63	70

Table 6 gives a comparison of the direction of lane changing, showing how lane changes to the left were affected by the various ramp closures. The results show significant variance from one period of study to another; again, proper analysis is difficult because of the influence of variables beyond the control of the experiment. The number of lane changes in the camera fields studied was generally higher with ramp closure. The percentage of lane changes to the left, as compared to the total number of changes, showed wide variations but no definite association with ramp closure. A future study should include the volumes of both entrance and exit ramps, along with freeway volumes in each study area, to obtain lane change rates. These would have to be compared to traffic stream velocities during the period of study.

TABLE 7

CONGESTION PERIODS COMPARISONS[a]

Period	Congestion Periods				
	Tuesday	Wednesday	Thursday	Friday	Avg. Ending Times
(a) Southbound—AM Peak					
Previous 3-week period	7:12-8:27	7:28-8:58	7:16-8:53	7:15-8:54	8:46
	7:23-8:51	7:28-8:49	7:12-8:51	—	
	—	7:09-8:45	7:08-8:45	7:25-8:23	
Study week	7:14-8:36	7:08-8:19	7:15-8:46	7:14-8:22	8:30
(b) Northbound—PM Peak					
Previous 3-week period	3:42-5:45	3:39-6:05	3:39-5:57	3:47-6:06	5:57
	3:47-5:41	3:38-5:57	3:28-5:45	—	
	3:47-5:50	3:44-6:29	4:15-6:03	3:49-5:49	
Study week	3:41-5:00	3:49-5:33	3:42-5:00	3:36-5:51	5:20

[a] Periods with stoppages.

In Table 7 a comparison of congestion periods, with and without ramp control, was made. These periods were declared to be ended when stoppages, due to traffic congestion, were no longer present in the traffic stream. Under these circumstances, the freeway had sufficient capacity to handle the traffic demand. In the northbound direction, congestion periods with ramp control averaged 37 min less time; in the southbound direction the average was 16 min less. This was logical inasmuch as none of the southbound entrance ramps have the amount of traffic using the northbound Seward and West Grand Boulevard ramps. Also, because the distance to be traveled south by the freeway to the central business area is so short, drivers have already learned to avoid congestion by use of alternate routes.

Shortening of the congestion periods by ramp control appears to be consistent with facts because it is reasonable to assume that a fairly constant number of trips are generated from an area in a fixed period of time. If this traffic cannot be handled in accordance with demand, movement over the surface street and freeway system is retarded. This slow-speed storage of vehicles results in congestion that endures until the capacity of the streets again catches up with the demand. The experiment indicates that freeway capacity can be increased, thus handling more quickly a fixed supply of vehicles either entering or leaving the central business area of Detroit.

Table 8 shows the effectiveness of ramp signals at the various entrance ramps after a 2-mo period in which no enforcement was applied. The results appear to confirm previous studies in this area and conclusively point out the need for either continuous enforcement or a positive barrier.

TABLE 8

RAMP SIGN VIOLATIONS
(Two Months After Study Week)

Southbound Ramp Closures	Violation % of Normal Volume	Northbound Ramp Closures	Violation % of Normal Volume
Glendale	42	Chicago	28
Glendale	36	Chicago	44
Glendale	23	W. Grand Blvd.	24
Webb	37	W. Grand Blvd.	32
Chicago	30	Seward	54
Clairmount	11	Chicago	21
Clairmount	23		

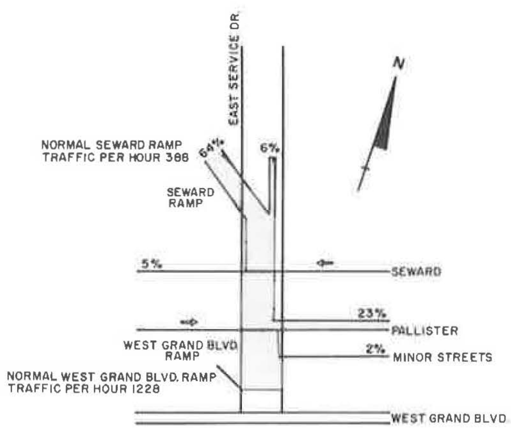

Figure 3. Alternate routes used when northbound West Grand Blvd. ramp is closed.

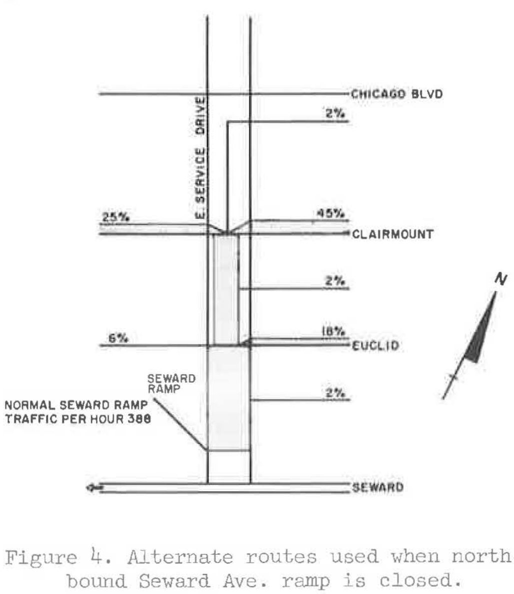

Figure 4. Alternate routes used when northbound Seward Ave. ramp is closed.

SURFACE STREET ANALYSIS

Figures 3 to 8 show the normal traffic volumes at various entrance ramps and their distribution to other ramps or surface streets as determined by machine counts at the various locations.

Figure 3 shows that 64 percent of diverted northbound West Grand Boulevard ramp traffic enters at Seward. The other distributions indicate that nearly 30 percent of the rest of the volume proceeded eastward to 2nd Avenue, or east on Davison. In any case, this group apparently travels no further on the John C. Lodge Freeway than to the Davison ramps. Seward's dispersion pattern indicates a rather high percent of short-trip travelers, inasmuch as all diverted ramp traffic was dispersed before it reached Chicago Boulevard. Conversely, traffic diverted from the Chicago ramp appears to be freeway oriented, inasmuch as 67 percent entered the freeway at the Webb ramp (Fig. 4).

In the southbound direction with Glendale closed, Figure 5 shows 37 percent of the normal Glendale ramp traffic enters at the Webb ramp, with most of the balance using surface streets. Sixty-nine percent of the Webb Avenue ramp traffic proceeds to Chicago ramp and most of the balance enters at Clairmount as shown in Figure 6. Figure 7 shows the combined distributions of Glendale and Webb ramp closures, most of which proceeds to Chicago and Clairmount entrances.

Some of the short-period ramp-closure volumes could not be traced reliably, as at Milwaukee and Clairmount; hence, they are not included in the sketches.

The approximate percentages of diverted entrance ramp traffic of the volumes moving on the freeway ranged from 9 to 17 percent for the 3-hr outbound periods. This traffic merely had to proceed to the next available entrance ramp to continue in the desired direction or proceed to a surface street if they were short-trip drivers. In the morning 2-hr period, the various closures diverted from 5 to 8.8 percent of the inbound volume on the freeway. By shifting the entrance point for relatively few vehicles, it was possible to improve the movement throughout the whole area, and these diversions, for the most part, did not stagnate surface street movements. Usually the surface street volumes would move steadily through the area between signal changes.

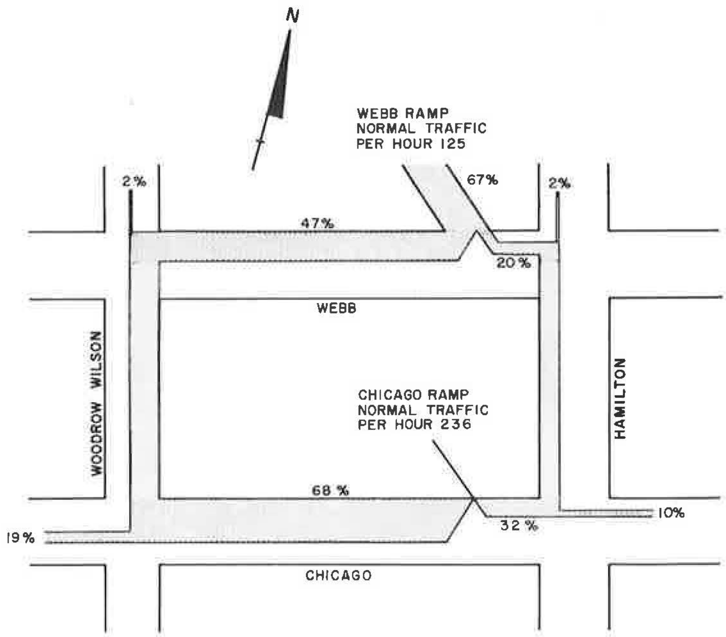

Figure 5. Alternate route used when Chicago Blvd. northbound ramp is closed.

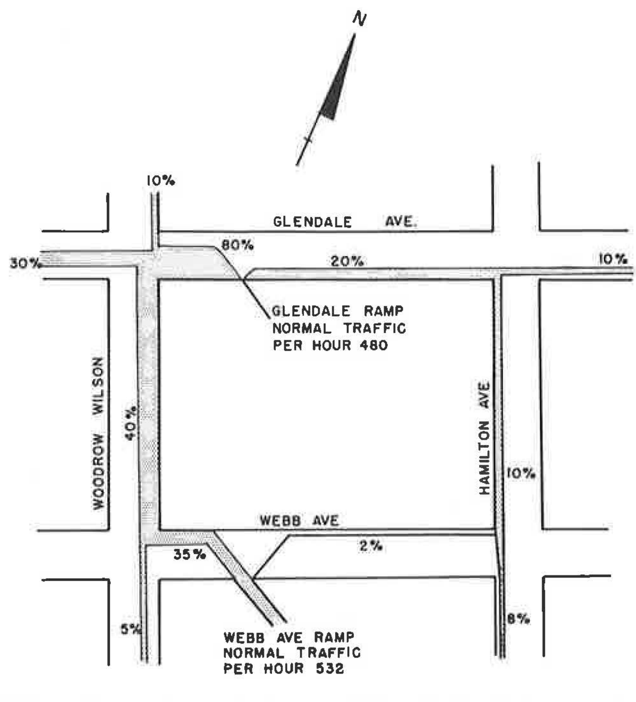

Figure 6. Alternate route used when southbound Glendale ramp is closed.

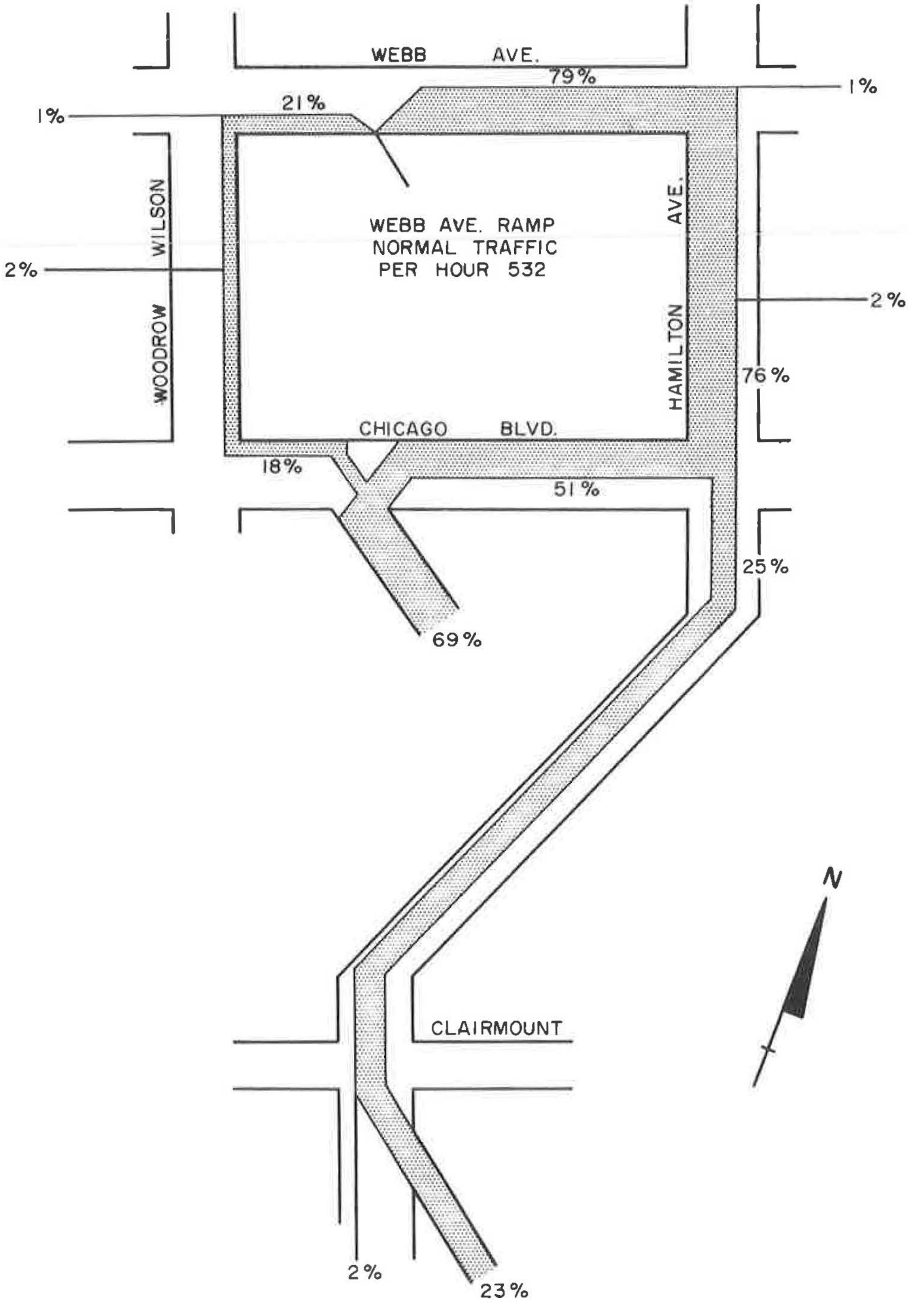

Figure 7. Alternate routes used when southbound Webb Ave. ramp is closed.

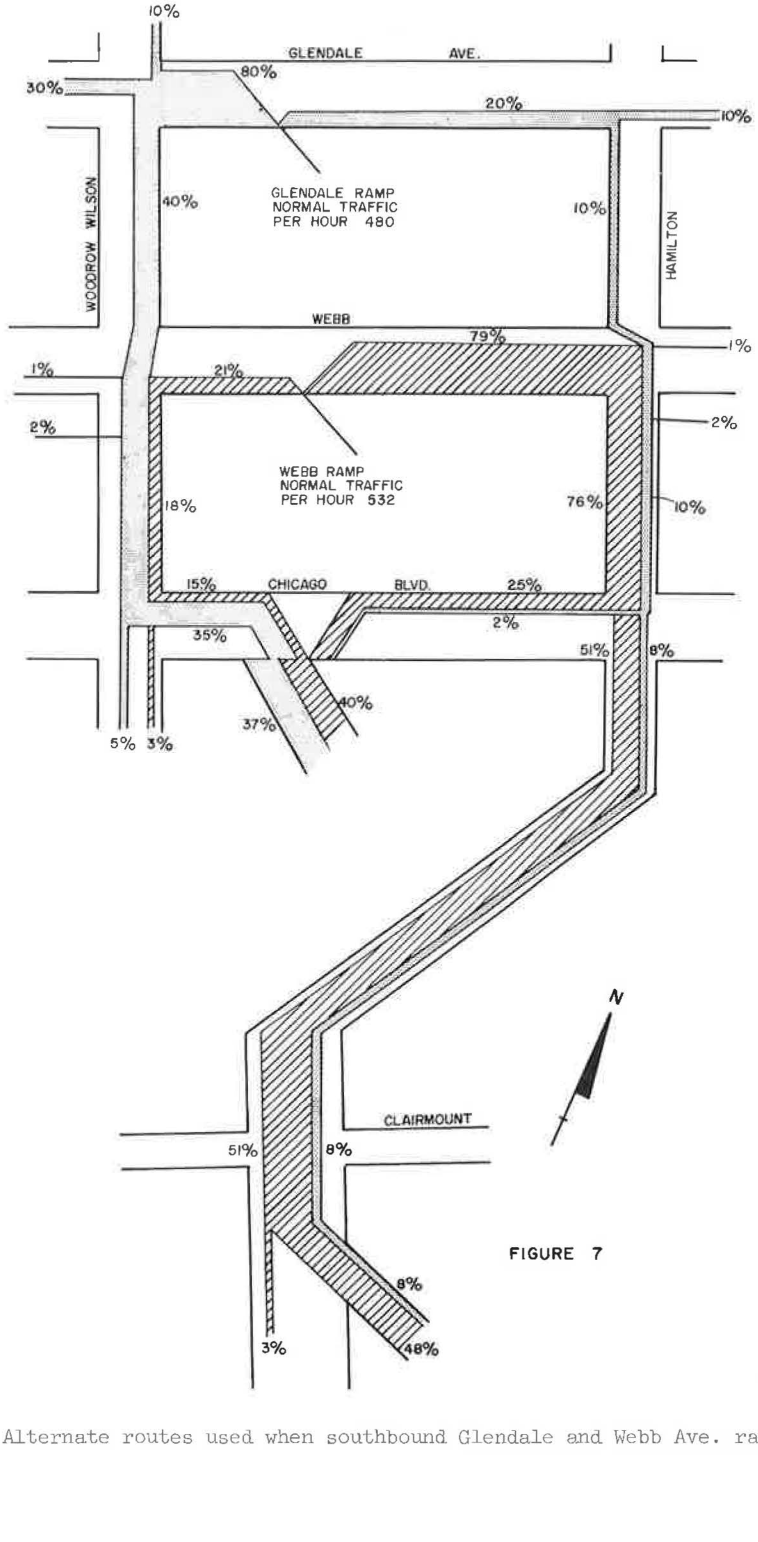

Figure 8. Alternate routes used when southbound Glendale and Webb Ave. ramp is closed.

SUMMARY

This review of the week's study definitely shows that a great gain in freedom of traffic flow is possible by the use of the ramp closure system, provided it is used wisely and prudently as conditions warrant and driver adherence to the sign indications remains at a high level. The ramp-closing experiment indicates that freeway traffic volumes are increased during the peak periods from 7:00 to 9:00 AM and 3:00 to 6:00 PM. This increase is accomplished even though some of the diverted ramp traffic is bypassing the freeway traffic sensors during some of the ramp-closing incidents. The passage of greater traffic volumes is reflected by a definite shortening of the periods when congestion occurs. This tends to prove the theory that much of the congestion on freeways and surface streets can be minimized by a traffic control system designed to keep traffic moving at greater speed through or around areas of capacity "bottlenecks."

Increases in peak-period traffic volumes are a result of producing higher average speeds on the freeway during ramp closing, along with a marked reduction in the number and extent of lane stoppages. A close examination of several ramp-closing incidents proves that, even after the freeway reaches a condition of very low speeds accompanied by stoppages, traffic capacity can be regained by closing ramps. Under normal conditions, without ramp control, low capacity caused by low vehicle speeds will persist until there is a definite lessening of peak-period traffic. The freeway is normally subjected to a series of shock waves traveling long distances upstream. With ramp control, the number of shock waves is greatly curtailed and could be dissolved without affecting large areas of the freeway.

Observations of traffic on the surface street system surrounding the freeway reveal many instances where improvements in traffic control would increase capacity. These changes could accommodate bypass traffic from the freeway in a reasonable manner. Although this statement must be confined to the situation found in Detroit, it would probably be true in many other areas. With greater experience in ramp control, if bypass capacity was found needed but unobtainable in a certain area, more capacity could be obtained by construction which, in turn, is implemented by a control system. This would be far more efficient than present methods in which surface street control is not integrated with freeway traffic.

To acquire more information for use in future studies on the utilization of the John C. Lodge Freeway by traffic entering at the West Grand Boulevard on-ramp, a license plate study was conducted. A record was kept of the departures of this traffic from

Figure 9. Service drive.

the off-ramps lying beyond the West Grand Boulevard entrance ramp and as far as the interchange with the Davison Freeway. This represents a distance of approximately $2\frac{1}{2}$ mi. From a total of 3,043 vehicles recorded entering at West Grand Boulevard, 54 left at Clairmount, 198 at Hamilton, 51 at Glendale, and 282 at the Davison Interchange. From these figures, it can be seen that 21 percent of the traffic entering at West Grand Boulevard left the freeway within a distance of $2\frac{1}{2}$ mi.

Figure 9 shows service drive being utilized by traffic prevented from entering the freeway by a closed entrance ramp. The statistical evidence showing the high percentage of short-trip lengths utilizing the freeway seems to account for another interesting observation. During "after" periods, without ramp control, large numbers of vehicles were observed using the service drives to travel along the freeway. Apparently the experience they obtained during ramp closures showed the value of using surface routes during rush hours.

More specific information gathered during the ramp-closing study shows that, because the major traffic restrictions for southbound traffic in the study area occurred in the first three camera fields with the major restriction located at the Davison entrance ramp, ramp closures of the Davison, Glendale, and Webb ramps provides the greatest betterment to freeway traffic operation. Traffic, diverted from the freeway by the closure of these ramps, had little difficulty entering the freeway further south at the Chicago and Clairmount ramps, due to the addition of another lane to the John C. Lodge in this area beginning at Chicago Boulevard. Closing of the Clairmount and Milwaukee entrance ramps showed little or no improvement in freeway traffic behavior. Whereas this might seem inconsequential to the experiment, actually it is quite important. It gives evidence to support the view that all ramps do not produce equal disruptions to freeway traffic; therefore, by a careful selection of ramps to remain open, widespread areas of a city need not be deprived of an entrance to the freeway. Sooner or later, traffic leaving a freeway permits the addition of more traffic downstream to the exit point.

Northbound or outbound traffic from the central business district regularly has stoppages in the area between the Edsel Ford Freeway (Camera 14) and Calvert (Camera 4). One hundred or more stoppages due to congestion normally occur daily between 3:30 and 6:00 PM. This condition is caused by a left- and right-hand entrance ramp from the Edsel Ford Freeway joining northbound John C. Lodge Freeway traffic. North of the Hamilton exit ramp, the John C. Lodge Freeway is reduced from four to three lanes without enough departing traffic to justify the lane reduction. Even with these obvious deficiencies in freeway capacity, ramp control was able to show definite improvements in freeway operation without any great sacrifice on the part of the motorists affected by ramp control.

A series of trace recordings, made by an X-Y pen recorder, show many interesting relationships between speed and volume, with and without ramp control. Although this method of recording does not lend itself to the acquisition of composite data, it shows a predominant number of individual examples giving evidence to support the fact that ramp control does improve speed and capacity of the freeway. Whereas the scope of this study and the limitation of present instrumentation do not permit conclusive evidence on the effects of ramp control, the results are most encouraging and show definite benefits to be derived from future studies. Experiences gained from the initial study will prove very beneficial in planning future work.

CONCLUSION

The results of the 1-wk experiment are being documented and publicity will be given to the Detroit public for the purpose of acquainting them with the advantages created by ramp closure. This will offset the complaints from some neighborhood groups who were of the opinion that ramps were being closed for the purpose of taking care of people living in the suburbs to the detriment of the people living in the city. In one respect, this may be true since long-trip lengths are given preference. The public must be informed, in an intelligent manner, why ramp control is needed to preserve good freeway operation. This will be a benefit to the majority of motorists by a small sacrifice on the part of a few. They must appreciate that the bypass routes offer a reasonable substitute to an otherwise congested freeway without benefit of ramp control. This situation is no different from the removal of left turns from a street during rush hours to improve the capacity of through movements. If the people of the city, who make the short-trip lengths on the freeway, are allowed to gain entrance and add their traffic volumes to the saturated freeway traffic, the total travel time may actually be longer for them as well as the people on the freeway. Once this point can be conveyed to the public, continuation of ramp closure experiments can probably be performed without difficulty.

On the whole, the public acceptance of ramp closure has been good. The majority of people like the results. The people driving the freeway during the week of the experiment were quite lavish in their praise. However, the minority who were closed off of the ramp which led from their local area did voice objections; and these are the people who must be convinced that the path, which is provided as an alternate, would be the equal to the one which they formerly traveled—if not even a little better.

To improve this travel path, alternate routes will be established in areas further from the freeway so areas of maximum congestion may be avoided. Animated signs may be used to give the driver information on the freeway conditions well in advance of the freeway area so he may choose an alternate route where more choices of routes are available. On the marked or signed alternate routes, changing arrow signals may be provided underneath the trail-blazer signs leading to ramps so the motorist will not be led to a ramp if it is closed. The arrow will always point the motorist along the alternate route to the ramp open to freeway traffic. This could be done quite simply with the present control system.

In future experiments on ramp closure, it is hoped that the surveillance system will be improved to better document actual traffic conditions. The problem of getting more positive obedience to the ramp signals can be alleviated by the provision of barrier gates along with the signals. This would free the police officers for duty elsewhere. Although it may not be possible to provide a gate at all entrance ramps, due to the limitation of present Project funds, certain entrance ramps may be selected in which the greatest benefits are obtained by ramp closure and gates may be provided. By closing these ramps in future experiments, more conclusive data may be obtained which will prove or disprove the benefits of ramp control.

REFERENCE

1. Auer, J. H., Jr., "A System for the Collection and Processing of Traffic Flow Data by Machine Methods." HRB Bull. 324, pp. 85-95 (1962).