324

HIGHWAY RESEARCH BOARD

Bulletin 324

Freeway Operations

National Academy of Sciences—
National Research Council

publication 1003

HIGHWAY RESEARCH BOARD

Bulletin 324

Freeway Operations

Presented at the
41st ANNUAL MEETING
January 8-12, 1962

National Academy of Sciences—
National Research Council
Washington, D. C.
1962

$2.00

Department of Traffic and Operations

Fred W. Hurd, Chairman
Director, Bureau of Highway Traffic, Yale University
New Haven, Connecticut

COMMITTEE ON FREEWAY OPERATIONS

Alger F. Malo, Chairman
Director, Department of Streets and Traffic
Detroit, Michigan

Adolf D. May, Jr., Secretary,
Thompson Ramo Wooldridge, Inc.
Canoga Park, California

Louis E. Bender, Chief, Traffic Engineering Division, The Port of New York Authority, New York, N. Y.
Donald S. Berry, Professor of Civil Engineering, Civil Engineering Department, Northwestern University, Evanston, Illinois
Harold Cooper, Director, Traffic Division, Michigan State Highway Department, Lansing
Donald O. Covault, Associate Professor, Department of Civil Engineering, Georgia Institute of Technology, Atlanta
Charles J. Keese, Civil Engineering Department, Texas Transportation Institute, Texas A & M College, College Station
E. E. Lewarch, Traffic Engineer, Seattle, Washington
Eugene Maier, Director of Public Works, Houston, Texas
Burton W. Marsh, Director, Traffic Engineering and Safety Department, American Automobile Association, Washington, D. C.
William J. Miller, Jr., Director of Operations, Delaware State Highway Department, Dover
Theodore F. Morf, Engineer of Research and Planning, Illinois Division of Highways, Springfield
O. K. Normann, Deputy Assistant Commissioner for Research, Bureau of Public Roads, U.S. Department of Commerce, Washington, D. C.
Edmund R. Ricker, Director, Traffic Engineering Bureau, Pennsylvania Department of Highways, Harrisburg
Carlton C. Robinson, Director, Traffic Engineering Division, Automotive Safety Foundation, Washington, D. C.
Harold W. Sullivan, City of Los Angeles Police Department, Los Angeles, California
John L. Vardon, Route Analysis Engineer, Traffic Section, Ontario Department of Highways, Toronto
George M. Webb, Traffic Engineer, California Division of Highways, Sacramento
Edward G. Wetzel, Special Assistant to the Director, Port Development Department, The Port of New York Authority, New York, N. Y.

Highway Research Correlation Service

HIGHWAY RESEARCH BOARD of the

National Academy of Sciences—National Research Council

2101 Constitution, Washington, D.C.

BULLETIN 324

FREEWAY OPERATIONS

Included in this 95-page bulletin are the following papers presented at the 41st Annual Meeting of the Highway Research Board:

"Predicting the Effectiveness of Highway Signs," by Albert Berg, and Slade F. Hulbert.

"Squirrel Hill Tunnel Operations Study," by Adolf D. May, Jr., and David G. Fielder.

"Operational Study of Signalized Diamond Interchanges," by Charles Pinnell and Donald G. Capelle.

"Some Fundamental Relationships of Traffic Flow on a Freeway," by Donald P. Ryan and S. M. Breuning.

"A System for the Collection and Processing of Traffic Flow Data by Machine Methods," by J. H. Auer, Jr.

Price: $2.00 July 1962

Contents

Predicting the Effectiveness of Highway Signs

ALBERT BURG and SLADE F. HULBERT, Institute of Transportation and Traffic Engineering, University of California, Los Angeles

A new highway sign may be in place for several years before a sufficient feedback from motorists has accumulated to permit adequate evaluation of its effectiveness. A study is described in which a film technique was utilized to predict the comparative effectiveness of several signs proposed for use in locations where a lane is to be dropped. Each sign was properly installed on the highway and filmed from a moving vehicle. The filmed drives thus obtained were presented (in random order) to groups of drivers who responded, after each, to questions designed to determine how accurately the intended sign message was perceived. Analysis of these responses permitted the signs to be ranked on several dimensions, and conclusions were then drawn as to the relative effectiveness of the signs.

• ONE MAJOR DIFFICULTY in evaluating the effectiveness of highway signs is that several months or years may be required to determine (through accident experience and/or complaints by motorists) whether the signing is adequate. Further, over a period of years changes are likely to occur in the signing needs of a given location. For example, realignment of the highway or the erection of billboards may affect the prominence of the signs,* which will necessitate reevaluation and possibly resigning.

Because of these and other reasons, it would be of considerable practical advantage to be able to perform quickly an evaluation of the effectiveness of a sign. At the request of the California Division of Highways, the Institute undertook to devise a rapid technique for predicting the potential influence of a sign on motorist behavior. Specifically, the study concerned a comparative evaluation of several signs, each warning the driver that the multilane highway on which he is traveling is about to drop a lane. The four signing configurations studied were the following:

Sign A—Two 40- by 40-in. diamond-shaped signs mounted on posts 400 ft apart, the first sign reading "Pavement Narrows" and the second "Squeeze Left."

Sign B—Same as Sign A, except second sign reads "Merge Left."

Sign C—A single 40- by 40-in. diamond-shaped sign reading "3 Lanes" underneath which are four vertical arrows, with the one on the right having a slash drawn through it.

Sign D—A single rectangular sign, 4- by 8-ft reading "Lane Ends Merge Left."

---

*Prominence is one of the six basic principles of freeway signing set forth by Schoppert et al (1).

In addition to the four primary signing configurations listed above, a fifth sign, Sign E, was added for limited evaluation. This diamond-shaped sign, similar to the European standard, was entirely symbolic, and consisted of a representative line drawing of the impending lane-drop situation. Figure 1 shows all of the signing installations used in the study, all of which had black figures on a yellow background.

To obtain results based on a large sample and in a relatively short period of time, a film technique was devised by means of which groups of drivers could be exposed to the test signing as it actually would appear when installed on the highway. Experience with a similar filmed-ride technique in an earlier study of highway signs (2) had demonstrated its feasibility for studies where actual on-the-road experimentation would be too costly, time-consuming, and dangerous.

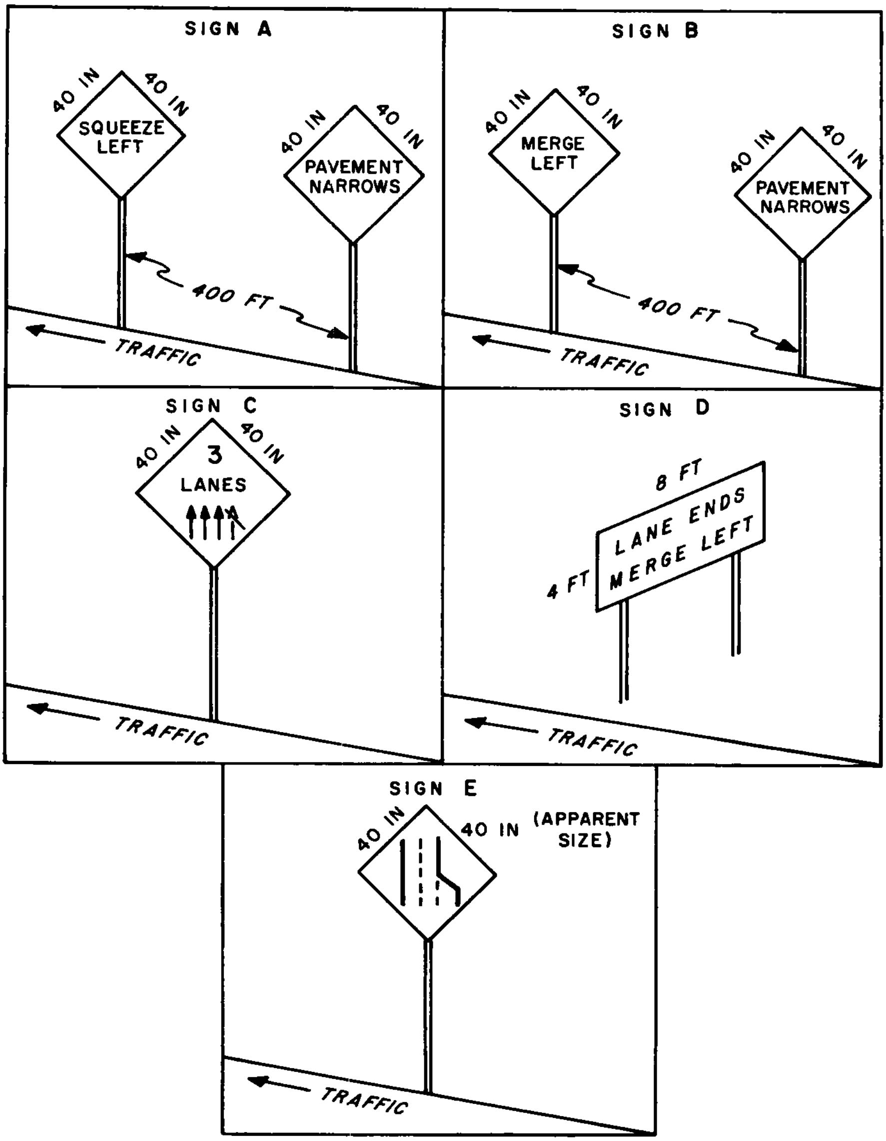


Figure 1. Signing installations.

METHOD

Signs were installed in a standard manner, motion pictures were taken from the driver's position in a moving vehicle, and these films were shown to the subjects.

A completed but unopened section of freeway in Los Angeles was chosen as the film site. The signs were placed just in advance of a curve, so that in viewing the film it would not be apparent to the subject that there was, in fact, no actual lane drop ahead.

Each of the four primary sign configurations, in turn, was installed on posts in a completely standard way. (The signs, as well as their installation (and removal), were provided by the California Division of Highways, District VII.) A 16-mm motion picture camera was mounted on a vehicle (from which the top and windshield had been removed) in such a way that it was in the position normally occupied by the driver's head. With the camera operating at a speed of 24 fps, the vehicle was then driven in the outside lane, at approximately 35 mph, for over 1 mi before approaching and passing the signing installation. This produced a filmed ride of approximately 2½-min duration (at 24 fps) for each sign, and all of the filmed rides thus started with a period of travel along an identical stretch of highway. Standardized conditions were maintained for the production of all four filmed rides, so that they differed only in the sign message. In addition, still photographs were taken of each sign installation, and lantern slides made for later use in the study.

Sign E was added to the study for evaluation after the above-mentioned filmed rides had been produced. Because the freeway section had been opened by this time, a drawing of the sign was photographed and a lantern slide made for viewing by a limited number of subjects. This slide was a composite, so that it matched the other slides as to background.

Subjects

In order to obtain a large sample of subjects in as short a time as possible, group testing was decided on. A total of 833 individuals was tested during a one-week period in the Los Angeles area, and 47 additional subjects were subsequently tested in Washington, D. C., as a check on regional differences in response to the signs.

In Los Angeles, the two major sources of subjects were (a) 560 students on the Campus of UCLA, and (b) 273 attendees at the Western Safety Congress, April 1960. The former are referred to as the "Student" group, and the latter as the "Safety" group. In many instances these two groups were analysed separately because of their markedly different compositions (see Table 1), in other analyses the two groups were combined. The "Washington" group is analysed separately throughout, for purposes of comparison.

The nine subgroups of the Student group ranged from 22 to 154 persons, while the four Safety subgroups ranged from 23 to 173. The two Washington groups were 20 employees of the American Automobile Association and 27 employees of the District of Columbia.

As is evident from Table 1, the subject population consisted almost entirely of drivers (of both sexes), with a considerable range of both age and driving experience. As was to be expected, most of the subjects were California residents.

Procedure

Each of the UCLA Student groups was tested in its own classroom, and each of the four Safety groups was tested in a large meeting room at the hotel at which their conference was being held. The two Washington groups were tested at their respective places of employment.

The experimenter started by stating to the group that they were about to see some short films of traffic signs, following each of which they would be asked to answer some questions. They were then told that detailed instructions would be found on the sheets to be handed to them. Each member of the group was then given an instruction sheet and two or four answer sheets. After the subjects had finished reading these, the instructor commented on them briefly, emphasizing the salient points and answering any questions. Appendix A contains samples of the instruction and answer sheets.

TABLE 1

DESCRIPTION OF SUBJECT POPULATIONS

Item	Student Group (N = 560)		Safety Group (N = 273)		Total Los Angeles Group (N = 833)		Washington, D C , Group (N = 47)	
	No.	%	No.	%	No	%	No.	%
Males	88	32.8	223	86.4	311	59.1	36	76.6
Females	180	67.2	35	13.6	215	40.9	11	23.4
Drivers	523	93.4	257	99.2	780	95.2	47	100.0
Nondrivers	37	6.6	2	0.8	39	4.8	0	0.0
Calif residents	273	96.8	-	-	273	96.8	0	0.0
Non-Calif residents	9	3.2	-	-	9	3.2	47	100.0
Driving experience (yr)								
Total								
None	37	7.4	2	0.7	39	5.1	0	0.0
Less than 1	11	2.2	0	0.0	11	1.4	1	2.1
1 to 3	92	18.5	2	0.7	94	12.2	2	4.3
3 to 6	216	43.5	2	0.7	218	28.3	6	12.8
6 to 11	88	17.7	19	7.0	107	13.9	3	6.4
11 or over	49	9.9	231	84.6	280	36.4	35	74.5
Not specified	4	0.8	17	6.2	21	2.7	0	0.0
Local								
None	25	10.0	0	0.0	25	4.8	1	2.1
Less than 1	13	5.2	1	0.4	14	2.7	2	4.3
1 to 3	80	32.0	8	2.9	88	16.8	3	6.4
3 to 6	93	37.2	17	6.2	110	21.0	6	12.8
6 to 11	7	2.8	39	14.3	46	8.8	9	19.1
11 or over	3	1.2	181	66.3	184	35.2	19	40.4
Not specified	29	11.6	27	9.9	56	10.7	7	14.9

TABLE 2

ANALYSIS OF FIRST IMPRESSIONS, LOS ANGELES SAMPLE

Apparent Message Interpretation or Comment[1]	Percent Making Comment[2]											
	Student Group (N = 560)				Safety Convention Group (N = 273)				Combined Groups (N = 833)			
	Sign A	Sign B	Sign C	Sign D	Sign A	Sign B	Sign C	Sign D	Sign A	Sign B	Sign C	Sign D
Seems to get message	63.7	70.2	72.4	79.1	45.3	63.0	66.0	60.0	57.7	69.1	70.4	77.0
Not sure whether subject got message	18.8	15.0	17.1	12.3	41.7	32.9	24.0	20.0	26.2	17.7	19.2	13.2
Seems not to have gotten message	17.5	14.8	10.6	8.6	13.0	4.1	10.0	20.0	16.0	13.2	10.4	9.9
"First (or only) sign confusing or dangerous"	16.6	16.5	30.5	7.1	14.8	8.2	14.5	4.0	16.0	15.2	25.6	6.8
"Second sign confusing or dangerous"	41.3	11.6	—	—	33.6	13.7	—	—	38.8	11.9	—	—
Driver's mistaken impression liable to cause trouble	14.7	12.6	8.4	8.4	9.4	1.4	8.0	12.0	13.0	10.9	8.3	8.8
Driver "would slow down" upon seeing sign	17.1	17.0	7.1	15.3	6.7	12.3	5.0	22.0	13.7	16.3	6.6	16.0
Driver's impression is that lane drop will occur very soon	4.3	1.5	4.8	8.4	0.4	—	1.0	—	3.1	1.3	3.6	7.5
Driver thinks "pavement" means shoulder	1.7	2.5	—	—	1.3	1.4	—	—	1.6	2.3	—	—
"Sign should be larger"	8.6	7.1	6.0	0.7	8.1	11.0	9.0	2.0	8.5	7.7	6.9	0.9
Number of people seeing each sign	463	406	463	406	223	73	200	50	686	479	663	456

[1] Comments in quotes.
[2] Based on the number of people seeing each sign.

Due to limitations in time for some of the groups, only two film strips were shown, while for the other groups, all four films were presented. For every group, however, the order of presentation of the signs (film strips) was randomly chosen in advance. Because group sizes varied, at the conclusion of the study each sign had not been seen by equal numbers of subjects; however, this fact was taken into account in the data analysis.

The experimenter then showed the first film, and immediately upon its conclusion turned the lights back on and requested the subjects to put down their first impressions as well as an indication of the clarity of these impressions. Once this was done, the procedure was repeated for the next film, and so on.

After the subjects had finished writing their responses to the last sign, they were asked to express their personal preference among the signs they had seen. For those groups who had seen all four signs, the signs were shown to them again by means of the previously mentioned slides, in the same order in which the film strips had been presented. When only two film strips had been shown to the group, no slides were shown. For one of these groups (Group DB, which had been shown Sign D followed by Sign B), the 23 subjects chose one or the other as their preference, and then were shown all five slides, in the order D, B, A, C, E, and then asked to choose the one they preferred. One of the Washington groups (N=27) was also shown the composite slide of Sign E and rated it in comparison with the other signs for both clarity of meaning and personal preference. Thus, a total of 50 subjects saw Sign E, and each of the other signs was viewed by at least 500 subjects.

RESULTS

Analysis of First Impressions

Table 2 gives the results of an analysis of the Los Angeles subjects' first impressions of the sign messages. (Due to time limitations, first impressions were not analysed for the Washington groups.) These first impressions were studied in an attempt to judge whether the driver was actually "getting" the message as it was intended. Other response categories were established to study these subjects' behavior as it might have occurred after seeing such signing situations in real life. This potential behavior was either explicit in the subject's comments or estimated by the rater from the implications of these comments. The left-hand column of the table gives the ten classifications used. Explicit comments by the subjects are given in quotation marks, and the remaining categories represent rater judgments. With the exception of the first three, the categories are not mutually exclusive.

In addition, a number of subjects made suggestions for improving the signing for a lane drop situation. These suggestions were tallied and are listed in Appendix B.

Analysis of Clarity Ratings

Immediately after giving his first impressions of a sign, each subject was asked to judge whether these impressions were Clear and Immediate, Confused, Obvious, Complex, and/or Slow in Forming. Table 3 summarizes these clarity ratings, distinguishing between the Los Angeles and Washington, D.C., populations. Table 4 combines "favorable" and "unfavorable" ratings to provide a more meaningful comparison among the signs.

Analysis of Preferences

1. Of the 307 Los Angeles subjects (all students) who saw all four signs and who expressed a preference,

19 (6.2 percent) preferred Sign A,
54 (17.6 percent) preferred Sign B,
57 (18.6 percent) preferred Sign C,
177 (57.7 percent) preferred Sign D.

TABLE 3

SUMMARY OF CLARITY OF IMPRESSIONS[a]

Sign	Number of Subjects Rating Impressions as				
	Clear and Immediate	Confused	Obvious	Complex	Slow in Forming
A	241(26)	196(24)	167(14)	76(4)	135(12)
B	263(29)	108(14)	177(14)	40(4)	95(14)
C	210(11)	168(27)	171(6)	95(16)	150(14)
D	269(32)	67(9)	180(13)	28(3)	90(8)
E	---(3)	---(14)	---(1)	--(9)	---(12)

[a]Washington sample data in parentheses.

TABLE 4

SUMMARY OF CLARITY OF IMPRESSIONS COMBINING "FAVORABLE" AND "UNFAVORABLE" RATINGS[a]

Sign	Clear and Immediate and/or Obvious (no.)	Confused and/or Complex and/or Slow in Forming (no.)
A	408 (40)	407 (40)
B	440 (43)	243 (32)
C	381 (17)	413 (57)
D	449 (45)	185 (20)
E	--- (4)	--- (35)

[a]Washington sample data in parentheses.

Seven groups composed this total of 307. Each group saw all four signs but in a different order, so as to avoid the possibility that the subjects' responses would be affected by the order in which they saw the signs. In every group Sign D was preferred over the other three signs by a wide margin, Sign A was always last choice, and Signs B and C fell somewhere in-between.

Comparable percentages are not available for the Washington subjects, but the rank order of their preference was (in decreasing order) D, B, A, C. For the 27 Washington subjects who also saw sign E, it was ranked last. (No Washington subject actually preferred Sign E over any of the other four, but one individual stated that it would be his preference if it could be used widely and motorists were educated to its meaning.)

2. Of the 833 Los Angeles subjects, 683 expressed a preference. Taking into consideration the various orders of presentation and the different number of subjects seeing each sign, it is possible to equate the signs for exposure. Table 5 shows comparisons between the number of subjects expected to prefer a given sign by the laws of chance alone and the number actually preferring each sign. Utilizing the χ^2 test of statistical significance, it is found that these differences between the expected and observed numbers would be expected to occur by chance fewer than one time in a thousand.

3. For Safety subgroup DB (N=23), which was the only Los Angeles group to be exposed to Sign E, the results were as follows:

a. In the first preference asked for (that between D and B), 14 preferred D, 6 preferred B, and 3 did not specify.
b. In the second preference asked for (between all five signs, as shown to the group on slides), 2 preferred A, 1 preferred B, 4 preferred C, 6 preferred D, 7 preferred E, and 3 did not specify.

TABLE 5

EXPECTED VS ACTUAL SIGN PREFERENCES, LOS ANGELES SAMPLE

Sign	Expected Preference (no.)	Actual Preference (no.)
A	194.25	110
B	157.25	149
C	184.25	169
D	147.25	255
Total	683.00	683

DISCUSSION

First Impressions

The first impression a driver receives from a traffic sign is important because the sign message may call for immediate action, or, at least, immediate planning for action. Equally important to the driver who is planning action is an awareness of just where this action is to take place. Making a called-for maneuver too soon or too late may result in a situation as dangerous as though no action had been taken.

It is for this reason that analysis of the subjects' first impressions took into account implicit or explicit indications of improper timing of the called-for action; i.e., preparing to move to the next left lane. Thus, several categories of response were derived which can be labelled as "potentially dangerous lane-merging actions," such as "immediately slowing down" or "immediately moving into the next left lane" on seeing the sign.

From Table 2, several conclusions seem apparent:

1. For the combined group, Sign D is most successful in conveying the intended message, is considered least confusing, is of adequate size, and is about average in leading to potentially dangerous behavior.

2. For the combined group, Sign A is least effective in conveying its message and is about average in leading to potentially dangerous behavior.

3. For the combined group, Signs B and C fall close together, somewhere between Signs A and D in effectiveness.

4. Marked differences exist between the Student group and the Safety group responses. This is not surprising, considering the disparity in age ranges, total driving experience, and California driving experience; however, due to the relatively small number of the Safety group seeing Signs B and D, as opposed to the larger (and relatively equal) numbers of the Student group seeing all four signs, firm conclusions about the differences between the two groups cannot be drawn. In general, however, the Safety group seemed "happier," or more at home, with the signs more commonly used in California up to that time (Signs B and C) than with the relative newcomer (Sign D) or the complete stranger (Sign A).

Clarity

To establish a comparison of the clarity of the impressions given by the different signs, favorable/unfavorable ratio for each sign can be derived from the data given in Table 4. Thus, Sign D ranks highest in this respect, followed in order by Signs B, A, and C. This rank order of clarity obtains for both the Los Angeles and Washington samples. Sign E was ranked last in clarity by the Washington group that evaluated it on this dimension.

Preference

The results of the preference analysis clearly indicate the following:

1. For both the Los Angeles and Washington samples, Sign D is first choice. This was true most strongly for the Student group and somewhat less emphatically for both the Safety group and the Washington sample.

2. Sign A ranks last for the Los Angeles sample (which is not exposed to this sign on California highways), while it ranks third for the Washington sample, which may have had some exposure to it in the East. Sign E ranks for the last-named group.

An explanation for such rankings may be found in the comments prevalent in the groups. Throughout there was an indicated acceptance of known messages and a rejection of new or different ones. This is perhaps an obvious observation, but nevertheless one of considerable importance to highway engineers who should take this resistance into account when planning to adopt new or drastically different signs.

SUMMARY

A laboratory study was conducted comparing the effectiveness of several types of signs, all attempting to convey the fact that a lane was to be dropped from the road ahead. A total of 880 persons saw motion pictures and slides of the signs. The written responses of these subjects revealed a consistent and statistically significant superiority for a sign reading "Lane Ends
Merge Left."

ACKNOWLEDGMENT

The authors gratefully wish to acknowledge the efforts of David W. Schoppert, Highway Transport Engineer, Automotive Safety Foundation, Washington, D.C., who was responsible for testing these subjects, analysing the responses, and forwarding the compiled results to the authors.

REFERENCES

1. Schoppert, D.W., Moskowitz, K., Hulbert, S.F., and Burg, A., "Some Principles of Freeway Directional Signing Based on Motorists' Experiences." HRB Bull. 244, 30-87 (1960).
2. Hulbert, S.F., and Burg, A., "The Effects of Underlining on the Readability of Highway Destination Signs." HRB Proc., 36: 561-574 (1957).

Appendix A

INFORMATION SIGN INSTRUCTION SHEET

_____ Total yrs. driving experience
_____ Years driving in Calif.

☐ Male
☐ Female
☐ Driver
☐ Non-driver

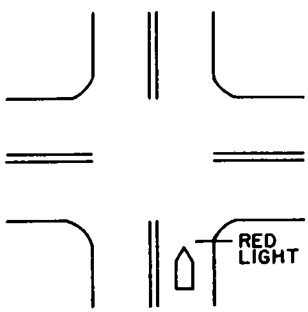
You are about to see a brief motion picture taken from a moving car. As you view this film, we ask you to assume that you are driving down a newly-opened, multi-lane highway, and you are paying the normal amount of attention to the various signs which are posted along the edge of the highway.

In the film, several such roadside highway signs, or combinations of signs will be shown, and we are interested in determining what your interpretations of these signs are, in terms of the decisions and actions, if any, you feel you are called upon to make as a result of the message given on the signs.

The film will be stopped after each signing situation, at which time please make a note in the space provided on the attached sheet of your FIRST impressions of the meaning that the sign message conveyed to you as a driver. Use one sheet for each signing situation.

Draw a sketch of the highway situation if it will aid you in adequately describing these first impressions. Try to describe in several ways what the traffic situation was like. Finally, give your opinion of the clarity of these impressions by checking one or more of the alternatives given below. As an example, below is a possible response to a traffic situation involving an approach to an intersection with the traffic signal turning red:

DRAWING OF TRAFFIC SITUATION:



DESCRIPTION OF FIRST IMPRESSIONS:

"I will be delayed for a moment; if I try and run the light I may have an accident, or get a ticket. I, and the drivers behind me, must come to a stop at the intersection; cars on the cross street get the green light and can proceed across the intersection. Also, pedestrians may cross in front of me. When the light turns green I can proceed again".

WERE THESE IMPRESSIONS AND MEANINGS:
(please check appropriate descriptions)

☒ a. CLEAR AND IMMEDIATE?

☐ b. CONFUSED?

☒ c. OBVIOUS?

☐ d. COMPLEX?

☐ e. SLOW IN FORMING?

NOW PLEASE TURN THE PAGE AND BE PREPARED TO GIVE YOUR IMPRESSIONS OF THE FIRST SIGNING SITUATION. USE ONE SHEET FOR EACH SITUATION. YOU WILL BE GIVEN ENOUGH TIME TO WRITE YOUR RESPONSE IN EACH CASE.

INFORMATION SIGN ANSWER SHEET

Signing Situation# ________

DESCRIPTION OF FIRST IMPRESSIONS:

____

____

____

____

____

____

____

DRAWING OF TRAFFIC SITUATION:

WERE THESE IMPRESSIONS AND MEANINGS:
(please check appropriate descriptions)

- ☐ a. CLEAR AND IMMEDIATE?
- ☐ b. CONFUSED?
- ☐ c. OBVIOUS?
- ☐ d. COMPLEX?
- ☐ e. SLOW IN FORMING?

Appendix B

Miscellaneous Suggestions for Signing Improvement

SUGGESTION	NUMBER OF PEOPLE MENTIONING
1. The signing situation should indicate how far ahead the lane drop occurs.	34
2. For signing situations A and B, the distance between the two signs should be greater.	10
3. For signing situations A, B and D, there should be an indication of how many lanes will remain after the lane drop occurs.	8
4. Arrows should be painted on the pavement showing the direction in which the driver should merge.	8
5. Sign D should read "Right Lane Ends, Merge Left".	6
6. For signing situations A and B, the first sign should read "Lane Narrows".	2
7. For signing situation B, the second sign should read "Merge to Left".	2
8. Suggested combination of "Pavement Narrows" sign followed by sign C.	2
9. Sign D should read "Lane Ends, Squeeze Left".	1
10. One-line, rather than two-line signs should be used.	1
11. Suggests a single sign saying "Lane Conversion".	1

Squirrel Hill Tunnel Operations Study

ADOLF D. MAY, Jr., Thompson Ramo Wooldridge and DAVID G. FIELDER,
Pennsylvania Department of Highways

• THIS PAPER contains the results of a traffic operations study of the westbound Penn-Lincoln Parkway in the vicinity of the Squirrel Hill Tunnel in Pittsburgh. The primary purpose of this study was to determine means of improving and maintaining optimum traffic operations during the presently congested morning peak period. It was also the purpose of this study to formulate freeway study techniques and to develop a better understanding of freeway characteristics to aid in improving traffic operations on other congested controlled access facilities.

The study employed photographic techniques which permitted the simultaneous observation and measurement of traffic characteristics at locations upstream, downstream, and at critical controlling points for intervals of time before, during, and after congestion. The use of the photographic technique has also resulted in a permanent record of traffic operations that can be reviewed qualitatively and quantitatively by technical personnel and that can be shown to civic groups as public information.

The study results include time and distance profiles of speed, volume, and density characteristics and the interrelationships among these characteristics. The analysis permitted the comparison of traffic characteristics between lanes and the estimation of traffic demand. The results of an investigation to determine the effect of trucks on time headways are included, and travel time studies were conducted.

The interpretation of the results of the study included locating critical points, determining causes of congestion and reduction in capacity, investigating means for improvement, and estimating benefits of such improvements. Investigation of possible means for improvement included consideration for ramp redesign, reversible lane operations, truck restrictions, ramp closures, metering ramp and/or freeway flow, and permitting lane changing in the tunnel. A general description of a traffic control system for maintaining optimum conditions during congested periods is included.

The final part of the paper describes the action taken by the Pennsylvania Department of Highways as a result of the operations study. This included the use of reversible lane operation, ramp closure and further investigation of a traffic metering system and road construction.

STUDY OBJECTIVES

The immediate objectives of this study were (a) to locate critical point or points controlling traffic operations, (b) to ascertain the cause or causes of congestion and reduction in capacity, (c) to determine possible means of improving parkway operations, and (d) to predict the effect of improvement on parkway operations.

The longer-range objectives of the proposed study were (a) to develop freeway study techniques, (b) to provide information for evaluating the effect of certain design features on freeway operations, and (c) to contribute to a better understanding of traffic behavior on freeways for the improvement of operation on this and other freeways.

BACKGROUND

The Penn-Lincoln Parkway is an east-west controlled-access facility that serves the Greater Pittsburgh Area and that was opened to traffic in 1953. The Parkway extends from the west in the vicinity of the Greater Pittsburgh Airport through the Fort Pitt Tunnel, across the Monongahela River on the Fort Pitt Bridge, along the southern fringe of the Golden Triangle, through the Squirrel Hill Tunnel, and to the east to connect with major highways to Harrisburg, Philadelphia, and beyond (see Fig. 1).

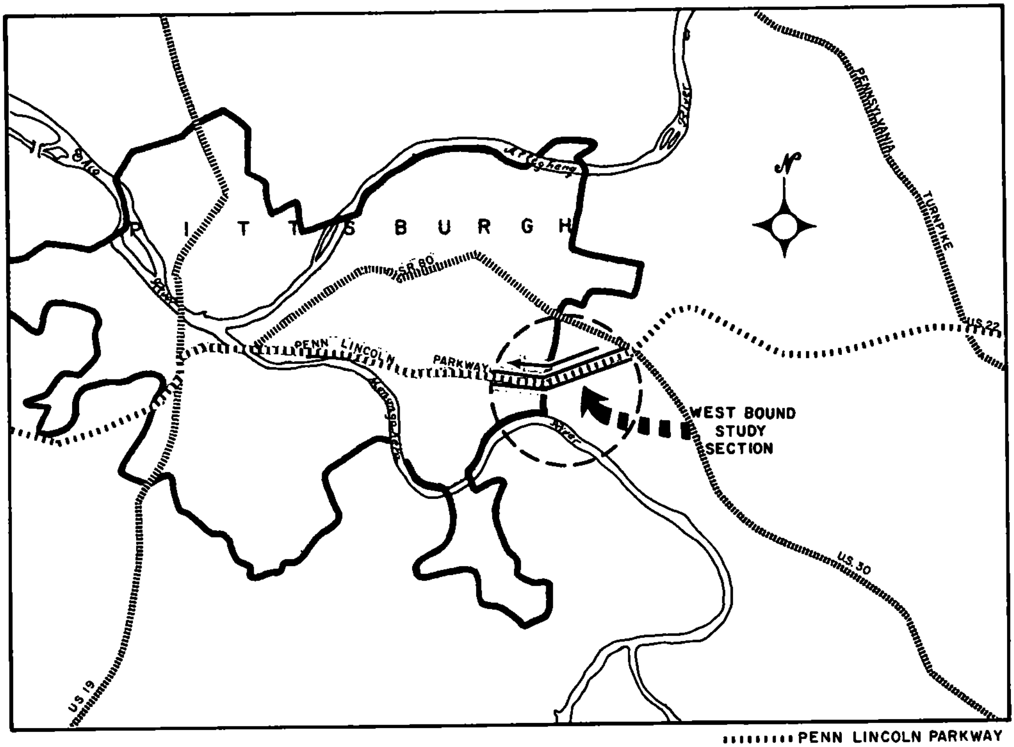


Figure 1. Penn-Lincoln Parkway in Greater Pittsburgh.

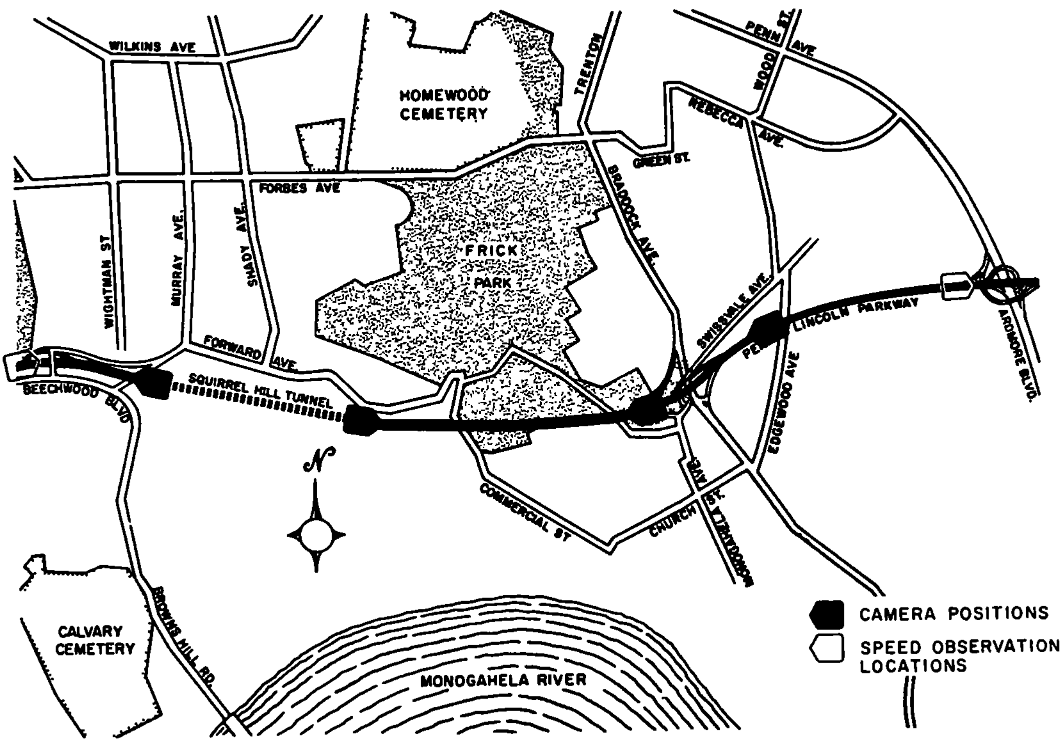


Figure 2. Study section on Penn-Lincoln Parkway.

The $3\frac{1}{2}$-mi study section extends from Ardmore Boulevard on the east to Greenfield Bridge on the west, as shown in Figure 2, and includes the Squirrel Hill Tunnel. There are two westbound lanes on the study section, separated from eastbound traffic by a narrow median. The route is on a rolling profile with a 2 to 5 percent downgrade from Ardmore Boulevard to Braddock Avenue, a 1 to $2\frac{1}{2}$ percent upgrade from Braddock Avenue through the Tunnel, and then a 4 to 5 percent downgrade. There are two on-ramps at Ardmore Boulevard, two on-ramps and an off-ramp at Braddock Avenue, an off-ramp near the exit from the Squirrel Hill Tunnel, and an on-ramp near the Greenfield Bridge where the roadway is widened to three westbound lanes. The design features receiving special attention were the proximity of two on-ramps at the foot of the 1.7-mi upgrade, the 1.7-mi upgrade, and the Squirrel Hill Tunnel near the crest.

The annual average daily traffic volume through the Squirrel Hill Tunnel has increased from 47,700 in 1957, to 56,300 in 1960. This traffic demand, particularly during the morning peak period in the westbound direction, has resulted in congestion that is of 1- to 2-hr duration and extends for distances of 2 to 3 mi each weekday morning.

Public reaction to this continuing problem demanded that prompt action be taken to reduce delay and decrease the accident potential on the approach to the Tunnel area. Considerable controversy existed as to the best method of meeting the situation.

TIME-LAPSE PHOTOGRAPHIC AND TRAVEL TIME STUDIES

Time-lapse photographic techniques were employed, and four intervalometer cameras were operated from selected ground locations along the study section (see Fig. 2). The camera locations were selected so that traffic flow at key portions downstream, upstream, and at the congestion points was recorded. At each location photographs were taken at a rate of 1 frame per sec for a 2-hr period (7 to 9 a.m.) for four normal weekdays (May 1 to 4, 1961). The 2-hr period included the photographing of traffic flow before, during, and after congestion.

A projector especially adapted for frame-by-frame analysis was employed to obtain volume, speed, and density on a per-lane basis and for 1-min intervals. Vehicle speeds and densities were sampled every 15 sec in each lane. Total volume, truck, volume, speed, and density measurements for each minute of the study, for each lane, for each camera location, and for two selected days (Wednesday and Friday, May 3 and 5) were punched on cards for machine analysis. Parkway volumes, percent trucks, and calculated densities were computed and punched on the same cards. Each card represents a 1-min interval at a particular camera for a specific day and there are 960 cards (2 days x 120 min x 4 cameras).

The specific analyses undertaken for the photographic data included speed-volume-density, profiles, speed-volume-density interrelationships, comparison of lane traffic characteristics, time headway distributions, and estimation of traffic demand.

Speed-Volume-Density Profiles

Time profiles of speeds, volumes, densities, and percent trucks for each of the four camera positions for May 3 (Wednesday) and May 5 (Friday) were prepared, and one such graph is shown as Figure 3. The two days of observations were placed on the same graph for each camera position, and indicate that traffic characteristics on the two days selected are very similar as to magnitudes and time of occurrences. Each plotted point represents 1 min of observation of a particular characteristic for the two westbound lanes, and the lines on the graphs indicate trends and are not mathematically fitted curves. The graphs indicate that periods of time before, during, and after congestion were analyzed, and that the data collection stations selected were located both upstream and downstream of congestion.

Distance profiles of speeds, volumes, densities, and percent trucks of May 3 (Wednesday) and May 5 (Friday) were prepared, and one such graph is shown as Figure 4. In addition to the four camera positions, speed measurements were obtained at a point upstream (Ardmore ramp overpass) and at a point downstream (Greenfield Bridge) and are shown on these graphs as locations 1 and 6 respectively. Each profile represents a 1-min interval, and five profiles are plotted on each sheet.

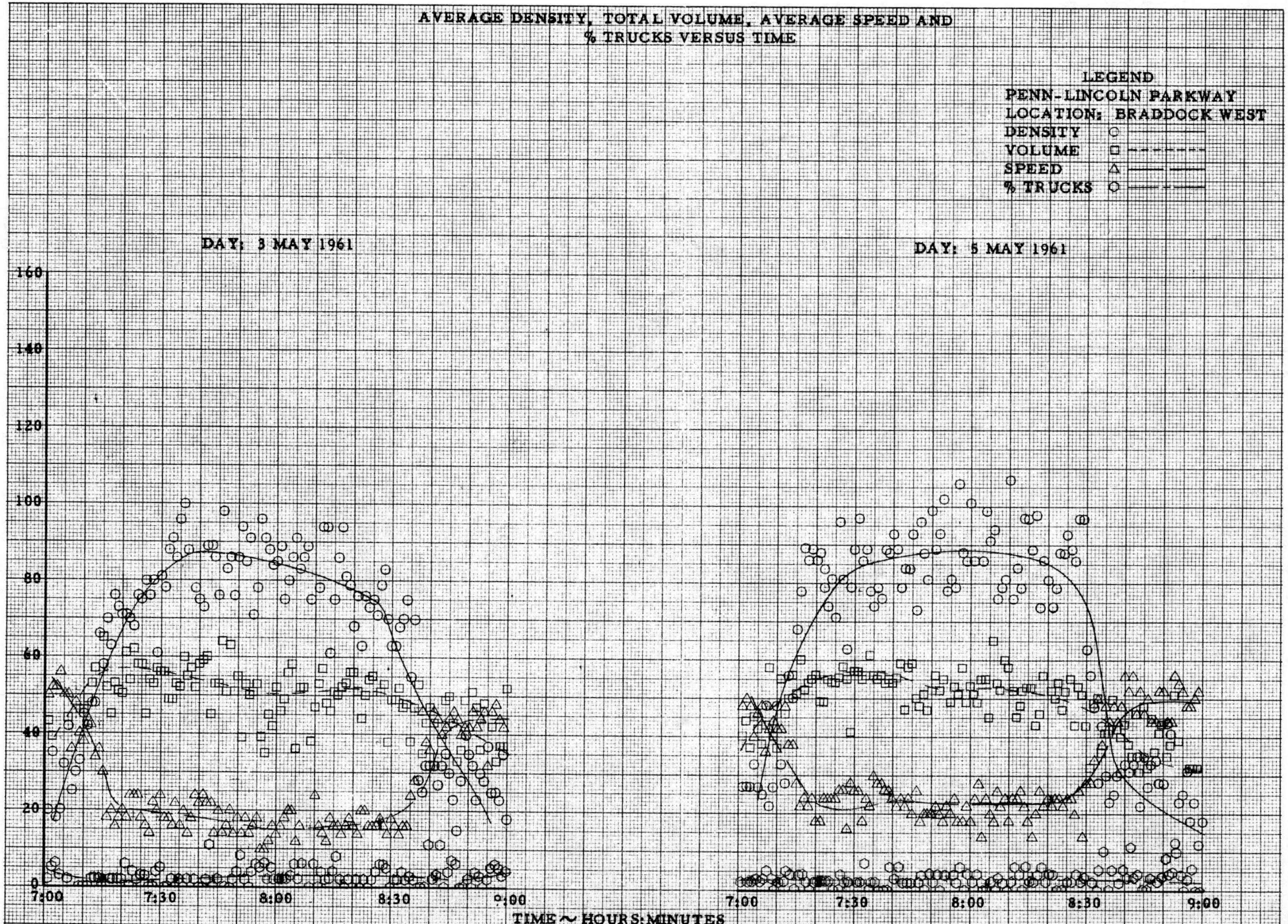


Figure 3. Sample of speed-volume-density time profiles.

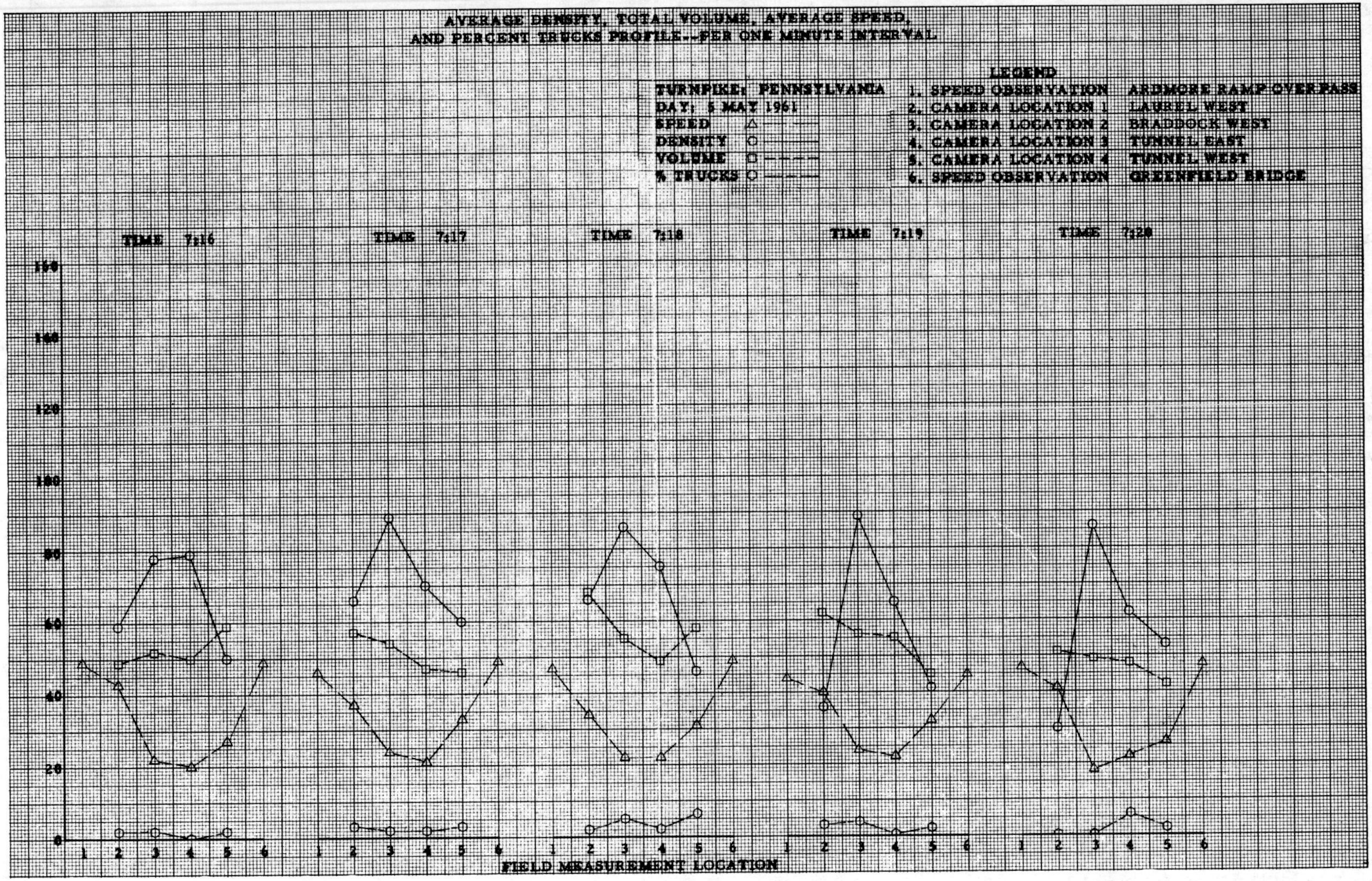


Figure 4. Sample of speed-volume-density distance profiles.

Speed-Volume-Density Interrelationships

Speed-volume, volume-density, and speed-density relationships were investigated to determine (a) the combination of speed and density when volume was maximum; (b) the effect of location (upstream of, at, or downstream from critical controlling point) on the relationships; and (c) if the two study days gave similar results.

The speed-volume relationships for each of the four camera positions and for the two study days were determined and are summarized in Figure 5. Each plotted point represented 1 min of observations and there were 120 points on each graph. The results obtained from the two days are quite similar for each camera position. The curves attempt to represent the enclosure or limits of the speed-volume relationships. The left portion of the figure shows enclosure curves or limits, and on the right side of the figure shows average curves. The diagonal lines sloping upward to the right are average density lines with the steeper slopes representing the lower density levels and the flatter slopes representing the higher density levels. Maximum volumes of 65 vehicles per min occurred at speeds of 25 to 35 mph and densities of 60 to 80 vehicles per mi. The speed-volume curves for Laurel West and Braddock West are quite similar and are typical for locations upstream from a congestion point. The curve for the Tunnel East location is 5 to 10 mph lower than the two curves farther upstream where the study section is not congested (lane densities less than 60 vehicles per mi), which indicates the influence of the tunnel even under light to moderate traffic loads. The curve for Tunnel West is typical for locations just downstream from a congestion point inasmuch as the lower portion of the curve does not slope down and to the left. This is significant because it implies that the congestion point or bottleneck is between Tunnel East and Tunnel West locations.

Volume-density relationships for each of the four camera positions and for the two study days were determined and a summary is given as Figure 6. Each plotted point represents 1 min of observations, and the curve is hand-drawn to indicate the average trend between volume and density. The average curves are shown in the upper diagram, and the enclosure curves are shown in the lower diagram. The radial lines sloping up and to the right indicate average speeds. The same types of conclusions can be drawn from this relationship as were drawn from the speed-volume curves. Maximum volumes occurred at speeds of 25 to 35 mph and at lane densities of 60 to 80 vehicles per mi.

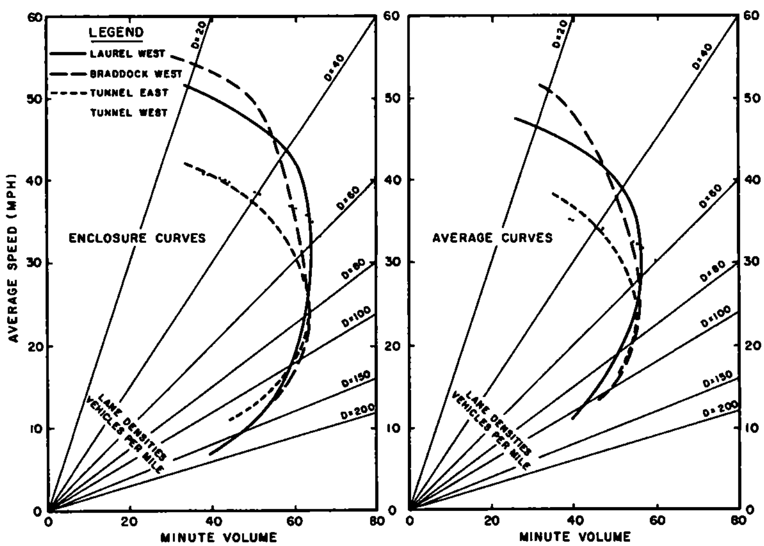


Figure 5. Summary of speed-volume relationships.

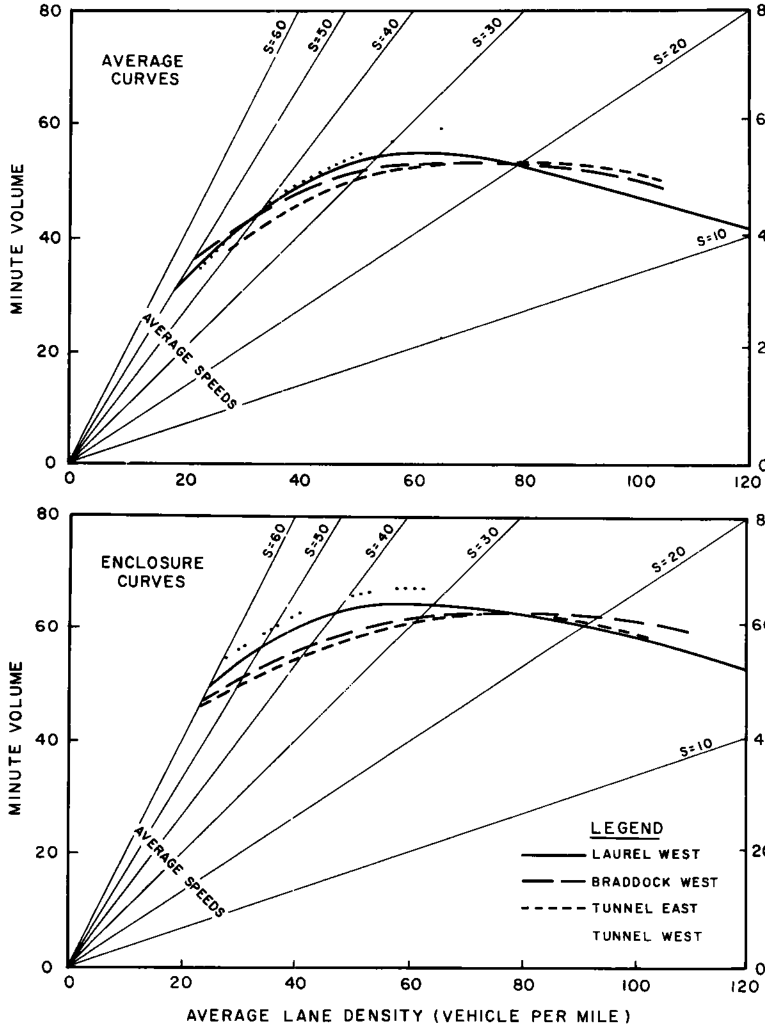


Figure 6. Summary of volume-density relationships.

Speed-density relationships for each of the four camera positions and for the two study days were determined and are shown in Figure 7. Each plotted point represents 1 min of observations, and the curve is hand-drawn to indicate the average trend between speed and density. The average curves are shown in the top diagram and the upper and lower enclosure curves are shown in the graph below. The light lines are indicative of volume rates of 30, 40, 50, 60, and 70 vehicles per min. At the first two locations maximum volume rates occurred at lane densities of 60 to 70 vehicles per mi and speeds of about 30 mph. The maximum volume rate at the entrance to the tunnel occurs at lane densities of about 80 cars per mi and a speed of 20 mph, but the maximum volume rate was not obtained at the tunnel exit.

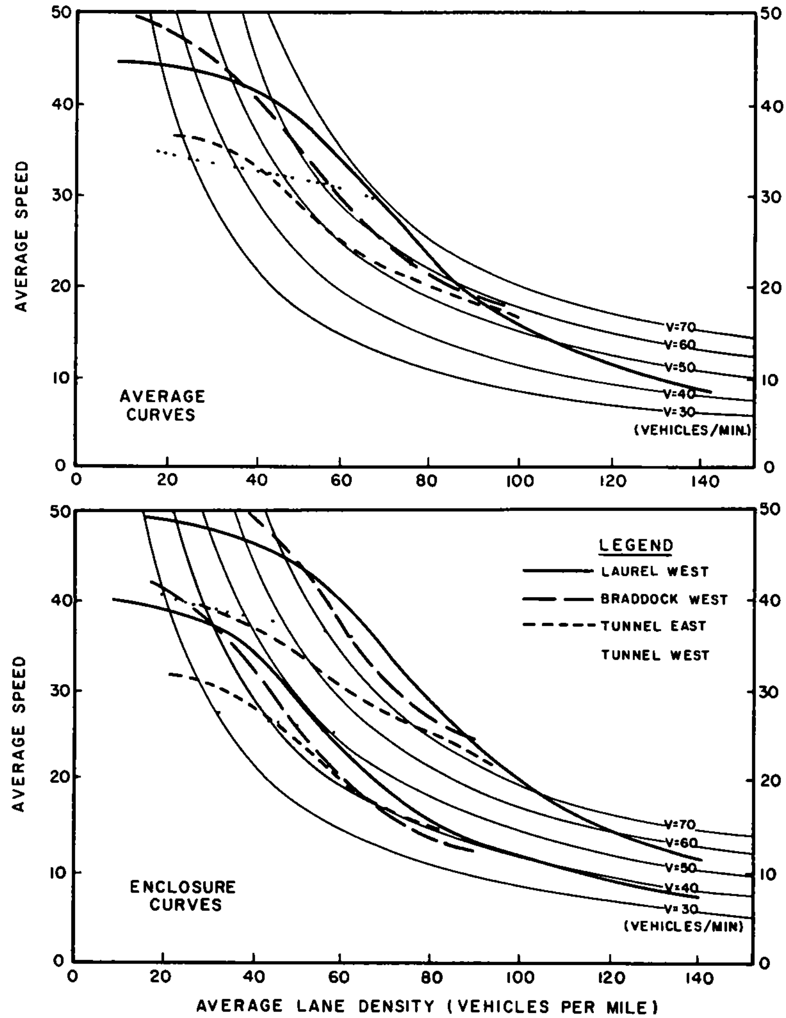


Figure 7. Summary of speed-density relationships.

Comparison of Lane Traffic Characteristics

The analysis procedure followed in the study permitted the comparison of lane traffic characteristics such as volume, speed, and density. As in the analysis presented earlier, the results of each study day and for each of the four camera positions are included.

Lane volume as related to total parkway volume by day and by location were determined. A summary comparing these lane volumes is given in Figure 8. The upper diagram is for the median lane and the lower diagram is for the shoulder lane. The shaded area represents when the indicated lane volume is less than the other lane volume. The median lane at all four locations throughout the morning peak traffic period carried more traffic than the shoulder lane. It appeared that when congestion seriously reduced parkway volume, the shoulder lane carried only 35 to 40 percent of the total volume. As the traffic volume moved from the upstream locations to the downstream locations (as the distance up the grade increased) the shoulder lane carried less and less of its share.

Lane speed related to average parkway speed by day and location was determined. A summary comparing lane speeds is shown in Figure 9. The upper portion of the diagram is for the median lane and the lower graph depicts the shoulder lane. The shaded areas indicate when the median or the shoulder lane speeds are below the average parkway speed. When parkway speeds were over 38 mph, the median lane speeds at all four locations were greater than the shoulder lane speeds. When parkway speeds were under 20 mph, the median lane speeds at the three locations upstream from the tunnel were less than the shoulder lane speeds. The median lane speeds at the tunnel exit were always 5 to 10 mph greater than the shoulder lane speeds. It appears that the reason for this speed pattern is that drivers approaching the tunnel are willing to travel at lower speeds in the median lane due to high density in order to travel through the tunnel section at the higher median lane speeds, with a net saving in time.

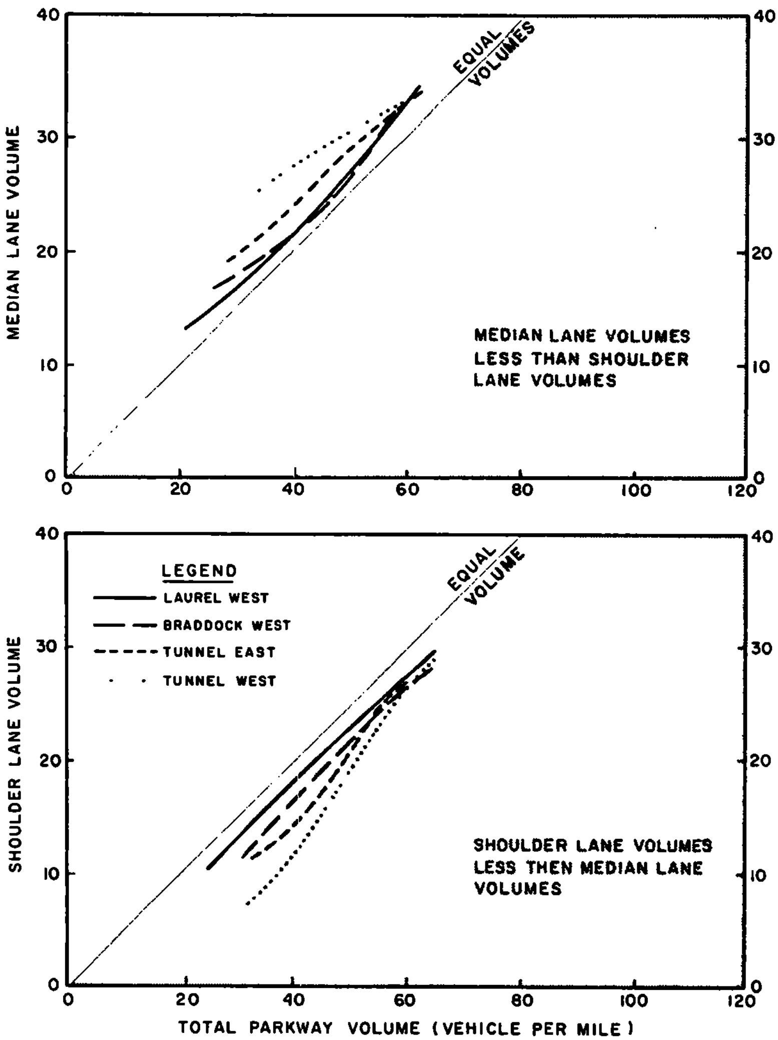


Figure 8. Comparison of lane volumes.

Lane density related to average parkway density by day and location was determined and a summary comparing lane densities is shown in Figure 10. The upper diagram is for the median lane and the lower diagram is for the shoulder lane. The shaded area indicates when the median or shoulder lanes are below the average parkway density. The median lane density is always greater than the shoulder lane density at the three locations upstream from the tunnel, and this is also true for the tunnel exit location for average parkway densities less than 60 vehicles per mi. However, when there is congestion in the tunnel (average parkway densities over 60 vehicles per mi), the shoulder lane has a higher density than the median lane. It would appear that when the tunnel is congested, the shoulder lane traffic does not recover from dense flow as rapidly as the median lane.

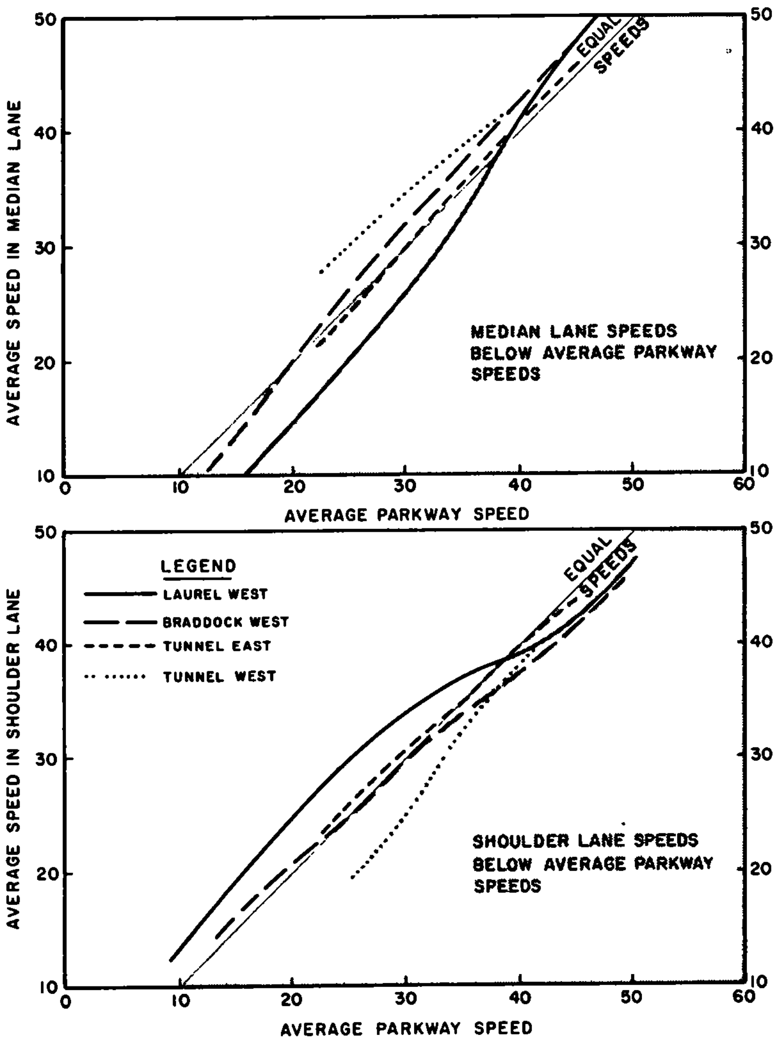


Figure 9. Comparison of lane speeds.

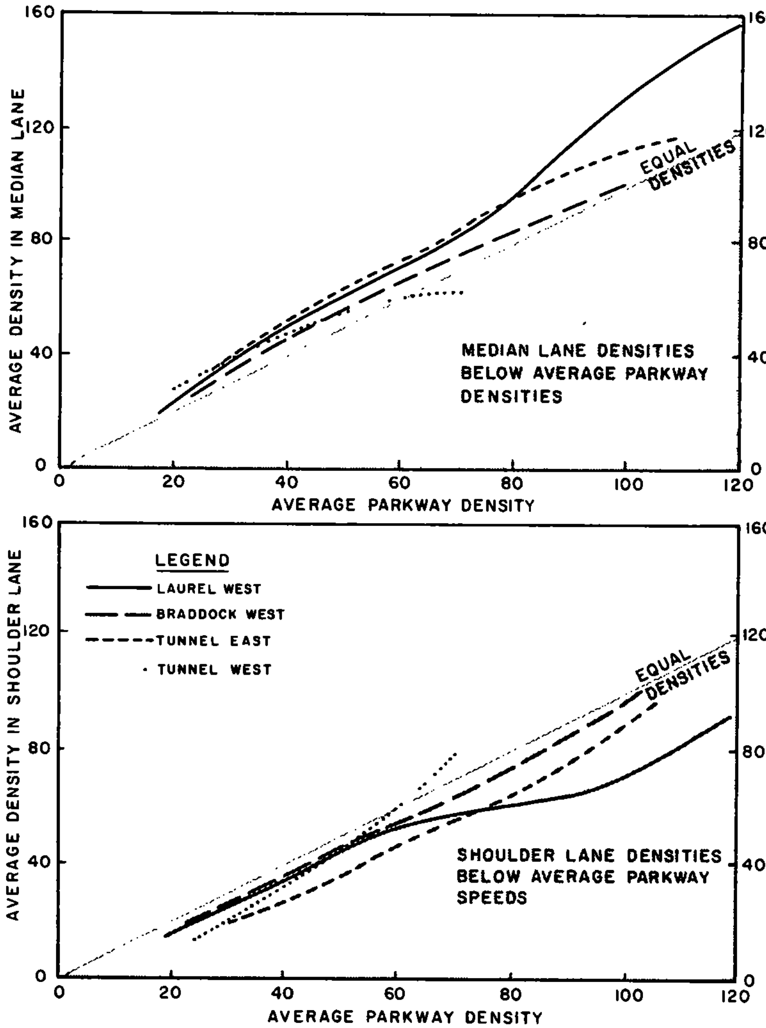


Figure 10. Comparison of lane densities.

Morning Peak Summary of Volumes, Trucks and Speeds

Total volume, truck volume, and average speed by 15-min, 30-min, 1-hr, and 2-hr periods for each camera position, and on an individual lane and parkway basis was ascertained. The total 2-hr volume at the tunnel exit on May 3 and May 5 was 5,991 and 6,008 vehicles respectively. This compares very closely with the total volume recorded at the permanent count station, which was 6,095 and 6,054 vehicles for the same period. Whereas the 7:00 to 8:00 and 8:00 to 9:00 a.m. volumes for the two days ranged from 2,959 to 3,049, the selection of the 1-hr period from 7:30 to 8:30 a.m. resulted in hourly volumes of 3,228 and 3,247 vehicles.

There were a total of 142 vehicles (2.4 percent) on May 3 and 129 vehicles (2.1 percent) on May 5 having six or more tires and these were classified as trucks. The average speeds for the individual camera positions during the morning peak periods for the two days varied from 25 to 33 mph with over-all average speeds on the study section of 28 to 48 mph.

Investigation of Time Headways

The time-lapse photography permitted the measurement of individual time headways and in addition gave the opportunity of identifying vehicles by type and by lane. The films taken from Tunnel West, the camera position at the top of the grade, were analyzed to determine the effect of vehicle type and lane on time headways, and conse-

TABLE 1

TIME HEADWAY DISTRIBUTIONS

Time Headway	TY Type															
	Median Lane		Shoulder Lane		Median Lane		Shoulder Lane		Median Lane		Shoulder Lane		Median Lane		Shoulder Lane	
	Wed.	Fri.	Wed.	Fri	Wed	Fri.	Wed.	Fri	Wed	Fri	Wed	Fri	Wed	Fri	Wed	Fri
1	713	724	378	355	8	7	47	38	2	3	3	5			1	3
2	692	781	598	562	7	7	39	43	2	3	14	17		2	5	3
3	227	213	149	179	1		8	13	4	6	17	18			6	5
4	65	61	45	47	2	1	2	2	4	4	16	12	1		4	1
5	27	19	12	15	1	1			3	2	10	6			2	1
6	13	10	3	9					1		6	11	1			
7	8	7	5	3						1	4	3				
8	3	1	3	1							1					
9	1	2	1	3	1					1	2	1				
10		2		4							1	2				
11	2	1	1	1							3	1				
12		1										1				
13	1		1						1		1					
14	1															
15											4					
16											1					
17																
18									3		1	1				
19		1									2					
20											1	1				
over 20			3								6	13[a]			1	

[a]Maximum gap measured 96 sec.

TABLE 2

AVERAGE TIME HEADWAYS

Vehicle Type	Wednesday						Friday						Wednesday and Friday					
	Median		Shoulder		Both		Median		Shoulder		Both		Median		Shoulder		Both	
	Avg	Freq.	Avg	Freq	Avg	Freq	Avg	Freq.	Avg	Freq	Avg	Freq	Avg	Freq	Avg.	Freq	Avg	Freq
PP	1.93	1,753	2.09	1,199	2.00	2,952	1.90	1,823	2.07	1,179	1 97	3,002	1.92	3,576	2.08	2,378	1 98	5,954
TP	2 35	20	1.64	96	1 76	116	1.88	16	1.78	96	1 79	112	2 14	36	1.70	192	1 78	228
PP+TP	1.94	1,773	2.06	1,295	1.99	3,068	1.90	1,839	2 05	1,275	1 96	3,114	1.92	3,612	2 05	2,570	1 97	6,182
PT	6.1	20	8 98	93	8 47	113	3 45	20	8 99	92	8.00	112	4 78	40	8.94	185	8 20	225
TT	5.0	2	7 21	19	7.0	21	2 00	2	2.54	13	2 47	15	3.50	4	5 31	32	5 11	36
PT+TT	6.0	22	8.68	112	8.24	134	3 32	22	8.19	105	7.35	127	4 66	44	8.40	217	7 77	261
PP+TP+PT+TT	1.99	1,795	2.59	1,407	2.25	3,202	1.92	1,861	2.52	1,380	2.17	3,241	1 95	3,656	2 55	2,787	2 21	6,443

quently is indicative of capacity. Headway distributions by day, lane, and vehicle combinations are shown in Table 1. The average headways and the frequency of headway observations between 7:30 and 8:30 a. m. (the period of greatest congestion) were determined and these values are summarized in Table 2. Vehicle type is identified by two symbols, with the first symbol representing the lead vehicle, and the second symbol representing the following vehicle. The letter P stands for passenger car, and the the letter T stands for truck. For example, PT means that a passenger car is being followed by a truck.

The over-all average time headway for all vehicles was 2.21 sec or a volume rate of 3,260 vehicles per hr. The average time headway for passenger vehicles only was 1.98 sec or a volume rate of 3,640 vehicles per hour. The difference, 380 vehicles per hr, indicates the effect of a relatively small percentage of heavy vehicles in the traffic stream. In comparing time headways between lanes, the passenger car time headways were 1.92 (1,875 vehicles per hr) and 2.08 (1,731 vehicles per hr) in the median and shoulder lanes respectively. The time headway for all vehicles in the median lane under existing conditions is 1.95 sec (1,845 vehicles per hr). In other words, the 22 trucks in the median lane reduce the lane capacity by at least 30 vehicles per hr. In their absence, 52 passenger vehicles could be accommodated.

In order to estimate the increase in capacity that might result if lane changing in the tunnel was permitted, it was assumed that a vehicle during high density flow would change lanes if a 6-sec headway existed (a distance headway between 260 to 350 ft). Using this criteria, the number of 6-sec headways was found to be 132, providing at least this number of lane change opportunities with resultant capacity increase.

Estimation of Traffic Demand

The number of vehicles passing through a bottleneck during congestion is indicative of how many vehicles can get through, but does not indicate how many vehicles would like to pass through the bottleneck. The increase in density upstream from the bottleneck is essentially a measure of the difference between maximum volume and traffic demand. Because volumes and speeds were measured at several upstream locations, density could be calculated and therefore the existing traffic demand estimated. It should be emphasized that the approach presented is an approximation and that this technique permits the estimation of present traffic demand only.

The tabulations required in estimating demand are not contained in this paper but included volume and speed observations at each of five locations and computed average lane densities for each 15-min period; and calculations needed to determine excess densities by 15-min intervals and for four upstream subsections. Because vehicle speeds were essentially unaffected until lane densities approached 40 vehicles per mi, it is reasonable to assume that at this density level the parkway is filled but no vehicles are being stored. When the lane density is in excess of 40 vehicles per mi, the difference between volume and traffic demand can be computed by the equations:

$$S_i = N_i L_i(\overline{D}_i - D_c)$$

and

$$S = \Sigma_i L_i(\overline{D}_i - D_c)$$

in which S_i = number of cars stored in subsection$_i$; S = total number of cars stored; N = number of lanes; L = length of section in miles; $\overline{D}$ = average lane density (vehicles per mile); and D_c = critical lane density (D_c = 40).

The traffic demand for each selected time interval can be computed by the equations:

$$(T.D.)_{T_1} = V_1 + \frac{S_1}{T}$$

$$(T.D.)_{T_2} = V_2 + \frac{S_2 - S_1}{T}$$

$$(T.D.)_{T_3} = V_3 + \frac{S_3 - S_2}{T} \quad \text{etc.}$$

in which

T. D. = traffic demand (vehicles per minute);
V = traffic volume at the bottleneck (vehicles per minute);
S = total number of cars stored; and
T = time interval.

These calculations serve as a basis for determining total vehicles stored, storage rates, and estimated present traffic demand.

Figure 11 shows these results, and can be used to estimate the effect of increased capacity on congestion. The lower boundary of the shaded area represents accumulated traffic volume, the upper boundary of the shaded area accumulated present traffic demand, and the shaded area shows the number of vehicles stored. It can be readily seen from the graph that under existing conditions congestion (vehicles stored) lasts for 1 hr 35 min, and the maximum number of vehicles stored was 220 and occurred at 8:15 a. m.

The effect of increased capacity on congestion is shown by the dashed line. If the volume rate could be increased from 52 to 58 vehicles per min, the duration of congestion would be reduced to 40 min and the maximum number of cars stored would be reduced to 30. The graph also indicates that if the volume rate was increased to 60 vehicles

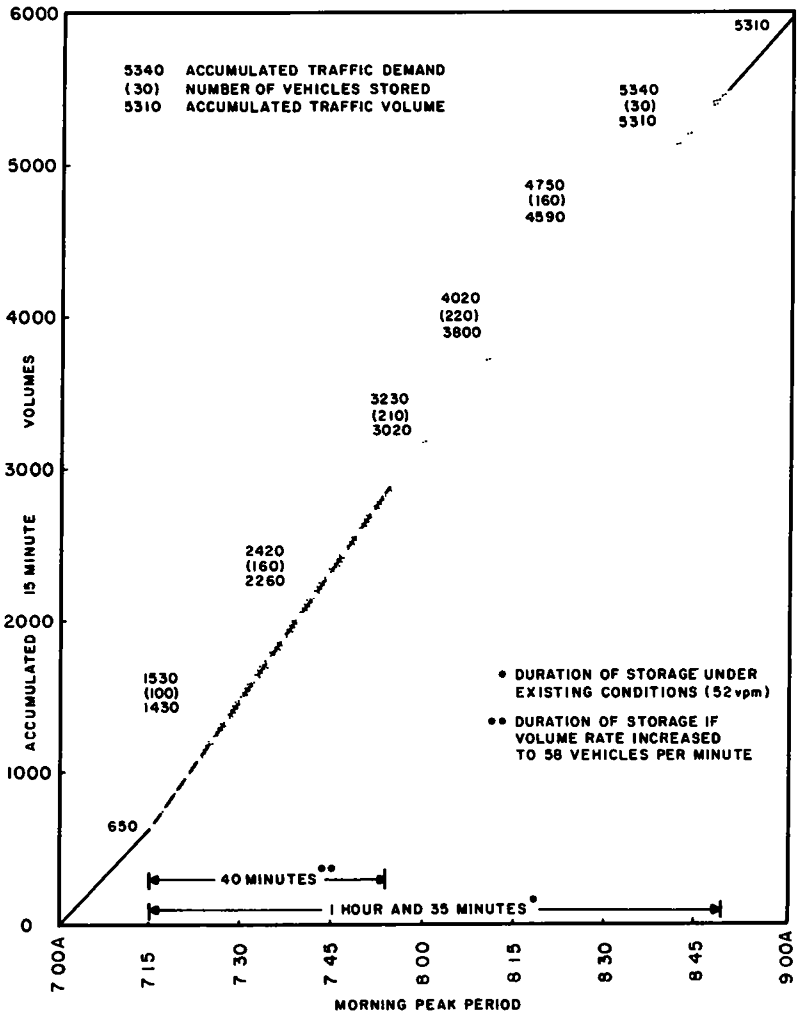


Figure 11. Effect of increased capacity on congestion.

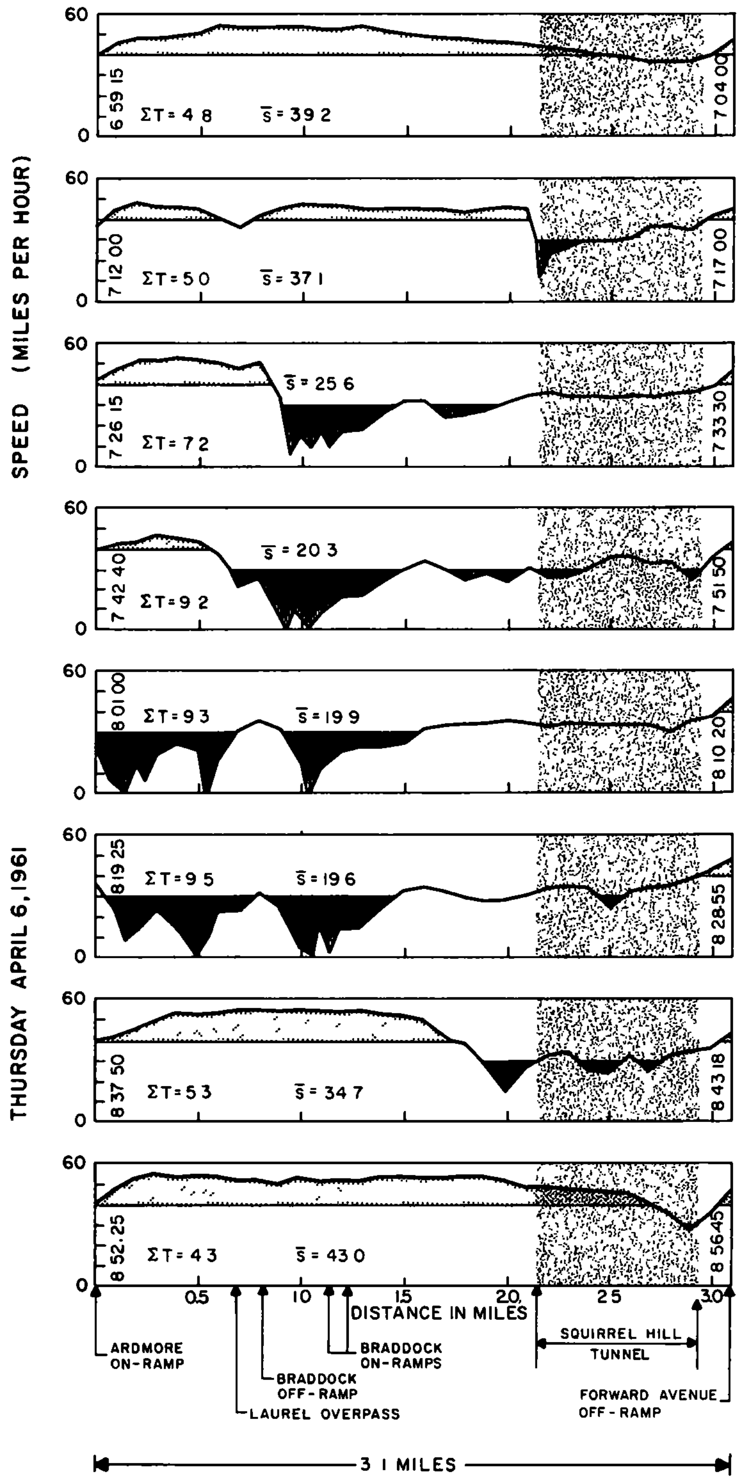
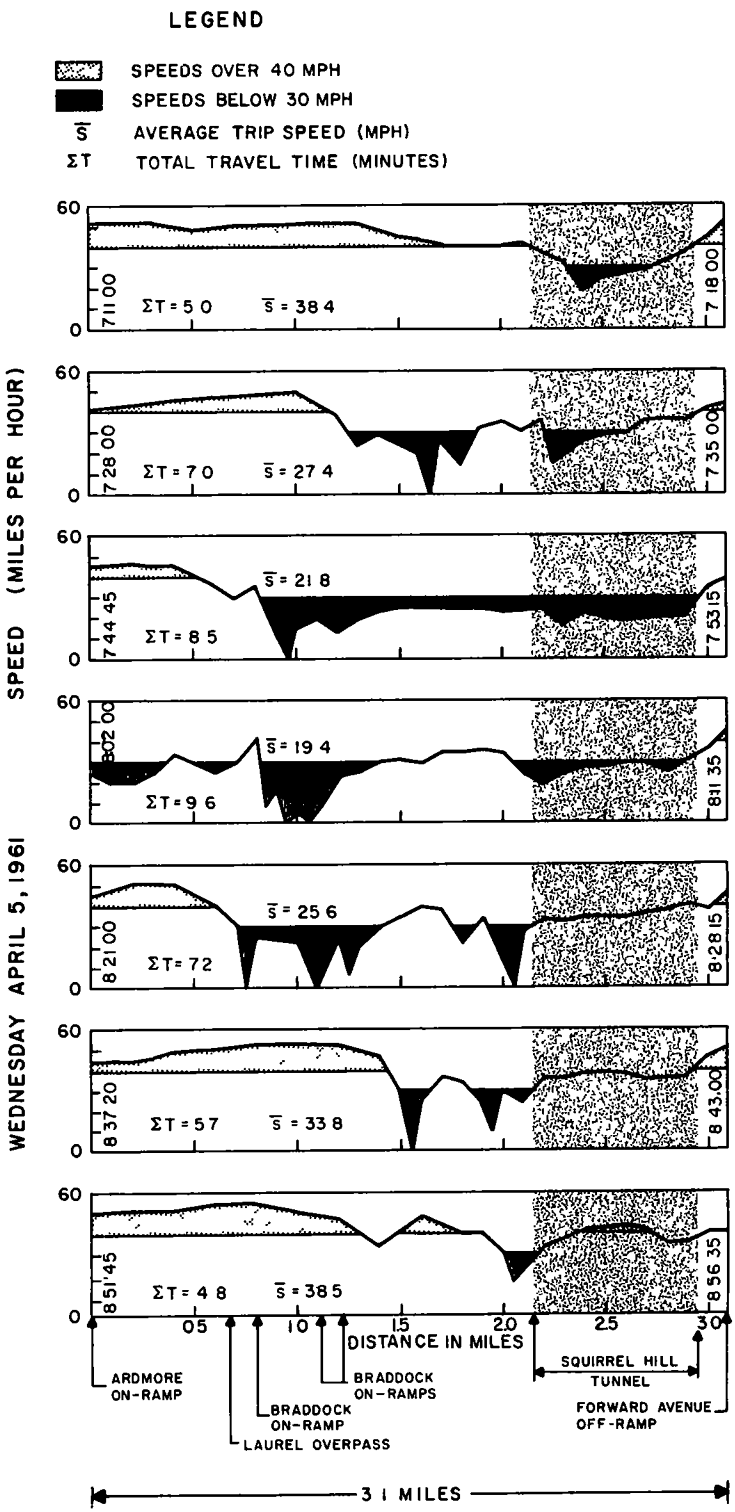


Figure 12, Tra


LEGEND
SPEEDS OVER 40 MPH
SPEEDS BELOW 30 MPH
S̄ AVERAGE TRIP SPEED (MPH)
ΣT TOTAL TRAVEL TIME (MINUTES)
SPEED (MILES PER HOUR)
WEDNESDAY APRIL 5, 1961
7 11 00
ΣT = 5 0
S̄ = 38 4
7 18 00
7 28 00
ΣT = 7 0
S̄ = 27 4
7 35 00
7 44 45
S̄ = 21 8
ΣT = 8 5
7 53 15
8 02 00
S̄ = 19 4
ΣT = 9 6
8 11 35
8 21 00
S̄ = 25 6
ΣT = 72
8 28 15
8 37 20
ΣT = 57
S̄ = 33 8
8 43.00
8 51 45
ΣT = 4 8
S̄ = 38 5
8 56 35
60
0
05
10
15
20
25
30
DISTANCE IN MILES
ARDMORE ON-RAMP
BRADDOCK ON-RAMPS
BRADDOCK ON-RAMP
LAUREL OVERPASS
SQUIRREL HILL TUNNEL
FORWARD AVENUE OFF-RAMP
3 1 MILES

ed profiles.

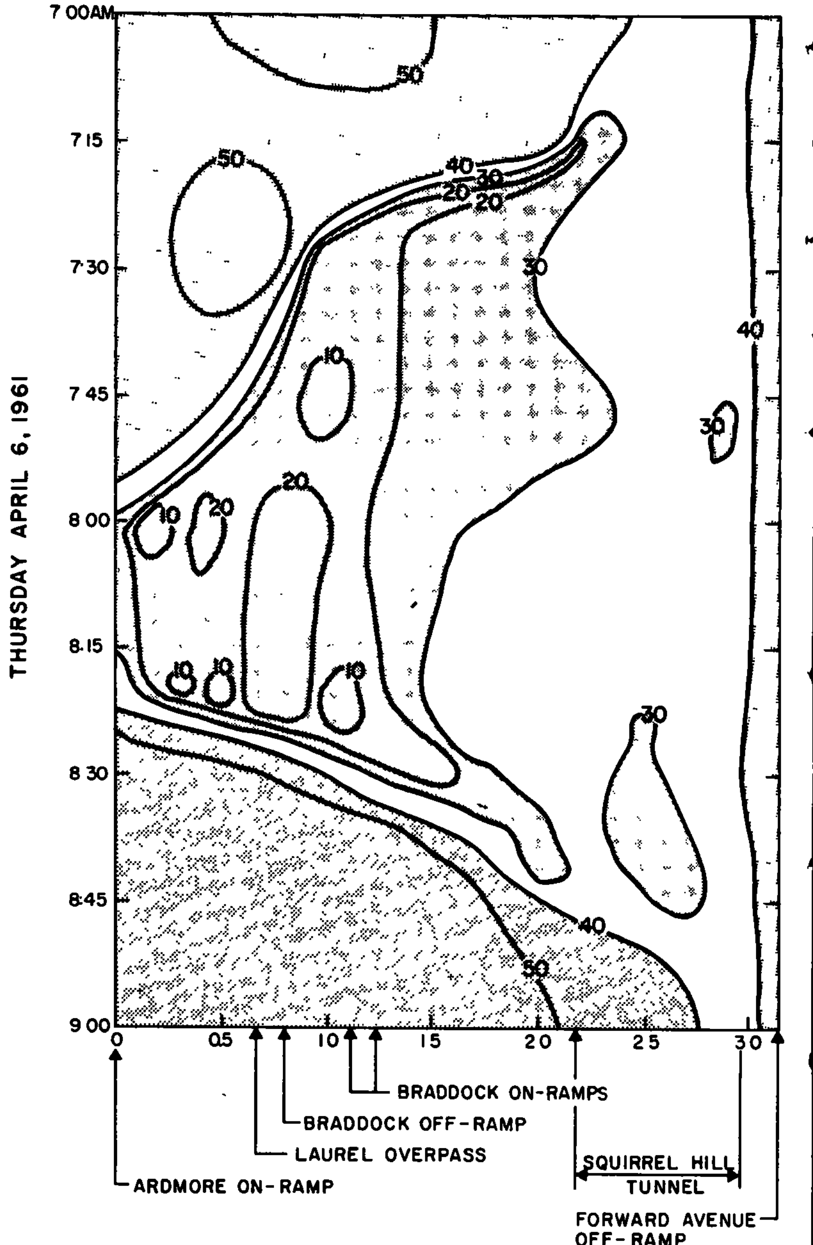
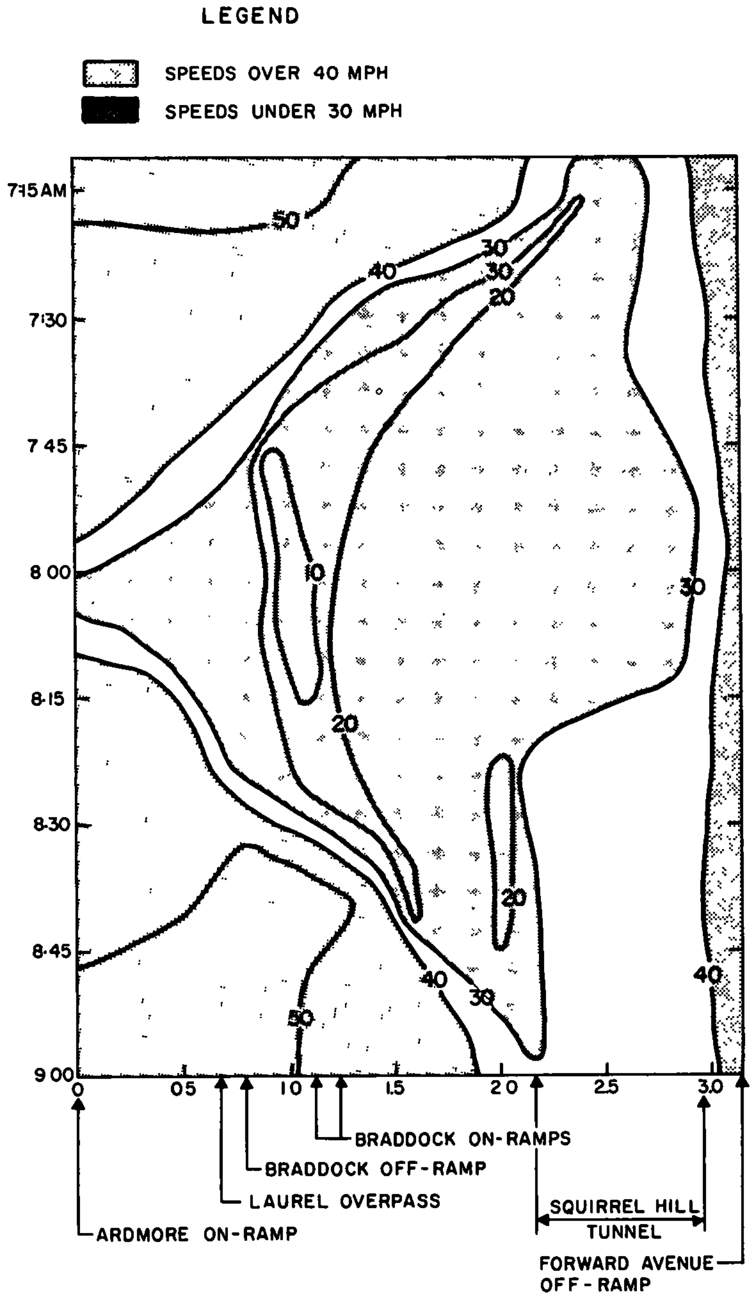


Figure 13. Tra



ed contour maps.

per min no congestion would occur. However, this assumes that the traffic demand is uniform and individual minutes do not exceed 60 vehicles per min. Discussion later in the report will present methods for regulating traffic demand to insure uniformity of flow and better use of the study section.

Travel Time Study

A modified travel time study was conducted on Wednesday and Thursday, April 5 and 6, 1961 on the 3-mi study section from Ardmore Boulevard to the Foreward Avenue off-ramp. In addition to noting the beginning and ending time on each of the 15 test runs, instantaneous speeds were recorded at 0.2-mi intervals when speeds were over 30 mph and at 0.1-mi intervals when speeds were below 30 mph.

The travel speed profiles for the 15 test runs are shown in Figure 12, the beginning and ending time for each test run is indicated as well as total travel time. Speeds over 40 mph were assumed to be satisfactory (no congestion), and speeds below 30 mph were assumed to be unsatisfactory. The light shaded areas indicate portions of the test runs where speeds were satisfactory, and the dark shaded areas indicate portions where congestion was encountered. Average trip speeds varied from 19 to 43 mph (total travel times of 4.3 to 9.6 min).

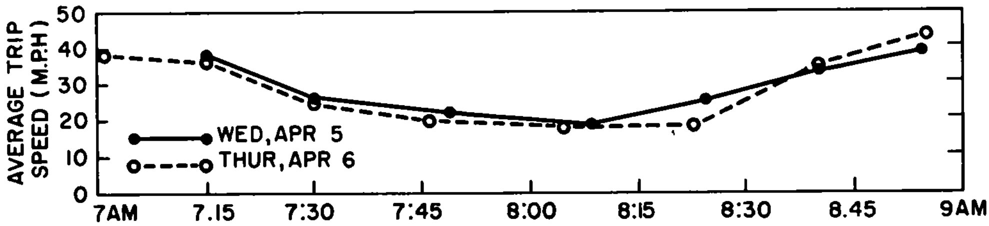
This modified travel time study technique permits a rather intriguing and fact-revealing means of studying congestion. Travel speed contour maps for the two days of study are presented in Figure 13. For each test run where the instantaneous speed changed from one 10-mph group to another (i.e., from 40-49 to 30-39 mph group) the time and location were indicated on this time-distance map. Contours were then drawn connecting points of equal speed. Again speeds over 40 mph were assumed to be satisfactory while speeds below 30 mph were assumed to be indicative of congestion. The lightly shaded area represents periods of time and portions of the study section when speeds were satisfactory, and the darker shaded areas denote periods of time and portions of the study section where congestion existed.

Certain important observations can be made from these contour maps.

1. The travel speed contour maps showing congestion for the two study days are very similar.
2. The congestion on both days originated in the vicinity of the Squirrel Hill Tunnel entrance.
3. The portion of the Penn-Lincoln Parkway downstream from the Squirrel Hill Tunnel was free of congestion throughout the morning peak period and gives evidence of additional available capacity.
4. Congestion began at approximately 7:10 a.m. and ended at about 8:50 a.m. resulting in a congested period of 1 hr 40 min.
5. Stop-and-go traffic conditions extended upstream from the Squirrel Hill Tunnel to east of Ardmore Boulevard (a distance in excess of 2 mi).
6. Shock waves moved upstream at velocities of 4 to 7 mph, while recovering shock waves had velocities of 3 to 6 mph.

The total travel time and the average trip speed was determined for each test run in order to calculate the increased travel time and costs due to congestion. An average trip speed profile for the period 7:00 to 9:00 a.m. for each day is shown in the upper portion of Figure 14, and the graph reveals that the profiles for the two days are quite similar. Knowing the total travel times and calculating the average traffic volume for each of the 15-min periods, the total vehicle hours expended can be computed. The excess travel time due to congestion can be determined by subtracting the total vehicle hours expended if congestion did not occur from the existing vehicle hours expended. An assumed noncongested speed of 40 mph was selected; however, the graph in the lower portion of Figure 14 permits the reader to enter with any assumed noncongested speed between 30 and 50 mph. If the 40-mph value is used, the daily vehicle-hours saved would be 260.

It is difficult to determine the exact value of time to the motorist, but vehicle hourly rates of \$1.00 to \$2.00 per hr are commonly used. The graph in the lower portion of



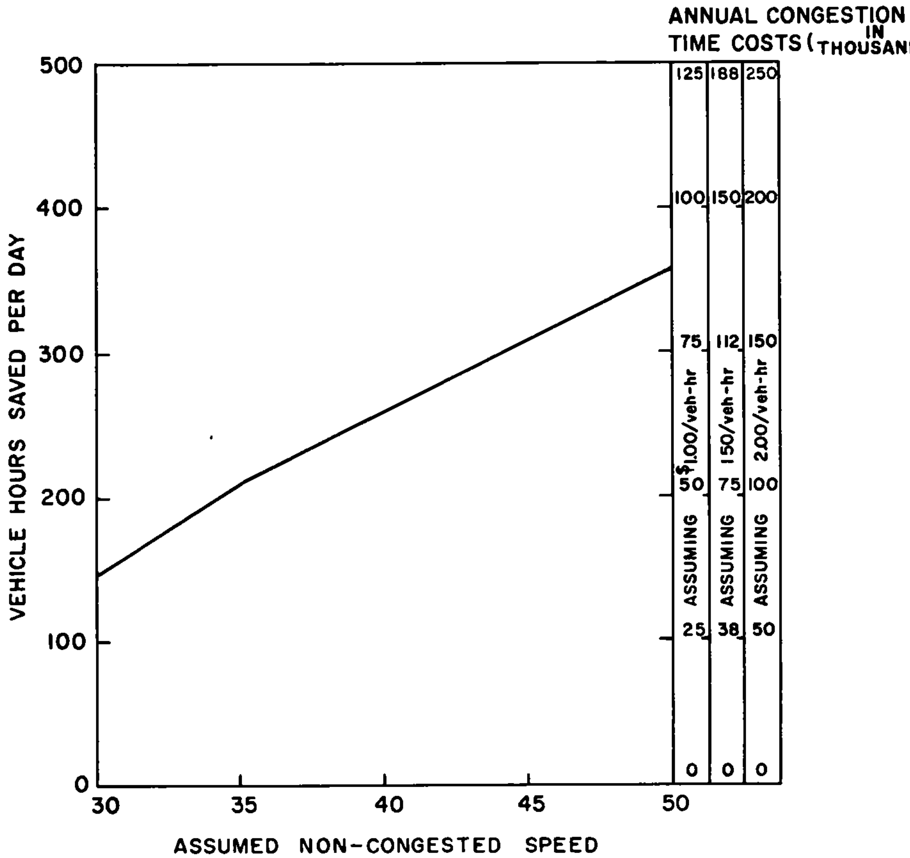


Figure 14. Congestion time costs.

Figure 12 is so constructed that annual congestion time costs (in thousands of dollars) can be read from the graph when hourly rates of $1.00, $1.50, or $2.00 per vehicle-hour selected. For example, the annual congestion time cost is $97,000 when 40-mph noncongested speed is selected and a vehicle hourly rate of $1.50 is used. Future traffic increases will undoubtedly substantially increase the annual congestion time costs.

SUMMARY

Identification of Critical Locations

The results of the speed-volume-density profiles, speed-volume-density relationships, and the travel speed contour maps clearly indicate that the critical location on the study section is at the Squirrel Hill Tunnel. Congestion originates near the tunnel entrance and restricts the capacity of the study section. This restriction to traffic flow transmits speed and density shock waves upstream as far as Ardmore Boulevard, and also reduces speed throughout the length of the tunnel. The portion of the Penn-Lincoln Parkway from the Squirrel Hill Tunnel toward downtown Pittsburgh is not being fully utilized because of the restriction to traffic flow at the tunnel.

Determination of Causes of Congestion

The restriction to traffic flow near the entrance to the Squirrel Hill Tunnel is caused primarily by the combination of the relatively long grade that increases close to the tunnel, presence of trucks, surges in input volume, and the psychological effect of the tunnel. Other contributing factors include trucks in the median lane and prohibition of lane changing within the tunnel.

Investigation and Evaluation of Means for Improvement

Possible means for improving traffic operations on the study section that were investigated include the following:

1. Permitting passenger car lane changing and truck lane restrictions;
2. Control system plan;
3. Truck use restrictions;
4. Reversible lane operations;
5. Control system plan combined with truck use restrictions and reversible lane operations; and
6. Additional two-lane roadway.

In evaluating the effect of these improvements, the estimated peak hour traffic demand for the period 1961 to 1980 is based on present peak hour demand of 3,600 vehicles per hr with an assumed 3 percent annual increase. A summary of the effects of these improvements are shown in Figure 15 and are described in the following subsections.

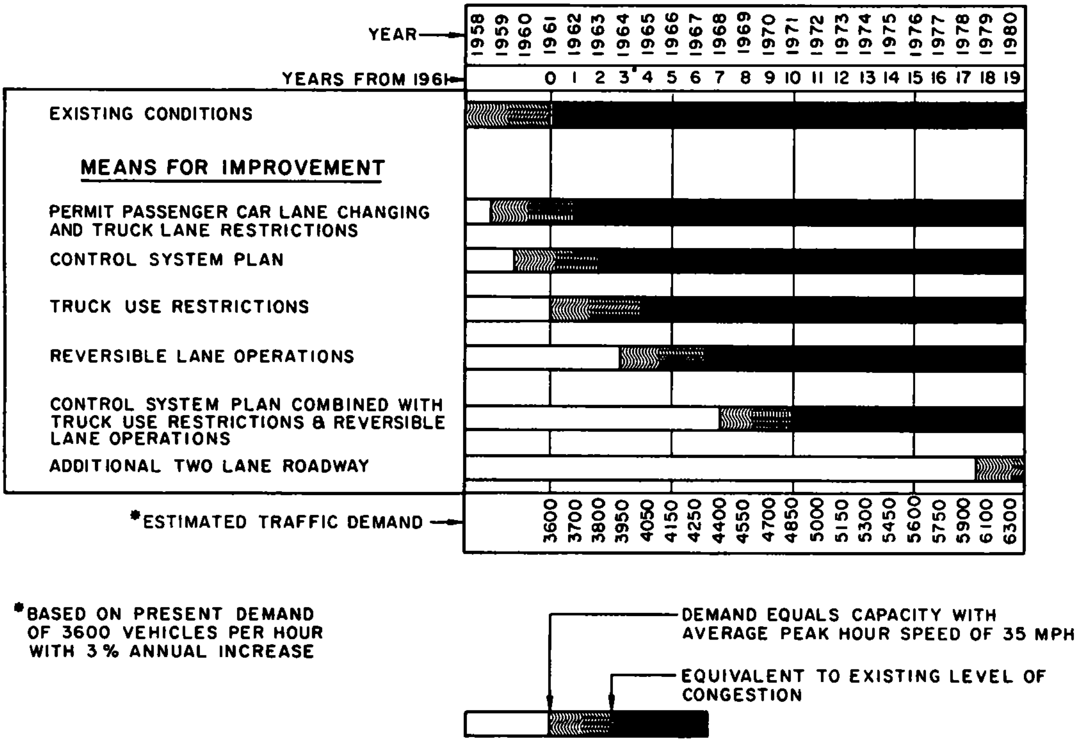


Figure 15. Estimated effect of improvements.

The transition between the white and gray portions of the bar graphs indicates the year when the peak hour traffic demand is expected to reach the practical capacity of the critical location and therefore would result in an average peak speed of about 35 mph. The transition between the gray and black portions of the bar graphs indicates the year when the present 1961 level of congestion would be expected to recur.

Permitting Passenger Car Lane Changing and Truck Lane Restrictions. —The prohibition of lane changing and the presence of trucks in the median lane slightly reduces the capacity at the critical section. It is estimated (see previous section, "Investigation of Time Headways") that permitting passenger cars to change lanes in the tunnel and the restriction of truck traffic to the shoulder lane will increase the tunnel capacity by 100 to 200 vehicles per hr. Figure 15 indicates that such action would postpone the recurrence of the existing 1961 level of congestion for 1 to 2 years. The immediate effect would be reducing the duration of congestion from 1 hr 40 min to 1 hr, and the congested portion from 2 mi to 1 mi.

Control System Plan. —Volume input rates fluctuate from 2,400 to 3,900 vehicles per hr, and occurrence of an extremely high volume rate prematurely causes a high density condition that results in reduced capacity. One means of preventing this from occurring would be to regulate the arrival rate to a predetermined maximum level, resulting in a smooth steady flow past the tunnel section and increased volume. Experience in the New York Holland Tunnel indicates that the regulation of input has resulted in an increased capacity of 6 percent. Figure 15 indicates that such an increase would postpone the recurrence of the existing 1961 level of congestion for two years. The immediate effect would be reducing the duration of excess demand from 1 hr 40 min to slightly less than 1 hr, and excess vehicles would be stored on ramps or diverted to other routes. Techniques for regulating input volume are discussed later.

Truck Use Restrictions. —The presence of trucks on the upgrade at the tunnel reduces the capacity by 380 vehicles per hr. Figure 15 shows that truck use restrictions would essentially eliminate congestion during 1961 and postpone the recurrence of the existing 1961 level of congestion for four years.

Reversible Lane Operations. —Reversing the flow on the eastbound median lane between Edgewood Avenue and the Greenfield Bridge would increase the westbound capacity by 900 vehicles per hr. Figure 15 shows that such action would result in congestion-free operations for three years, postpone the recurrence of the existing 1961 level of congestion for seven years.

However, restricting the present eastbound traffic flow to a single lane would result in 1-hr duration of congestion and an economic time loss of approximately one-third of the present westbound amount. One means of temporarily eliminating this congestion would be to close the Beechwood Boulevard on-ramp during the morning peak period. (This ramp is now closed during the afternoon peak traffic flow because of its design and proximity to the Beechwood Boulevard off-ramp.) With the reversible lane operations and with the Beechwood Boulevard on-ramp closed, no appreciable eastbound congestion would occur for three years.

Control System Plan Combined with Truck Use Restrictions and Reversible Lane Operations. —The maximum capacity obtainable without constructing additional roadway would be to install an over-all traffic control system that would incorporate truck use restrictions and reversible lane operations. Figure 15 shows that such action would result in congestion-free operations for seven years and postpone the recurrence of the existing 1961 level of congestion for ten years.

A schematic diagram of such a parkway control system is presented in Figure 16. The control system would consist of (a) measurement, (b) decision, and (c) control. Continuous measurements of volume, speed, and density would be obtained from sensors located at the tunnel, and in advance of the Ardmore Boulevard and Braddock Avenue on-ramps. These measurements would be sent to a central computer location for computation, data collection, and control decision. If operations were satisfactory, advance speed and lane use information would be conveyed to the drivers. If operations were unsatisfactory, control decisions could be made to restrict trucks, meter or close ramps, reverse lanes, alert responsible agencies, and convey speed and lane use information to the drivers. The recording of operations and controls would permit the

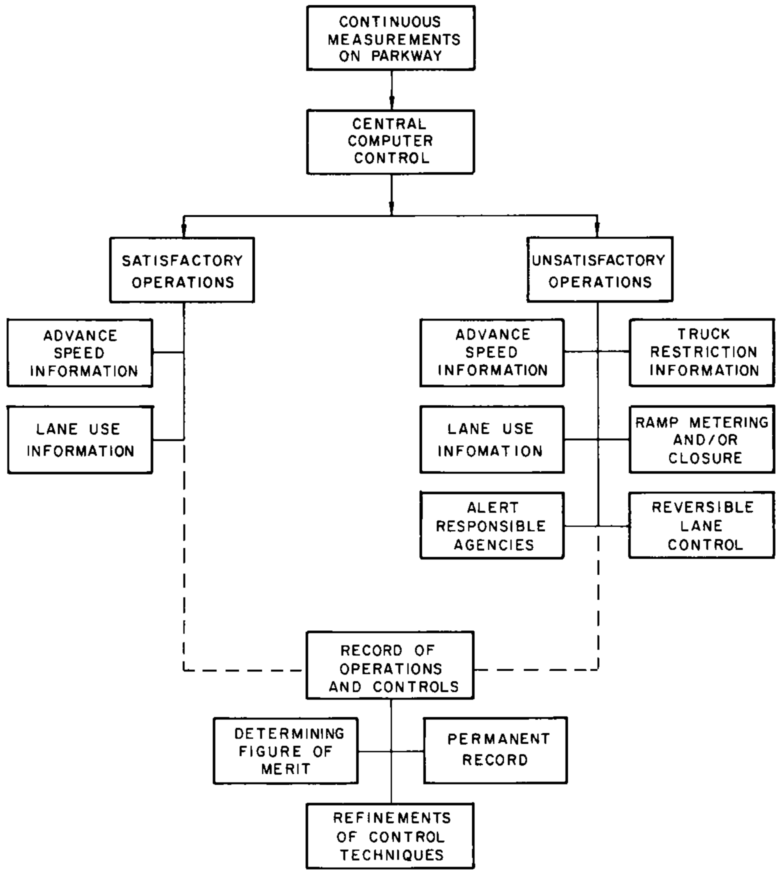


Figure 16. Schematic diagram of parkway control system.

refinement of control techniques, determine the figure of merit, and provide the permanent record of quantity and quality of flow.

Additional Two-Lane Roadway. — The construction of an additional two-lane roadway would increase practical capacity by about 3,000 vehicles per hr. Figure 15 indicates that such action would result in congestion-free operations until 1978. This additional two-lane roadway would not only provide the needed capacity for future westbound traffic, but by reversing the operations during the afternoon peak period, this same roadway could provide the needed capacity for future eastbound traffic.

ACTION TAKEN BY PENNSYLVANIA DEPARTMENT OF HIGHWAYS

The various solutions offered by this study to the immediate problem of congestion at the Squirrel Hill Tunnel were considered by the Pennsylvania Department of Highways at an early October meeting and decisions made based on the forecast of relief to be expected, cost of the time necessary for execution, and political and other considerations. The short-term plan chosen was the lane reversal operation to provide three inbound lanes through the Squirrel Hill Tunnels during the morning peak hours. The truck ban, although easiest and least expensive to put into effect, was rejected due to the lack of suitable routes through the City of Pittsburgh. Longer-range projects include the traffic metering proposal which is now under study by Thompson Ramo Woolridge and the ultimate provision of added capacity through roadway or tunnel construction.

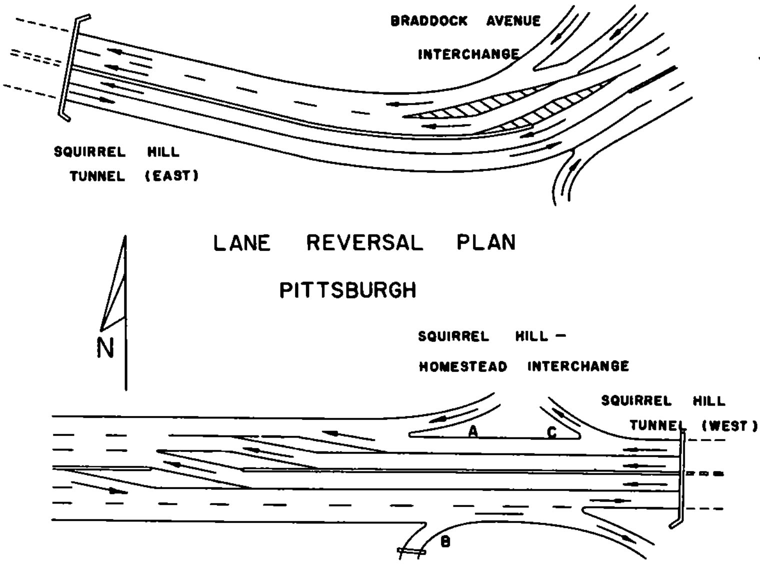


Figure 17. Lane reversal plan used.

Lane Reversal Plan

Due to the impending cold weather and constant public pressure, immediate relief through the lane reversal plan was scheduled for initiation on November 1, 1961, before the first snowfall. Despite some rather caustic comments earlier, the newspapers gave good advance publicity to the project, including some sketches showing what was planned. Guide and control signs were installed where possible and additional portable signs located. Rubber traffic cone placement was planned by using paint spots on the pavement—over 320 cones were to be used. Two 300-ft sections of median were removed and crossovers constructed by Department forces. Figure 17 shows the general area involved and the points chosen for the median breaks—the first at the Braddock Avenue Interchange where between 700 and 800 vehicles per hr entered the Parkway, and the other, west of the Tunnel where the Parkway widened to three lanes westbound. With this heavy an entry volume and to reduce delay and confusion, it was decided that no choice of path could be given to any motorist once he had entered the reversal area. The shaded area on the sketch shows the first phase of cone placement, clearing the lane to be reversed and providing for dispersal at the west end. Placement sequence is carefully planned to reduce hazard to workmen and delay to the public. This operation is usually completed shortly before 6:00 a.m. and takes the two crews about 45 min. Cone placement (cross-hatched area) on the east end at approximately 7:00 a.m. started the three lane operation somewhat in advance of the peak arrival time. (This requires approximately 8 min more for the east crew of five men). Ramp B in the Squirrel Hill-Homestead Interchange contributed about 300 vehicles per hr to the eastbound flow at a critical point very close to where this traffic is reduced to a single lane. To eliminate this confusion and the resultant loss of capacity as well as to lower the demand on the Tunnel, this ramp was closed from 6:00 to 9:00 a.m. (It has been closed during the afternoon rush period for several years.) The lane reversal is discontinued at 8:45 now and within 20 min, two lanes are available to eastbound movement with the westbound flow back in its usual path. Ramp B is opened as soon as these two lanes are again available. The final clean-up of the dispersal pattern west of the tunnel takes a little longer, but all is back to normal by 9:30 a.m.

Operational Experience

The first days of the experiment were free from rain or snow, and almost complete success was realized in eliminating delay to inbound traffic. Outbound flow, reduced to one lane, experienced some delay as expected, but as time passed, some vehicles used other routes and the "novelty effect" wore off resulting in very little congestion on most days. Other problems included the following:

1. Ramp B closing time. A study of the variation in traffic flow during the period before maximum movement eastbound showed that Ramp B could remain open until 6:30 a.m. and could reopen at 9:00 a.m. without causing congestion. This accommodated a number of rather vocal area residents and aided the public relations on the whole project.
2. Congestion on Ramp A. With the dispersal pattern at the west end, approximately 1,200 vehicles per hr from the Squirrel Hill area were forced to merge into the shoulder lane of the Parkway where almost 1,000 vehicles were traveling. Once again, voluntary rerouting of ramp traffic as well as decreased use of the shoulder lane has almost eliminated this delay.
3. Tunnel ventilation. The design of the tunnel ventilation made use of the motion of vehicles in the tubes to assist in air movement. The introduction of two-way traffic in the south tube eliminated this aid and the carbon monoxide content of the air rose almost to the critical danger level. The same problem had been faced in the other tube during periods of congestion. It was solved by operating all fans at maximum speed for about 30 min before the traffic flow was reversed, which greatly reduced the residual carbon monoxide level and prevented the dangerous accumulation during the 2-hr period.
4. Speed limits were increased somewhat as drivers became experienced with the new operation. Trucks were also restricted to shoulder lanes only on both approaches to the tunnel, reducing long gaps and loss of capacity.

Throughout this period, the local newspapers gave the operation good coverage and generally favorable comment. The only complaint received concerning the inbound improvement came from a man whose employer used the Parkway. He had been accustomed to arriving a few minutes late each day, knowing his superior would be later, but no more; the operation had spoiled his fun.

When the new pattern of traffic flow had settled down, estimates showed that, with essentially no delay through the tunnel, a savings of up to 10 min was being given to the westbound movement, with a benefit in time and gasoline saving to motorists which showed a cost-benefit ratio of approximately 3 to 1. During the first several weeks, traffic was attracted to the free-moving Parkway and volumes increased during the 2-hr peak period by approximately 6.5 percent. The pattern of inbound motorists was also changed in that the hour from 7:00 to 8:00 a.m. showed an increase of 15.0 percent in volume, and the 8:00 to 9:00 a.m. period actually decreased by 2.0 percent (no appreciable change from 6:00 to 7:00 a.m.). The new pattern of movement handled the additional load without measureable delay. Volume comparisons show a generally higher use of the Penn-Lincoln Parkway during 1961-62 than during similar periods over the past few winters.

Counts show that, from 7:00 to 8:00 a.m., the reversible lane was carrying about 1,300 vehicles per hr and the median lane through the north tube carried about 1,200 per hr. The shoulder lane on the north, which still contains the heavy trucks, remained lowest with about 900 vehicles per hr (a total of 3,400 vehicles per hr). The single eastbound lane has carried a maximum volume of 1,613 during the same hour, but usually averages about 1,500 vehicles, a decrease of 500 over the former two-lane volume. During the summer of 1962, it is expected that the traffic demand will at least reach the 3,700 vehicles per hr assumed by the consultant and that this flow will be handled without appreciable delay by the lane reversal plan.

To date, no serious accidents have occurred during the lane reversal period, although this is the first time a tunnel within the metropolitan area has been used for two-way traffic. Minor accidents and vehicle break-downs have been handled by the standard emergency procedures. It is expected that accidents will be reduced on the approach to the tunnel as the start-stop type of operation is no longer required. During the winter months, the liberal use of salt as well as the usual plowing activities has aided in safe and convenient use of the Parkway in the unusual manner.

Future Plans

Now that all concerned are sure of the workability of the lane reversal plan, there remains the problem of a permanent method of operation to replace the large labor force employed. Lane control signals, designed as a part of a traffic metering system, may replace some of the cones now used but the terminal treatment is still under study, as a physical barrier is considered mandatory for a positive diversion of a lane of traffic. Experience gained in other parts of the country will be of great value in solving this control problem.

REFERENCES

1. Forbes, T.W., "Speed Headway and Volume Relationships on a Freeway." ITE Proc., pp. 103-125 (1951).
2. Olcott, E.S., "The Influence of Vehicular Speed and Spacing on Tunnel Capacity." Jour. Operations Res. Soc. of America, 3: 147-167 (May 1955).
3. Ricker, E.R., "Automation in Traffic Control." ITE Proc., pp. 80-84 (1956).
4. Webb, G., and Moskowitz, K., "California Freeway Capacity Study—1956." HRB Proc., 36: 587-641 (1957).
5. Edie, L., and Foote, R., "Traffic Flow in Tunnels." HRB Proc., 37: 334-344 (1958).
6. Keese, C.J., Pinnell, C., and McCasland, W.R., "A Study on Freeway Traffic Operations." HRB Bull. 235, 77-132 (1959).
7. Ricker, E.R., "Monitoring Traffic Speed and Volume." Traffic Quarterly, 13: 128-140 (Jan. 1959).
8. Berarducci, A., and Dobelek, J., "Freeway Entrance and Exit Controls." Traffic Quarterly, 8: 20-33 (Jan. 1959).
9. May, A.D., J., and Wagner, F.A., "Some Interrelationships on the Fundamental Characteristics of Traffic Flow." HRB Proc. 39: 524-547 (1960).
10. Covault, D.O., "Time-Lapse Movie Photography." Traffic Engineering, pp. 11-14, 62 (March 1960).
11. "Squirrel Hill Tunnel Delay Study." Pennsylvania Dept. of Highways and City of Pittsburgh, unpublished report (June 1960).
12. Pearson, P.H., and Ferreri, M.G., "Operational Study—Schuylkill Expressway." HRB Bull. 291, 104-123 (1961).
13. May, D., Jr., "Traffic Characteristics and Phenomena on High Density Controlled Access Facilities." Traffic Engineering 31: 11-19, 56 (March 1961).

Operational Study of Signalized Diamond Interchanges

CHARLES PINNELL and DONALD G. CAPELLE, Texas Transportation Institute, Texas A & M College

This paper was developed from a research project on diamond interchanges conducted by the Texas Transportation Institute in cooperation with the Texas Highway Department. The project had the general objective of developing criteria useful in the capacity evaluation, design, and signalization of diamond interchanges. An earlier report, presented at the 40th Annual Meeting, included a procedure for evaluating the capacity of signalized diamond interchanges. This paper presents data from operational studies designed to develop criteria for the design and signalization of diamond interchanges.

Four separate field studies were conducted at a signalized diamond interchange on Interstate 35W in Fort Worth, Tex. By modifications to the control equipment at this interchange, three separate signal phasing arrangements were evaluated. Also, operations with both fixed-time and traffic-actuated control equipment were studied. Recommendations are made regarding both the signal phasing and the equipment for use at signalized diamond interchanges.

The operational studies also permitted the evaluation of interchange design elements that affected traffic operations at the facility. Various aspects of design that affect operation are discussed and illustrated by a recommended design.

Finally, examples illustrating the capacity-analysis, design, and signalization of diamond interchanges are presented. These examples demonstrate the application of the research findings to specific design and operation problems.

• THIS REPORT presents results from research studies on diamond interchanges conducted by the Texas Transportation Institute in cooperation with the Texas Highway Department. These studies were conducted in connection with Research Project RP-16, "Ramps and Interchanges," and had the objective of investigating the capacity, design, and operation of conventional-type diamond interchanges.

The initial studies were designed to evaluate only the capacity of a conventional-type diamond interchange. The procedure and results of these studies were reported in a paper by the authors (1) which was distributed to the various districts and offices of the Texas Highway Department.

In addition to the capacity studies, there existed a need for additional research to develop data that would serve as criteria in the design and signalization of diamond interchanges. The results of this research are presented as a major portion of this report.

INTERCHANGE TYPE

There are many design variations of the diamond interchange, and numerous conditions may exist affecting its operation. The research studies conducted in connection with this project involved only the conventional-type diamond interchange, as shown in

Figures 1 and 2. The sites selected for study were free of any special conditions such as signalized intersections in the near vicinity of the interchange. The data and recommendations presented are directly applicable only to the type interchange illustrated and similar diamond interchanges without frontage roads. However, they should serve as a guide to engineering judgment in treating special designs and conditions.

Figure 1.

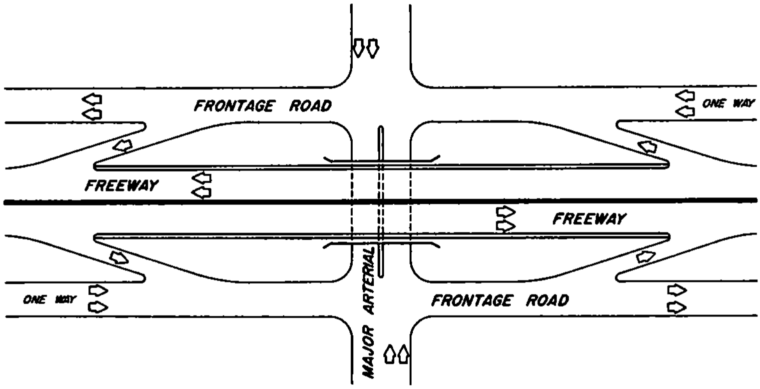


Figure 2. Diamond interchange, conventional arrangement.

SIGNAL OPERATIONS

Requirements

A complex signal system is required to control traffic at a diamond interchange due to the proximity of the two signalized intersections and the variations of traffic maneuvers encountered in interchanging traffic. The signalization should perform the following two basic functions:

1. Separate all high-volume conflicting movements in the interchange area.
2. Minimize storing of vehicles between the two intersections.

In considering the signal system to be used at a diamond interchange, it is natural to assume that the type of signalization would depend on the traffic volumes and movements experienced. Because many of the future diamond interchanges will be a part of new freeways extending into relatively undeveloped areas, it is conceivable that numerous volume conditions will be experienced during the design life of an interchange. Thus, it appears that numerous phasing arrangements would be necessary to accommodate the various traffic conditions that are likely to be encountered. However, this is not entirely true due to the peculiarities of diamond interchange operation.

Inasmuch as the two signalized intersections of a conventional-type diamond interchange are usually spaced approximately 220 to 290 ft apart (center to center), two factors that influence the signal phasing are (a) amount of vehicle storage between the two intersections and (b) volume of left turns from interior approaches. Extreme care must be exercised to assure that the storage limit of the interior approach is not exceeded. In addition, high-volume left turn movements can quickly exceed the storage capacity on interior approaches because only one lane is generally available for the storage of left turning vehicles.

Signal Phasing

Figure 3 shows two possible phasing arrangements for diamond interchange signalization. Although these phasing arrangements represent only two of numerous possible sequences, they illustrate the problems incurred in developing signal phasings for diamond interchanges.

In Sequence I (Fig. 3) clearance phases must be added following phases A and B to clear the interior approaches for storage of the frontage road movement on phase C. This in effect creates a four-phase cycle (if the two clearance intervals are considered approximately equal to one phase) with a considerable waste of time. A second disadvantage of Sequence I is the sluggish operation frequently encountered on the phase A movement. This results from numerous cars being stored on the interior approaches during phase C. When the phase A movement is initiated, the traffic on the major street approach is delayed until the interior approach traffic can move out. Thus, in effect, a double starting delay is imposed.

The third and most serious disadvantage of Sequence I is the capacity limitations placed on the frontage road movements. The amount of traffic that can be moved from a frontage road approach during phase C is governed by the storage capacity of the interior approaches. Therefore, this sequence is inadequate to accommodate large frontage road movements.

Sequence II allows the interior approaches to clear on phase B and gives preference to moving major street traffic. However, a serious left-turn storage problem is often created by this sequence. Left turns from both of the major street approaches are stored during phase A, and it must be considered that an average diamond interchange can store only a maximum of 7 left-turning vehicles per lane on an interior approach. When a heavy left-turn movement from a major street approach occurs (which frequently happens), the storage capacity for left-turning vehicles is exceeded and blocking of the intersections results. Sluggish operation will follow phase C, and storage capacity limitations will exist on the ramp and/or frontage road movements as in Sequence I.

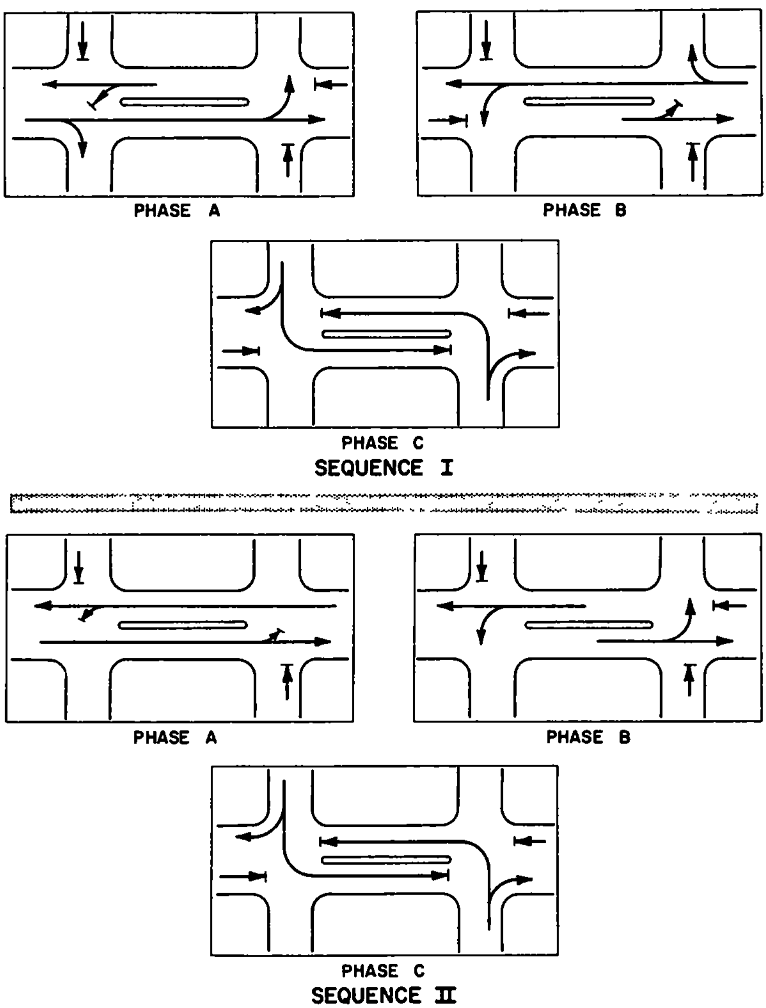


Figure 3. Signal phasings.

Recommended Phasing

Other three-phase arrangements such as Sequences I and II yield the same basic problems of inadequate storage and sluggish, inefficient operation. Consideration of these facts and the requirements of diamond interchange signalization led to the conclusion that the four-phase sequence shown in Figure 4 would serve best for all traffic conditions. This sequence has been utilized at diamond interchanges by the city of Houston, the California Highway Department, and perhaps other agencies, but the advantages and efficiency of this phasing have not been fully realized.

The two most serious problems encountered with a three-phase system (as previously discussed) are its inability to accommodate large frontage road movements and left turns from interior approaches. These problems are eliminated with the recommended four-phase system (Fig. 4).

Each of the four approach movements is given a separate phase and is permitted to move through the entire system upon receiving a green indication. This eliminates the storage capacity limitations that develop on the interior approaches with other phasing arrangements. Consideration of each movement in the four-phase sequence will show that storing of vehicles on the interior approaches is practically eliminated.

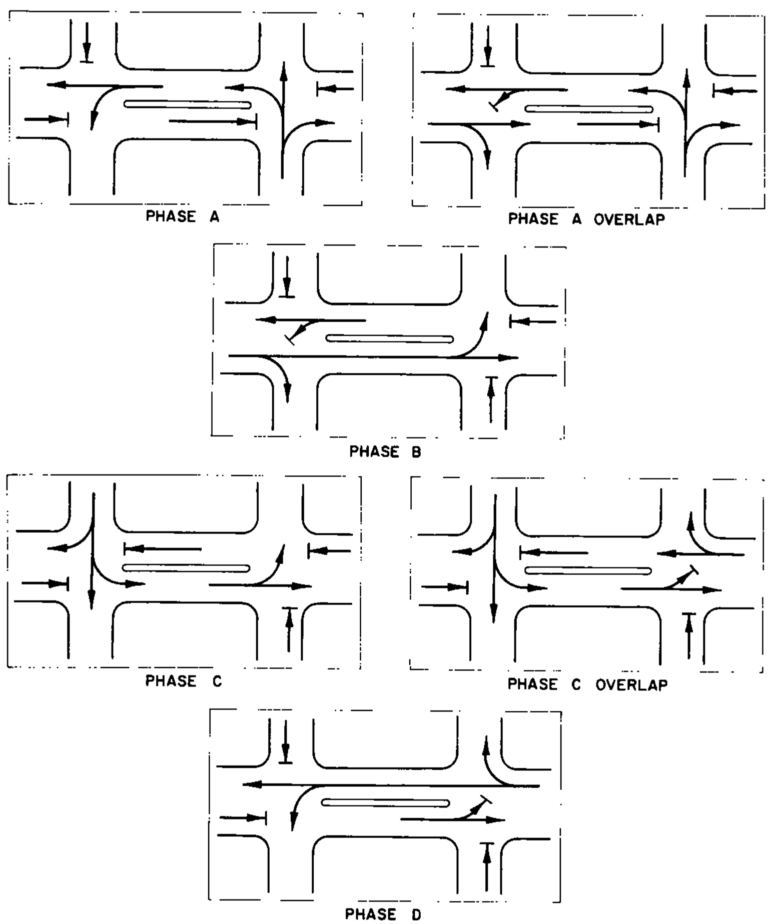


Figure 4. Recommended signal phasing for conventional-type diamond interchange.

The only vehicles requiring storage are those making a U-turn movement from a frontage road during the last 6 to 8 sec of a frontage road phase. This seldom stores more than 2 vehicles per cycle and has very little detrimental effect on operation. Thus, the left turn storage problem is eliminated with this phasing.

An additional advantage of the recommended phasing is the efficiency that can be obtained. An overlap of the frontage road and major street phases (phases A and C overlap) is possible due to the starting delay and travel time incurred by the major street traffic in moving from one intersection to the other. This overlap utilizes the green time per cycle better and permits the movement of large volumes through the interchange with average cycle lengths in the range of 60 to 80 sec.

Therefore, it was concluded that the recommended four-phase sequence is the best signal phasing for a conventional-type diamond interchange.

SELECTION OF CONTROL EQUIPMENT

A significant problem encountered with diamond interchange operation is the selection of the type of signal equipment (fixed time or vehicle actuated) that will yield the best results in controlling the interchanging traffic. One of the objectives of this project was to study this problem.

There are four significant factors that influence the selection of equipment:

1. Volume fluctuations;
2. Equipment flexibility;
3. Coordination; and
4. Economics.

Volume Fluctuations

Volume fluctuations during off-peak and peak periods of operations are common knowledge to traffic engineers and should be given consideration in any well-designed signal system. Studies of vehicle arrivals by 5-min intervals at several diamond interchanges revealed significant volume variations within the peak hour. A plot of 5-min demand volumes on the four approaches of the Cullen interchange on the Gulf Freeway in Houston, Tex. is shown in Figure 5. This plot shows that each of the approaches had a different peaking pattern and a wide fluctuation of 5-min demand volumes.

A superimposition of these volume plots will show a comparison of the short periods of peak flow. Little overlapping of the peak periods is evidenced for the four approaches and thus a single timing plan for the peak hour would be inefficient.

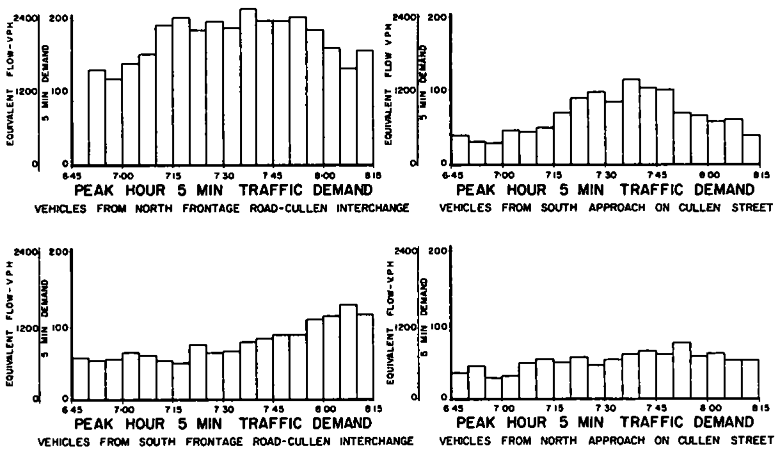


Figure 5. Demand volumes, Cullen Boulevard interchange, Houston, Tex.

This volume fluctuation emphasizes the need for equipment that can adjust cycle and phase lengths to traffic demand during both off-peak and peak periods of traffic flow, if maximum operational efficiency is to be obtained.

Equipment Flexibility

An important feature of signal flexibility is the ability to accommodate special conditions that may develop during peak periods of operation. Conditions such as stalled vehicles, minor accidents, or other disruptions of normal traffic flow can result in tremendous backlogs of traffic on the interchange approaches. These conditions commonly occur during peak periods and were observed frequently during the signalization studies. If the signal system does not have the flexibility to temporarily increase the cycle lengths to accommodate the accumulated demand, the interchange will be congested until the traffic demand diminishes.

Coordination

A third factor is that of progressive movement for the through traffic on the major street. Progression of this through traffic is desirable. However, because the signal system at the interchange must operate on a multiphase sequence, the interchange area represents a bad timing point in the coordinated system.

It must also be considered that the through traffic for which progression is desired represents a minor percentage of the total traffic entering the interchange area. Analysis of volume counts at the Berry Street interchange in Fort Worth and the Cullen and Wayside interchanges in Houston indicated that the volume of the through movements represented only approximately 25 percent of the total interchanging traffic. Therefore, efficient operation of the entire interchange system should receive more priority than that of providing progression for the through traffic on the major street.

The interchange can be designed so that traffic will not be delayed for more than one cycle, and some progression for the major street traffic can be obtained by timing away from the interchange as shown in Figure 6.

Economics

The relative cost of fixed-time and traffic-actuated equipment for complete signalization should also be considered. Specific equipment costs are not compared in this report because these will vary with time, type of equipment, and manufacturer. However, consideration of the requirements of one system over the other and the relative costs can be made.

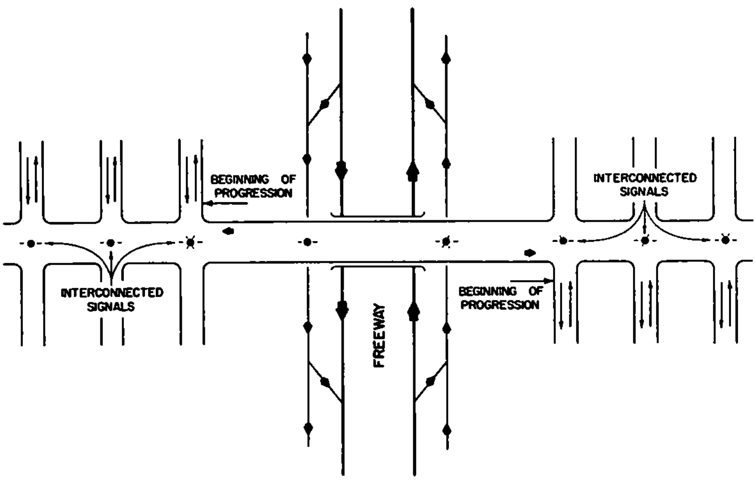


Figure 6. Progressive timing of signal systems adjacent to a diamond interchange.

Basic equipment, such as signal heads, mast arms, and much of the wiring, would be the same for both systems. The basic difference in cost for the two systems is related to the following factors:

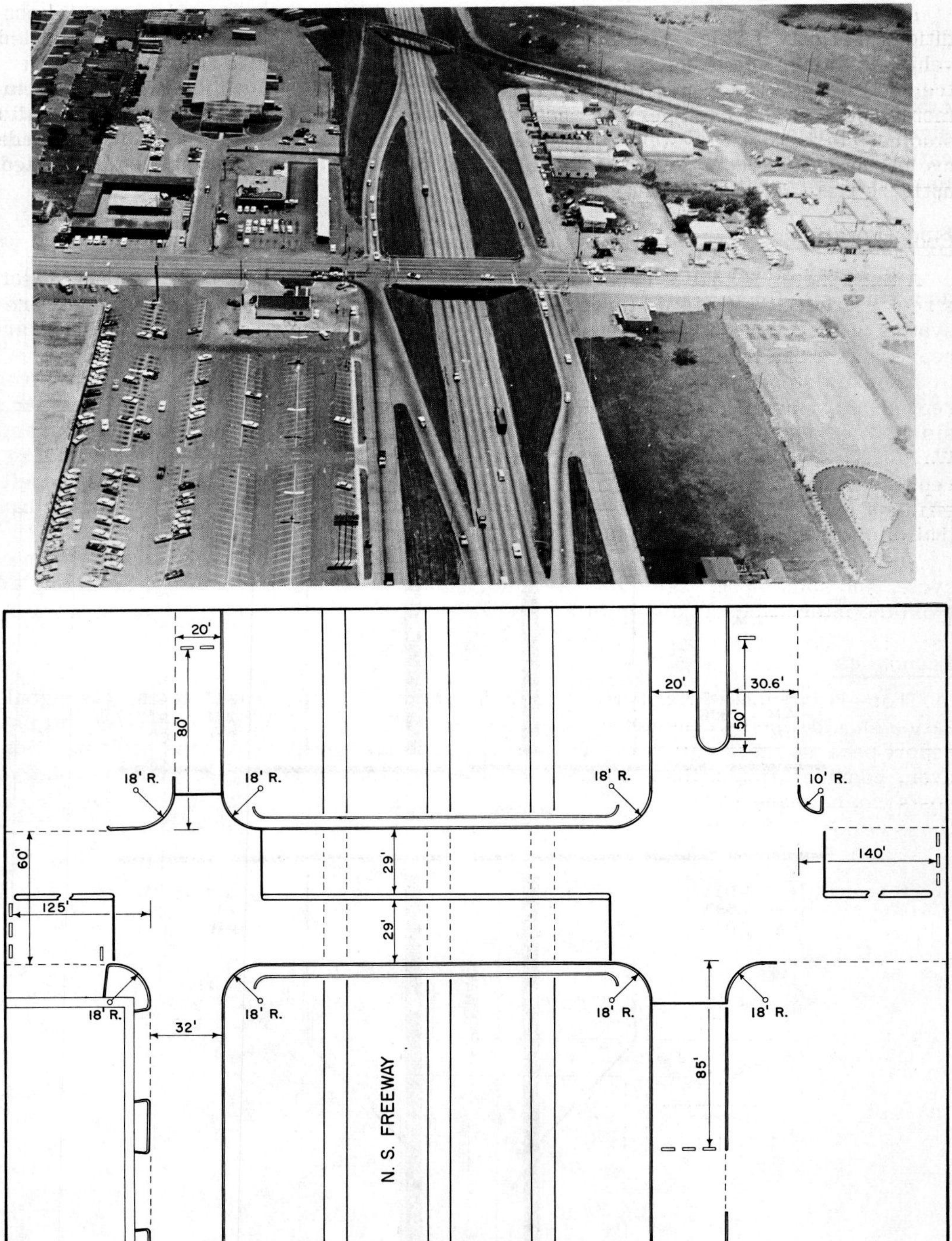


Figure 7. Berry Street interchange, Fort Worth, Tex.

1. Additional cost of actuated control units; and
2. Additional cost of the detectors (initial cost and installation) required for the actuated system.

The difference in cost for the two systems (fixed time vs traffic actuated) is a relatively small percentage of the total installation cost and loses significance when prorated over the design life of the facility.

OPERATIONAL STUDIES

The objective of the operational studies at an existing diamond interchange was to evaluate several signal phasing arrangements and to study the adaptability of actuated signal equipment to diamond interchange operation. Previous work had given an indication of the type of operation to be expected, but no field studies had been conducted to verify the operation anticipated.

Study Location

The Berry Street Interchange (Fig. 7) on IH 35W in Fort Worth, Tex., was selected as a study site for the operational studies. The geometrics of this interchange were

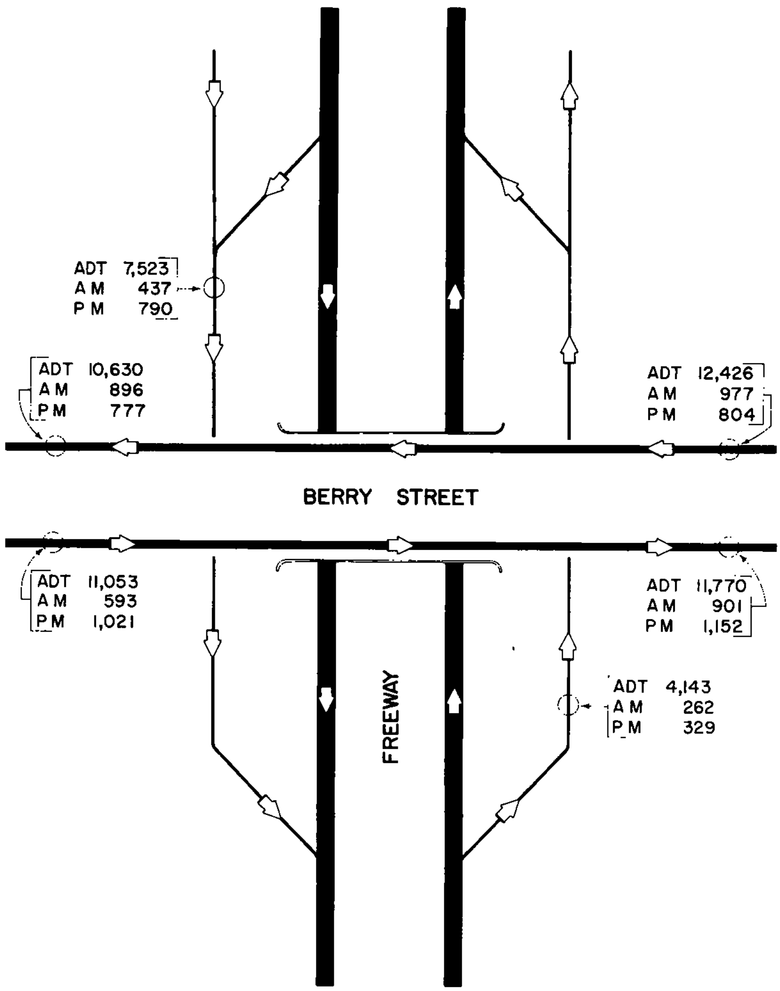


Figure 8. Traffic volumes, Berry Street interchange.

essentially those of a conventional-type diamond and are representative of numerous diamond interchanges that have been constructed. Traffic volumes (Fig. 8) at this interchange were of sufficient magnitude to provide adequate study conditions. The traffic control equipment at the interchange at the beginning of the studies consisted of a three-phase volume-density controller with dual-clearance timers.

The Traffic Engineering Department of Fort Worth agreed to modifications of the signal phasing and control equipment; consequently, the site provided an excellent study location for evaluating actuated control equipment under various phasing arrangements.

Study Procedure

Four separate studies of traffic operations were conducted at the Berry Street interchange. All of these studies were conducted during the late afternoon peak period of flow (4:00 to 5:30 p.m.) on either a Wednesday or Thursday. Data were recorded on the following:

1. Traffic volumes per cycle from each approach;
2. Cycle lengths; and
3. Phase lengths.

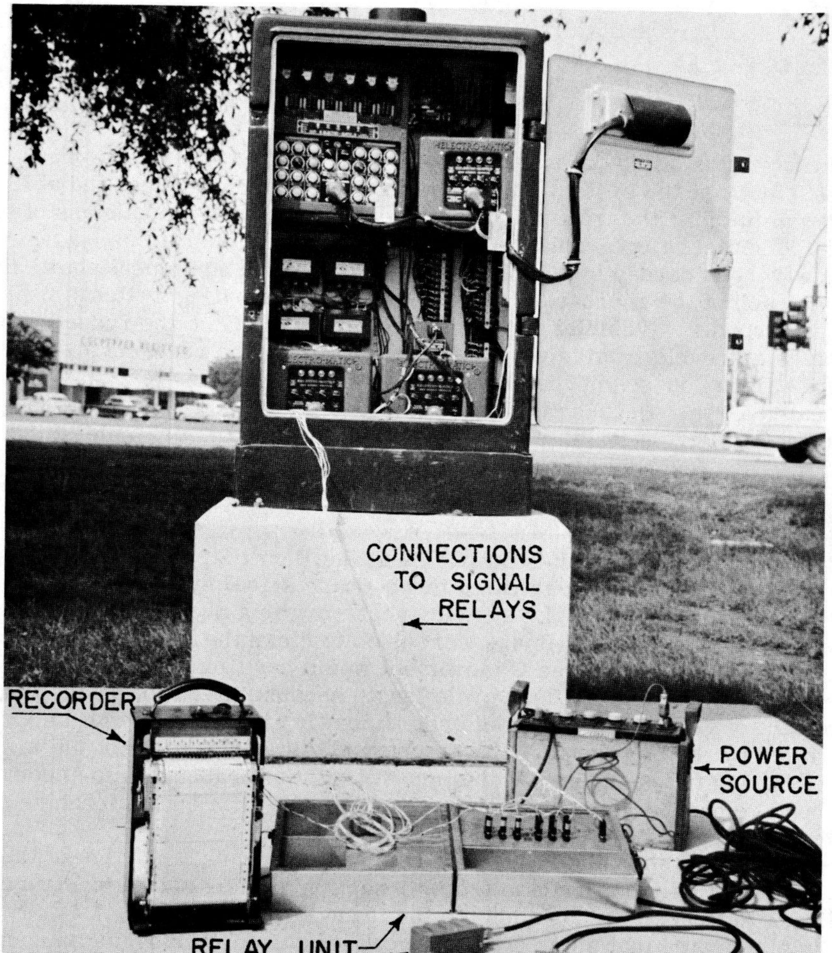


Figure 9. Recording equipment.

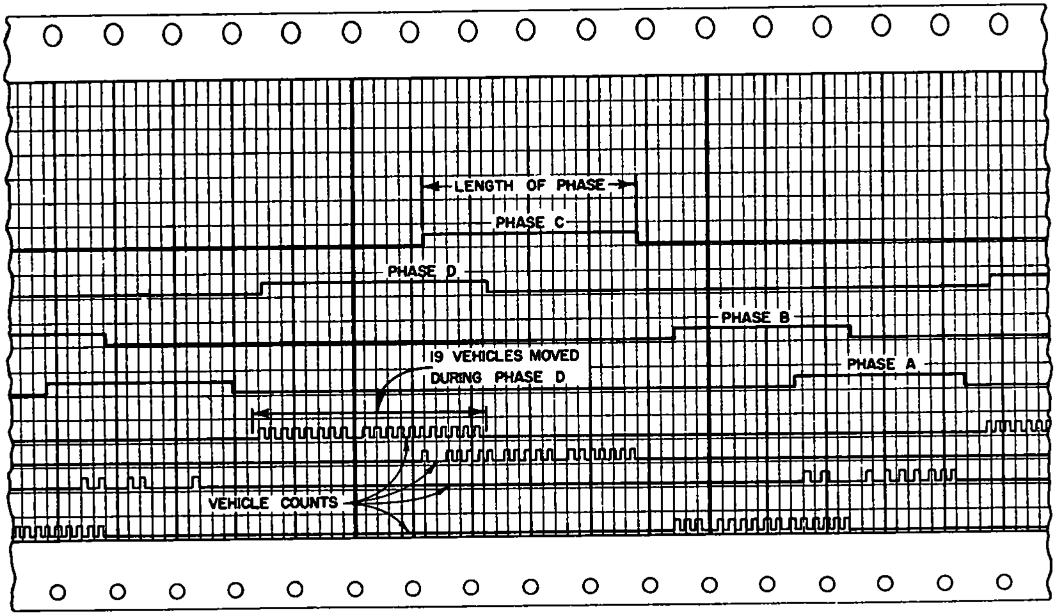


Figure 10. Sample data chart.

In addition, interchange operations were observed and notes were made on the general efficiency of the system. Queue lengths on the approaches, the ability of the signal system to clear the traffic demand on each cycle, and smoothness of operation were used as criteria for evaluating the efficiency of the operations.

The data were recorded by the multi-pen recorder and equipment shown in Figure 9. Volumes on each of the approaches were recorded by actuating switches connected to pens on the recorder. Vehicles were indicated by a "blip" on the recording tape. The phase lengths and cycle lengths were obtained by wiring the recorder to the relays controlling the various phases of the signal cycle. The energizing of a signal on a particular phase actuated a pen. Because the chart moved at a constant speed, the length of each phase and the total cycle length could be readily measured. A sample of the recording chart and the data recorded is shown in Figure 10.

Berry I: Three-Phase Operation

The first of a series of studies conducted at the Berry Street interchange consisted of an evaluation of the existing actuated three-phase signal system which had been in operation for several years. The system operated with a phasing arrangement as shown in Figure 11. Dual clearance timers were used to clear the interior approaches after phases A and B so that the phase C movement would have adequate storage. These clearance intervals contributed to a long cycle length because they required a total of 16 sec.

The amount of green time that could be efficiently allotted to the phase C movement was controlled by the storage capacity (approximately 14 vehicles) of the interior approaches. During the peak period, this green time was inadequate to accommodate the frontage road demand. This resulted in a backlog of traffic on the frontage road and contributed to sluggish operation on the A and B phases.

Figures 12 and 13 show the congestion experienced during a portion of the field study. The congestion not only occurred in the intersection area as shown in Figure 12 but extended back to the exit ramps and onto the freeway (Fig. 13).

The study indicated that some of the approaches were experiencing more demand than could be accommodated. A comparison of 5-min demand volumes on the west frontage road with the number of vehicles cleared (Fig. 14) indicated that the number of vehicles forced to wait at the end of each cycle was increased by small increments until a large backlog existed.

A tabulation of cycle lengths during the peak period (Table 1) showed that the cycle lengths varied from 89 to 159 sec., with an average of 118 sec. This long cycle length imposed an excessive delay with a majority of the traffic having to wait for more than one cycle in order to clear through the interchange.

TABLE 1

BERRY I DATA

CYCLE NO.	"A" "B" PHASE LENGTH	MOVEMENT "A"	MOVEMENT "B"	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	CYCLE LENGTH	TOTAL GREEN	TOTAL VEHICLES
1	25	21	8	29	23	37	40	116	91	92
2	25	25	12	27	20	32	32	109	84	89
3	28	19	8	22	12	31	32	106	81	71
4	24	16	9	27	21	41	45	117	92	91
5	25	22	12	35	36	32	37	117	92	107
6	23	24	17	49	40	34	34	131	106	115
7	29	25	12	46	38	27	24	127	102	99
8	27	15	12	38	35	26	21	116	91	83
9	25	12	11	13	13	26	26	89	64	62
10	25	14	10	21	20	22	22	93	68	66
11	27	18	12	31	22	26	24	109	84	76
12	26	21	8	29	24	30	26	110	85	79
13	26	22	14	33	24	28	27	112	87	87
14	26	24	3	19	15	24	19	94	69	61
15	25	20	10	25	17	34	32	109	84	79
16	26	24	6	31	27	27	21	109	84	78
17	27	15	3	24	28	29	23	105	80	69
18	20	21	11	31	18	29	23	105	80	73
19	26	23	14	17	14	39	34	107	82	85
20	27	23	11	31	37	27	23	110	85	94
21	28	26	15	40	36	31	30	124	99	107
22	29	22	11	27	22	33	38	114	89	93
23	25	35	8	55	42	35	41	141	115	126
24	29	26	11	21	7	29	26	104	79	70
25	32	31	10	54	51	35	27	146	121	119
26	29	28	15	47	43	32	29	133	108	115
27	32	24	18	50	44	25	22	132	107	108
28	29	16	14	33	29	18	18	105	80	77
29	25	23	12	31	25	31	22	112	87	82
30	23	31	11	47	40	27	22	122	97	104
31	27	32	7	31	19	36	27	119	94	85
32	30	30	8	44	36	37	32	136	111	106
33	30	33	11	64	50	40	44	159	134	138
34	30	26	12	21	18	28	24	104	79	80
35	29	26	14	40	39	50	47	144	119	126
36	26	35	8	36	38	32	36	119	94	117
37	31	27	12	42	35	23	21	121	96	95
38	27	28	15	51	44	26	22	129	104	109
39	28	31	10	35	31	35	30	123	98	102
40	31	26	12	60	53	42	39	158	133	130
41	29	29	10	38	39	34	33	126	101	111
42	31	18	12	38	42	26	21	120	95	93
43	26	26	8	43	37	33	33	127	102	104
44	29	28	7	30	31	29	32	113	88	98
45	25	28	5	24	18	31	23	105	80	74
46	29	19	5	17	14	23	22	94	69	60
47	28	17	9	39	23	30	23	122	97	72
48	30	30	12	40	38	27	20	122	97	100
49	27	27	14	39	35	21	24	112	87	100
50	28	29	13	15	11	23	23	91	66	76

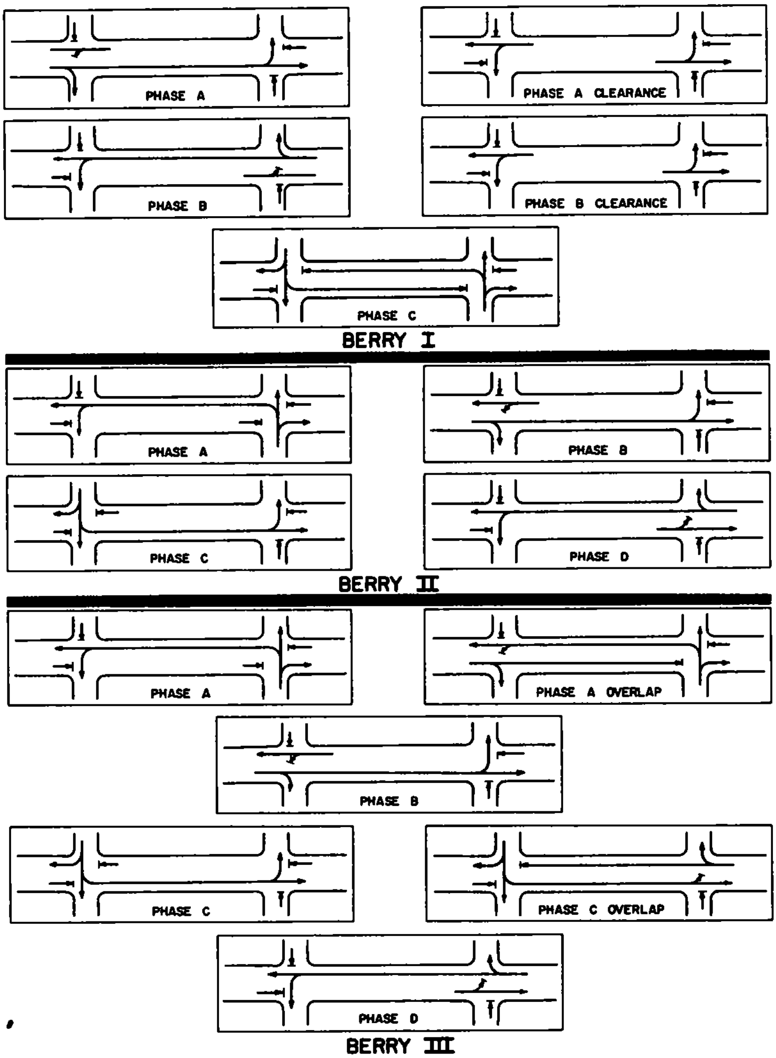


Figure 11. Phasing sequences.

Berry II: Four-Phase Operation, No Overlaps

After completion of the Berry I study, a different phasing and control system was installed at the Berry Street interchange. The three-phase controller was replaced by a two-phase volume-density controller with two minor movement controllers. The minor movement controllers were used to control the frontage road traffic, and the signal system operated with the four-phase sequence shown in Figure 11.

This phasing arrangement eliminated the storage problem and congestion that occurred with the existing three-phase system (Berry I). With this sequence, traffic on each approach was permitted to clear through the entire interchange upon receiving a green indication, and the green time was also allotted to each approach on the basis of traffic demand.

The results of the study indicated that the four-phase system was capable of handling the traffic demand at the interchange with a moderate degree of efficiency. Two approaches had backlogs of vehicles for several cycles during the study, but no serious congestion was experienced.

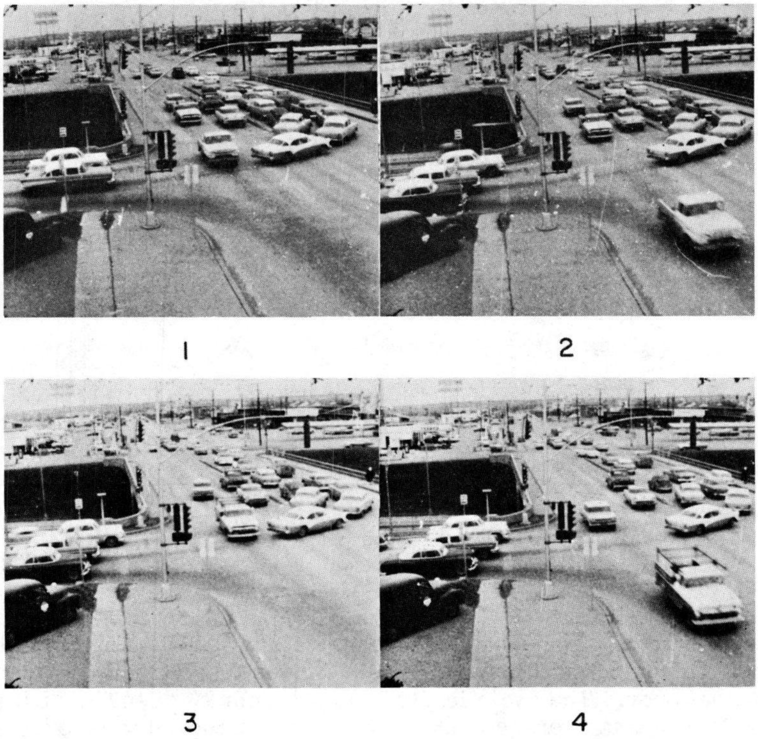


Figure 12. Congestion at intersection.

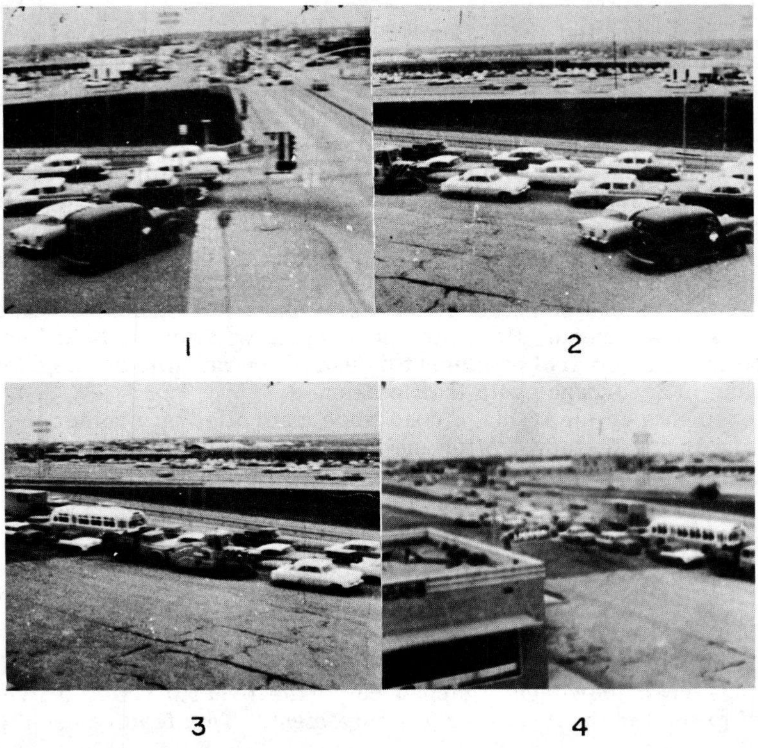


Figure 13. Congestion of ramp.

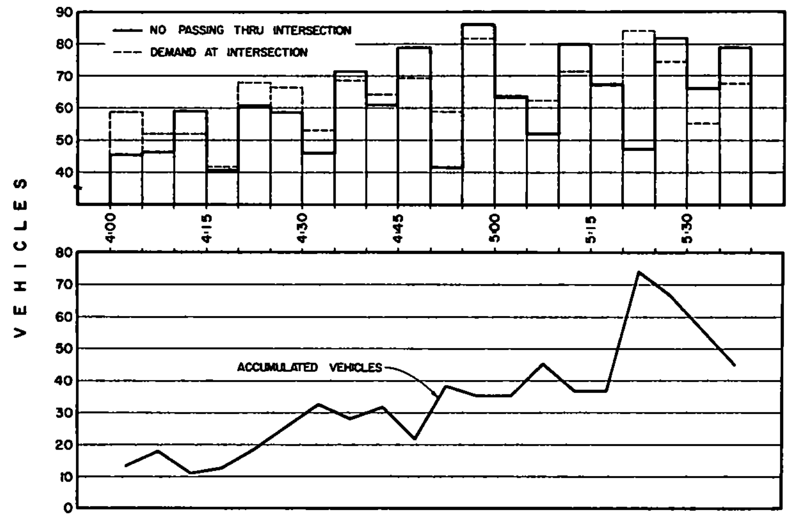


Figure 14. Accumulation of traffic demand.

The greatest disadvantage of the four-phase system was the long cycle lengths which occurred during the peak period. Table 2 gives the cycle and phase lengths recorded during the study. The cycle lengths ranged from 76 to 149 sec with an average of 105 sec. Although this average cycle length was considered to be excessive, it represented a 10 percent decrease over that obtained in the Berry I study.

Berry III: Four-Phase Operation with Overlap

The third study conducted at the Berry Street interchange evaluated operations with the recommended phasing arrangement shown in Figure 11. This phasing differed from that used in the Berry II study in that two overlap units were added to provide for an overlap of the traffic movements on the frontage road and major street approaches. The time required to move the major street traffic (starting delay plus travel time) from one approach to the other and the amber time required to terminate the frontage road movement were wasted by the sequence used in the Berry II study. By utilizing this wasted time, increased efficiency was obtained by the Berry III sequence.

Initially, it was reasoned that an effective overlap would be very difficult to obtain with actuated equipment. This was due to the fact that effective use of the overlap depended on initiating the major street green while there was still considerable traffic on the opposite frontage road. However, the overlap was accomplished very satisfactorily with the actuated control equipment by use of its variable settings for termination of a green phase in accordance with traffic demand.

Traffic movements on the frontage road were controlled by a minor movement controller as in the Berry II study. After the initiation of a frontage road green, a time gap in traffic less than the minimum vehicle interval (as set on the control unit) caused the signal control to pass to an overlap timing unit. This unit accomplished the following:

1. It terminated the left turn arrow at the opposite interior approach.
2. It initiated the major street green.
3. It timed a fixed overlap.

At the completion of this overlap phase, the frontage road green was terminated. When a frontage road movement "gapped out," the overlap timer provided an additional 5 sec of green for the frontage road movement. This feature permitted a very low setting of the vehicle interval dial on the minor movement controllers (approximately 2 sec) and provided an efficient overlap of the frontage road and major street traffic. With the 2-sec allowable gap on the frontage road, an extension of the green indication required a closely spaced platoon of vehicles. As soon as this platoon passed the frontage road detectors, the controller "gapped out" and initiated the major street green. The additional green time provided for the frontage road movement by the overlap unit was usually sufficient to clear all "stragglers" on the frontage road approaches. If there was no traffic on a frontage road approach during the overlap phase, no time was wasted because the major street green had already been initiated.

The study indicated that the addition of the overlap units greatly increased the efficiency of operation at the interchange. The cycle lengths during the study period varied from 58 to 107 sec with an average cycle length of 77 sec (Table 3). This average cycle

TABLE 2

BERRY II DATA

CYCLE NO.	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	CYCLE LENGTH	TOTAL GREEN	TOTAL VEHICLES
1	21	18	21	10	25	22	14	9	89	81	52
2	21	17	21	16	28	27	14	8	92	84	68
3	22	18	31	25	33	36	25	13	118	111	92
4	31	31	24	18	26	27	20	8	109	101	84
5	19	19	35	17	24	24	24	12	110	102	72
6	23	28	27	23	43	46	10	5	111	103	102
7	24	28	16	14	32	37	30	18	111	102	97
8	21	23	23	22	36	38	14	12	104	94	95
9	24	17	33	24	17	17	28	14	110	102	72
10	23	27	17	17	33	34	12	5	93	85	83
11	27	31	28	22	29	27	27	11	119	111	91
12	25	23	18	16	23	23	14	7	88	80	69
13	16	13	18	15	26	14	10	3	78	70	45
14	21	16	29	25	22	15	15	9	96	87	65
15	27	22	26	23	24	23	17	10	102	94	78
16	28	24	29	25	19	21	10	3	94	86	73
17	23	24	10	7	22	20	18	8	82	73	59
18	25	20	15	19	30	28	22	10	109	92	77
19	20	26	27	17	28	35	24	14	108	99	92
20	23	23	31	32	41	40	14	7	118	109	102
21	23	20	20	18	32	28	28	16	111	103	82
22	30	35	38	40	26	29	13	7	115	107	111
23	25	32	20	19	27	27	29	11	107	101	89
24	34	41	28	26	25	26	27	16	125	114	109
25	29	37	30	27	26	32	30	14	123	115	110
26	35	42	37	37	41	37	13	6	134	126	122
27	20	19	37	44	25	20	28	16	119	110	99
28	26	32	23	21	29	32	14	7	101	92	92
29	24	22	38	26	30	38	16	6	116	108	92
30	23	30	13	9	26	19	14	7	85	76	65
31	23	20	33	22	21	22	10	5	94	87	69
32	21	20	23	20	25	19	22	12	99	91	71
33	19	18	20	18	25	16	10	3	83	74	55
34	17	16	22	18	25	18	29	18	103	93	70
35	25	24	24	23	19	12	25	15	100	93	74
36	21	23	21	22	27	19	10	7	87	79	71
37	26	20	19	17	24	23	20	13	97	89	73
38	24	22	38	37	26	28	30	12	126	118	99
39	30	38	37	41	22	26	19	13	115	108	118
40	41	52	38	34	34	35	28	14	149	141	135
41	33	52	20	18	28	28	19	11	106	100	109
42	26	28	31	24	27	27	17	6	109	101	85
43	30	28	30	25	16	16	19	9	102	95	78
44	27	29	38	30	34	34	30	15	137	129	108
45	22	27	25	24	38	41	19	11	121	104	103
46	21	22	16	17	33	29	12	5	91	82	73
47	27	28	31	28	16	16	20	7	104	94	79
48	21	24	15	12	23	28	10	4	76	69	68
49	15	8	23	22	16	15	18	9	79	72	54

length was a 26.7 percent reduction from that recorded in the Berry II study and a 31.6 percent reduction from that recorded in the Berry I study.

In addition to reducing the average cycle length to a satisfactory level during the peak period, the signal system operated with a very high degree of efficiency. The actuated equipment permitted the allotment of green time to each phase in accordance with traffic demand and assured the clearance of all approaches during each cycle. The equipment also provided the flexibility to cope with unusual conditions (stalled vehicles, etc.) that occurred during the peak period.

TABLE 3

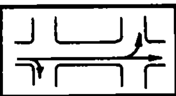
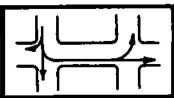
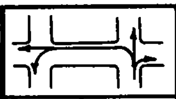
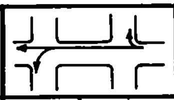
BERRY III DATA

CYCLE NO	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	CYCLE LENGTH	TOTAL GREEN	TOTAL VEHICLES
1	15	7	20	14	14	14	16	5	64	65	40
2	16	8	27	20	14	9	20	14	73	77	51
3	15	7	23	15	22	19	25	21	80	85	62
4	19	8	21	20	17	11	20	21	76	77	60
5	15	5	22	18	25	15	23	16	80	85	54
6	15	6	25	25	25	24	25	30	86	90	85
7	15	6	21	24	14	10	22	21	68	72	61
8	16	6	18	14	20	13	18	10	67	72	43
9	17	7	19	14	14	10	26	28	72	76	59
10	18	8	30	25	14	14	46	37	102	108	84
11	15	11	19	19	23	18	36	35	90	93	83
12	22	11	30	31	25	15	35	32	107	112	89
13	15	9	30	27	25	18	32	28	97	102	82
14	23	12	18	21	24	11	16	19	79	81	63
15	17	8	31	26	22	17	25	22	91	95	73
16	15	6	20	13	15	11	24	17	68	74	47
17	15	8	15	12	14	7	20	15	59	64	42
18	15	5	16	8	14	6	27	17	67	72	36
19	17	7	20	14	22	17	14	13	71	73	51
20	15	5	18	16	14	12	18	15	61	65	48
21	15	4	19	11	14	13	22	16	65	70	44
22	15	9	19	13	15	10	19	13	63	68	45
23	15	3	23	19	10	13	23	20	78	81	55
24	16	9	17	11	22	12	31	28	81	86	60
25	22	12	17	15	31	21	18	20	84	88	68
26	15	6	19	17	14	10	21	24	64	69	57
27	15	5	15	2	21	13	26	26	73	77	46
28	17	7	24	24	32	24	17	18	85	90	73
29	17	10	18	18	19	16	20	19	70	74	63
30	15	8	18	15	16	12	18	13	62	67	48
31	17	4	22	23	17	18	25	19	76	81	64
32	15	6	15	6	18	17	15	12	58	63	41
33	18	4	15	4	26	20	23	14	77	82	42
34	15	1	15	8	33	23	20	21	79	83	53
35	15	7	15	6	20	16	19	15	64	69	44
36	22	12	25	32	15	14	12	21	76	74	79
37	15	7	24	24	14	16	19	15	66	72	82
38	22	8	21	22	32	30	22	24	92	97	84
39	20	9	24	24	35	30	28	27	102	107	90
40	17	9	21	20	23	15	24	26	82	85	70
41	17	6	23	24	27	17	28	30	90	95	77
42	15	3	19	22	24	21	18	10	71	76	56
43	15	4	20	22	15	6	17	12	62	67	44
44	17	5	22	15	18	14	17	7	69	74	41
45	14	4	18	18	15	11	19	15	61	66	48
46	15	6	17	8	22	15	34	23	83	88	52
47	14	4	19	18	17	10	21	14	67	71	46
48	15	5	23	21	15	12	19	28	67	72	66
49	14	8	26	23	37	28	18	18	91	95	77
50	18	10	21	30	30	26	19	18	83	88	84
51	23	11	29	36	15	15	37	33	99	104	95
52	14	3	17	10	30	26	31	27	88	92	66
53	14	5	26	22	25	25	29	30	89	94	82
54	17	8	21	28	33	24	19	17	85	90	77
55	17	9	25	30	30	28	18	21	86	90	88
56	14	6	27	27	18	13	18	8	73	77	54
57	17	8	22	19	26	20	27	24	87	92	71
58	15	4	16	14	35	25	31	27	91	97	70
59	17	9	18	19	30	23	17	14	76	82	65
60	14	5	29	29	17	11	24	20	80	84	65
61	14	5	22	12	22	17	19	19	73	77	53
62	19	6	15	12	20	14	16	15	61	70	47
63	14	6	22	5	20	18	18	17	69	74	46

Berry IV: Fixed-Time Operation

The fourth study conducted at the Berry Street interchange evaluated traffic operations with a fixed-time signal system. The fixed-time control was obtained by "short circuiting" some of the detector circuits and regulating the phase lengths by the "maximum time" dials on the actuated controllers. An 80-sec cycle was used with the phase lengths

TABLE 4
FIXED TIME DATA

CYCLE NO.									CYCLE LENGTH	TOTAL GREEN	TOTAL VEHICLES
	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT			
1	21	13	23	20	19	4	23	12	80	86	49
2	"	18	"	15	"	6	"	19	"	"	58
3	"	16	"	22	"	6	"	15	"	"	59
4	"	10	"	21	"	7	"	15	"	"	53
5	"	13	"	18	"	5	"	20	"	"	56
6	"	16	"	21	"	11	"	16	"	"	64
7	"	11	"	30	"	11	"	17	"	"	69
8	"	16	"	22	"	7	"	19	"	"	64
9	"	24	"	29	"	6	"	9	"	"	68
10	"	15	"	17	"	5	"	23	"	"	60
11	"	15	"	20	"	8	"	20	"	"	63
12	"	18	"	24	"	9	"	19	"	"	70
13	"	13	"	23	"	8	"	14	"	"	58
14	"	15	"	20	"	9	"	17	"	"	61
15	"	15	"	19	"	9	"	14	"	"	57
16	"	10	"	21	"	10	"	21	"	"	62
17	"	18	"	27	"	3	"	12	"	"	60
18	"	8	"	27	"	6	"	13	"	"	54
19	"	16	"	19	"	5	"	19	"	"	59
20	"	18	"	19	"	7	"	18	"	"	62
21	"	15	"	19	"	10	"	9	"	"	53
22	"	12	"	23	"	11	"	17	"	"	63
23	"	9	"	22	"	7	"	9	"	"	47
24	"	20	"	21	"	7	"	23	"	"	71
25	"	22	"	19	"	3	"	25	"	"	69
26	"	20	"	21	"	5	"	23	"	"	69
27	"	20	"	19	"	8	"	21	"	"	68
28	"	23	"	20	"	9	"	23	"	"	75
29	"	19	"	23	"	11	"	13	"	"	66
30	"	10	"	14	"	4	"	19	"	"	47
31	"	8	"	20	"	7	"	22	"	"	57
32	"	15	"	21	"	6	"	22	"	"	64
33	"	16	"	17	"	9	"	32	"	"	74
34	"	17	"	27	"	10	"	32	"	"	86
35	"	13	"	18	"	11	"	22	"	"	64
36	"	20	"	19	"	8	"	27	"	"	74
37	"	19	"	21	"	10	"	20	"	"	70
38	"	16	"	12	"	7	"	17	"	"	52
39	"	10	"	18	"	9	"	15	"	"	52
40	"	20	"	15	"	6	"	21	"	"	62
41	"	12	"	13	"	8	"	17	"	"	50
42	"	16	"	16	"	17	"	15	"	"	64
43	"	19	"	12	"	8	"	17	"	"	56
44	"	20	"	23	"	3	"	16	"	"	62
45	"	13	"	23	"	5	"	9	"	"	50
46	"	7	"	23	"	9	"	14	"	"	53
47	"	20	"	18	"	10	"	14	"	"	52
48	"	20	"	16	"	6	"	21	"	"	63
49	"	22	"	19	"	3	"	15	"	"	59
50	"	18	"	24	"	6	"	25	"	"	73
51	"	25	"	17	"	9	"	20	"	"	71
52	"	18	"	15	"	4	"	30	"	"	67
53	"	22	"	11	"	6	"	?	"	"	?
54	"	20	"	22	"	10	"	29	"	"	81
55	"	21	"	27	"	4	"	24	"	"	76
56	"	25	"	27	"	3	"	33	"	"	88
57	"	18	"	20	"	10	"	25	"	"	73
58	"	20	"	16	"	4	"	21	"	"	61
59	"	20	"	22	"	5	"	20	"	"	67
60	"	19	"	16	"	6	"	17	"	"	58
61	"	20	"	15	"	11	"	24	"	"	70
62	"	21	"	17	"	8	"	24	"	"	70
63	"	18	"	10	"	12	"	22	"	"	62
64	"	16	"	22	"	7	"	25	"	"	70
65	"	14	"	15	"	6	"	24	"	"	59

given in Table 4. This cycle length was determined from the traffic volumes to be handled on each approach and was comparable to the average cycle length obtained in the Berry II study. The phasing arrangement used was identical to that used in the Berry III study.

The inability of the signal system to adjust to fluctuating traffic demands created long queues on some of the approaches during the peak period. This forced a number of vehicles to wait for more than one cycle and thereby caused considerable vehicular delay.

Visual observation of traffic operation during the study indicated that the system was able to accommodate the traffic demand but with a much lower degree of efficiency than obtained with the Berry III actuated system. Vehicular delays were greater, and longer queues of waiting traffic were observed on all of the approaches.

Traffic movements through the interchange appeared to approach a congested condition during most of the peak period of flow. It is believed that the occurrence of any

TABLE 5
OFF-PEAK DATA (12:00 AM - 01:00 AM)

CYCLE NO	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	PHASE LENGTH	VEHICLE MOVEMENT	TOTAL GREEN	CYCLE LENGTH	TOTAL VEHICLES
1	17	0	0	0	34	1	14	3	65	69	4
2	16	0	14	1	14	4	14	2	58	60	7
3	16	0	14	1	12	1	14	3	56	57	5
4	15	4	14	1	12	3	14	1	55	57	9
5	16	2	0	0	7	2	12	3	35	30	7
6	12	3	0	0	6	0	14	1	32	35	4
7	15	1	14	1	12	4	14	2	55	57	8
8	15	1	0	0	11	5	14	2	40	45	8
9	16	1	0	0	6	2	14	2	36	41	5
10	16	0	0	0	6	1	14	1	36	40	2
11	21	1	0	0	6	1	0	0	27	35	2
12	11	2	0	0	6	1	15	1	32	34	4
13	15	1	15	1	13	3	15	1	58	56	6
14	16	4	0	0	6	1	15	3	37	40	8
15	17	5	0	0	6	0	0	0	23	31	5
16	10	2	0	0	6	0	15	4	31	32	6
17	16	3	0	0	10	3	15	3	41	43	9
18	16	4	0	0	72	5	0	0	88	99	9
19	8	2	0	0	7	1	0	0	15	22	3
20	68	5	0	0	6	0	15	2	89	95	7
21	16	0	0	0	6	4	15	2	37	40	6
22	7	1	15	1	13	2	0	0	35	46	4
23	9	3	0	0	6	0	16	1	31	31	4
24	21	0	15	1	13	2	16	2	65	61	5
25	17	1	0	0	36	3	16	1	69	69	5
26	17	2	0	0	6	1	0	0	23	29	3
27	9	0	15	0	13	0	17	4	54	50	4
28	20	3	15	1	13	0	16	1	64	59	5
29	17	4	0	0	6	1	16	1	39	39	6
30	44	2	0	0	6	0	16	1	66	66	3
31	17	0	0	0	6	0	16	7	39	40	7
32	17	1	15	1	18	1	16	5	66	57	8
33	17	1	15	2	14	2	16	3	62	58	8
34	17	0	0	0	6	1	16	3	39	39	4
35	24	3	15	0	13	0	16	3	68	64	6
36	24	1	0	0	6	0	16	2	46	46	3
37	17	1	0	0	11	1	16	2	44	43	4
38	17	1	0	0	6	0	16	4	39	39	5
39	17	2	15	1	13	2	16	2	61	57	7
40	17	0	0	0	6	2	16	2	39	40	4
41	17	1	0	0	6	0	16	2	39	40	3
42	18	5	15	1	13	2	16	5	62	58	13
43	17	2	0	0	6	0	16	1	39	39	3
44	17	0	0	0	6	0	16	1	39	39	1
45	17	0	0	0	6	5	16	3	39	39	8
46	17	0	0	0	6	1	16	4	39	39	5
47	17	2	16	1	13	2	16	2	62	57	7
48	18	2	0	0	6	1	0	0	24	30	3
49	14	2	0	0	6	0	16	3	36	37	5
50	17	1	15	1	12	1	16	1	60	56	4
51	17	3	0	0	75	4	0	0	92	97	7
52	15	4	0	0	7	0	16	1	38	37	5
53	22	2	0	0	8	1	16	2	46	46	5
54	18	1	0	0	11	3	16	2	45	44	6
55	17	2	15	0	13	0	16	1	61	56	3
56	22	2	15	0	22	2	16	1	75	70	5
57	51	0	0	0	6	0	16	1	73	73	1

unusual condition temporarily interrupting the normal flow of traffic would have created excessive demands on all approaches. Because this system lacked the flexibility to vary cycle lengths, the congestion would have remained until the traffic demand diminished.

Off-Peak Operation

A study of off-peak operation was also conducted to evaluate the efficiency of the actuated equipment during periods of low traffic demand. This study was conducted from 12:15 to 1:00 a.m. on a Thursday. The signal control equipment was the same as that used during the Berry III study.

Data on approach volumes, cycle lengths, and phase lengths were recorded during the off-peak study (Table 5). Analysis of these data plus observations made during the study indicated that satisfactory operation could be obtained during off-peak hours.

The cycle lengths recorded during this study are not indicative of the actual operation. Because there were some cycle lengths that ranged from 50 to 97 sec, it is possible that inefficient operation occurred. However, the long cycles resulted from the equipment "dwelling" on a particular phase in the absence of traffic demand from any other approach. This did not result in inefficiency, however, because traffic on any approach was accommodated as soon as the demand was indicated.

The short cycle lengths that were recorded show the efficiency obtained. The cycle length data indicated that 25 out of 57 cycles had a length of 40 sec or less with a minimum cycle length of 22 sec. Therefore, when enough traffic was present to cause the equipment to cycle, minimum cycle lengths were observed.

Another advantage observed was the ability of the equipment to omit or "skip" phases that had no traffic demand. The data indicated that the frontage road phases were skipped 47 out of 114 times during the off-peak study.

This study revealed one possible problem that may be encountered during low-volume operation. If a vehicle desiring to make a U-turn arrives on a frontage road approach at a time when no other traffic is present at the interchange, there is a possibility that this vehicle will not clear through the entire interchange. This vehicle will be "trapped" on the interior approach while the signal remains on the following major street phase.

The off-peak studies at the Berry Street interchange indicated no "trapping" of U-turn vehicles, and it is felt that the problem of "trapping" U-turning vehicles is not as serious as it may appear. An actuation of a detector on any other approach will cause the signal to advance and release the U-turning vehicle. This problem may also be eliminated by the provision of a U-turn lane.

EVALUATION OF CONTROL EQUIPMENT

Study Results

The operational studies conducted at the Berry Street interchange provided useful data for evaluation of traffic control equipment and phasing arrangements for diamond interchange signalization. A summary of the data recorded during each of the studies is given in Tables 1 through 5.

The individual cycle lengths recorded during each study proved to be a good indication of the relative efficiency of each system. A plot of this cycle length data (Fig. 15) shows the fluctuations experienced during the four separate studies. The improved efficiency obtained during the Berry III study can be seen by comparing the cycle lengths experienced during this study with those recorded during the Berry I and Berry II studies. The Berry III study had a maximum cycle length of 107 sec and an average cycle length of 77 sec. This indicates a significant reduction in individual vehicular delay when compared with the Berry I and II studies. The Berry II study also showed an improvement over conditions existing in the Berry I study. The significance of the cycle length reduction is emphasized by the increase in efficiency and reduction in congestion observed to accompany the cycle length reduction.

Another comparison of the relative efficiency of each of the systems was made from the data presented in Table 6 which shows the total volume of vehicles moved through the interchange during the period 4:00 to 5:20 p. m. This volume actually represents the traffic demand for this period, as it is essentially the same for all of the studies. Comparison of the total interchange volume with the average cycle length for each of the studies indicates the significant improvement in efficiency and reduction in delay obtained with the four-phase overlap system.

TABLE 6

INTERCHANGE DATA, 4 TO 5:20 PM

Study	System	Average Cycle Length (sec.)	Interchange Volume (no.)
I	3-phase	118	3,856
II	4-phase	105	3,930
III	4-phase/overlap	77	3,825
IV	Fixed time	80	3,778

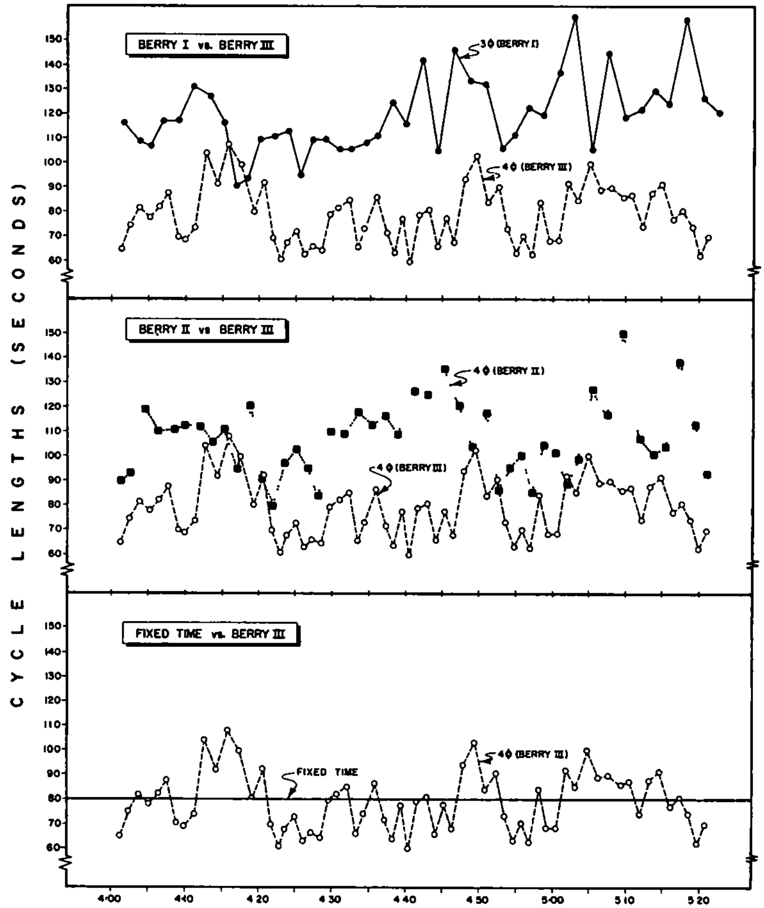


Figure 15. Cycle length comparisons.

A comparison of the Berry III cycle length plot with the 80-sec fixed-time cycle (Fig. 15) shows the inefficiency of a fixed cycle length during a peak period. The traffic-actuated equipment constantly adjusted to the varying traffic demand with a resultant variation in cycle length above and below the 80-sec fixed-time value.

A further indication of efficiency was given by a plot of the cycle lengths vs the number of vehicles moved on all approaches (Fig. 16). This plot was made more realistic by expanding the number of vehicles moved per cycle to the number of vehicles that could be moved in an hour for a given cycle length (Figure 17). It should be realized that this figure represents a hypothetical condition and shows the volume of vehicles that could be moved with each system provided there were vehicles on the

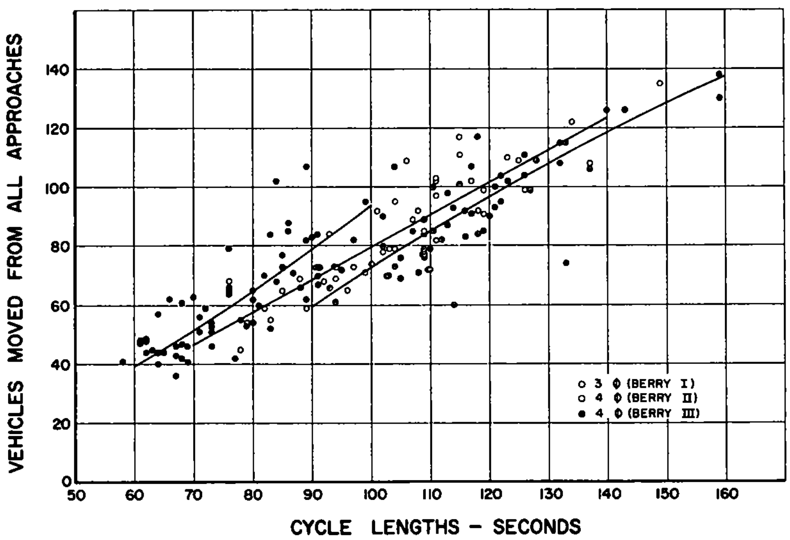


Figure 16. Plot of cycle lengths vs number of vehicles moved.

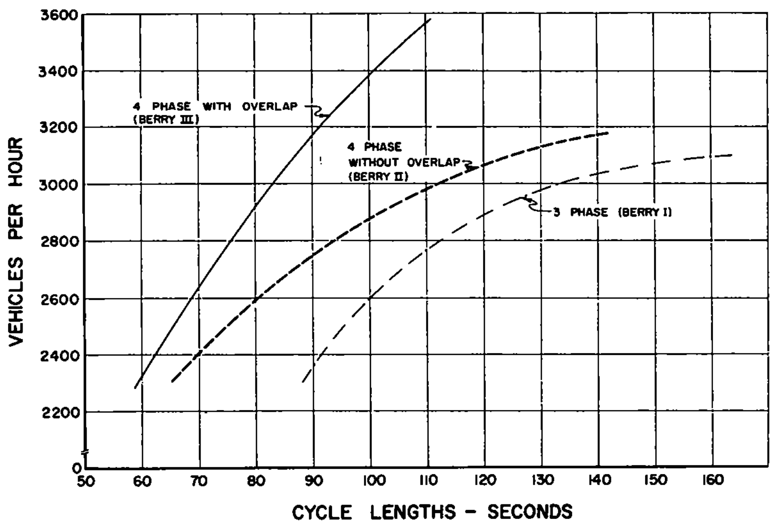


Figure 17. Plot of cycle lengths vs number of vehicles moved per hour.

approaches at all times during the hour. However, a relative measure of operation can be obtained from these curves. For example, suppose 3,000 vehicles per hr were to be moved through a diamond interchange. Referring to Figure 17, the following average cycle lengths would be required:

1. Berry III sequence, 85 sec;
2. Berry II sequence, 115 sec;
3. Berry I sequence, 134 sec.

This shows the ability of the Berry III sequence (as compared with the Berry I and Berry II sequence) to move the same traffic volume with a reduced cycle length and therefore a significant reduction in vehicular delay.

Visual observations during the studies gave the best indication of the efficiency of each of the signal systems. The congestion that occurred during the Berry I study (previously shown) was far from desirable. The peak traffic caused a breakdown of normal traffic operation on the facility and created long queues of vehicles on all approaches.

The Berry II study reflected a much more efficient operation. The vehicular movements were handled with only minor congestion during the peak hours. An actual emergency, which occurred during this study, disrupted the flow of traffic for a period of approximately 5 min and created large backlogs of traffic on all approaches. The system demonstrated its flexibility by clearing the backlog of traffic and returning to normal operation within a few cycles after the traffic disturbance was removed.

Observation of traffic movements during the Berry III study indicated operations very similar to Berry II. The shorter cycle lengths permitted more frequent green periods for each of the approaches and therefore eliminated the accumulation of long queues of vehicles. This type of operation moved the vehicles so efficiently that the peak period was not apparent to observers. This system also demonstrated the ability to adjust to unusual conditions with a minimum amount of congestion and delay.

Conclusions

The Berry Street studies completed a series of operational studies at diamond interchanges that utilized both fixed-time (Cullen and Wayside interchanges, Houston, Tex.) and traffic-actuated equipment (Berry Street interchange, Fort Worth, Tex.) It can be concluded from these studies that traffic-actuated, volume-density control equipment is the most desirable for use at signalized diamond interchanges. This conclusion is based on the following:

1. The traffic studies indicated that traffic demand at diamond interchanges fluctuates widely with respect to both individual approaches and total interchange volume. This is true of peak as well as off-peak flow. It was also observed that stalled vehicles, minor accidents, or other similar disruptions to normal traffic flow were a common occurrence during peak periods of traffic flow. These disruptions created a temporary need for increased cycle lengths in order to clear accumulated vehicles. Therefore, there was a demonstrated need for a flexible control system at diamond interchanges. If maximum operational efficiency is to be obtained, the cycle and phase lengths of the control system must be responsive to traffic demand.

2. In the past, some engineers have considered fixed-time control systems to be necessary at diamond interchanges in order to provide progressive movement for the through traffic on the major artery. Analyses of traffic movements at diamond interchanges showed that the major street through-movement represented only 20 to 30 percent of the total traffic moving through the interchange.

In view of the small percentage of through movement on the major street (as compared to total interchange volume), the provision of maximum operational efficiency for all movements through the interchange should receive primary consideration. Therefore, fixed-time systems are not warranted on the basis of providing progressive movement for major street through-traffic.

3. The Berry Street interchange studies demonstrated that actuated equipment could be adapted to provide the special sequences and overlaps required for handling diamond interchange traffic.

Control Equipment

Two basic traffic control systems are recommended for conventional-type diamond interchange signalization. System I (Fig. 18) uses a two-phase volume-density controller in conjunction with two minor movement controllers. Traffic on the major street (A and B phases) is controlled by the volume-density controller, and traffic on each frontage road (A' and B' phases overlap) is controlled by a minor movement controller. In addition, special timing units are used to provide for the overlap phases.

System II (Fig. 18) uses a three-phase volume density controller in conjunction with one minor movement controller. Traffic on the major street is controlled by the A and B phases of the volume-density controller. One of the frontage road movements is controlled by the C phase of the volume-density controller, and the other frontage road movement is controlled by a minor movement controller.

The detectors for each of these systems are located as shown in Figure 18. The major street detectors are placed a minimum of 250 ft back from the stop lines. This distance is required to obtain the advantages of the volume-density features. Detectors on the the frontage roads are located 60 ft from the stop line. This allows slow moving vehicles to travel from the detectors to the intersection during the overlap time at the end of a frontage road phase and permits clearing of the frontage road approaches. This detector spacing also facilitates introduction of a U-turn lane.

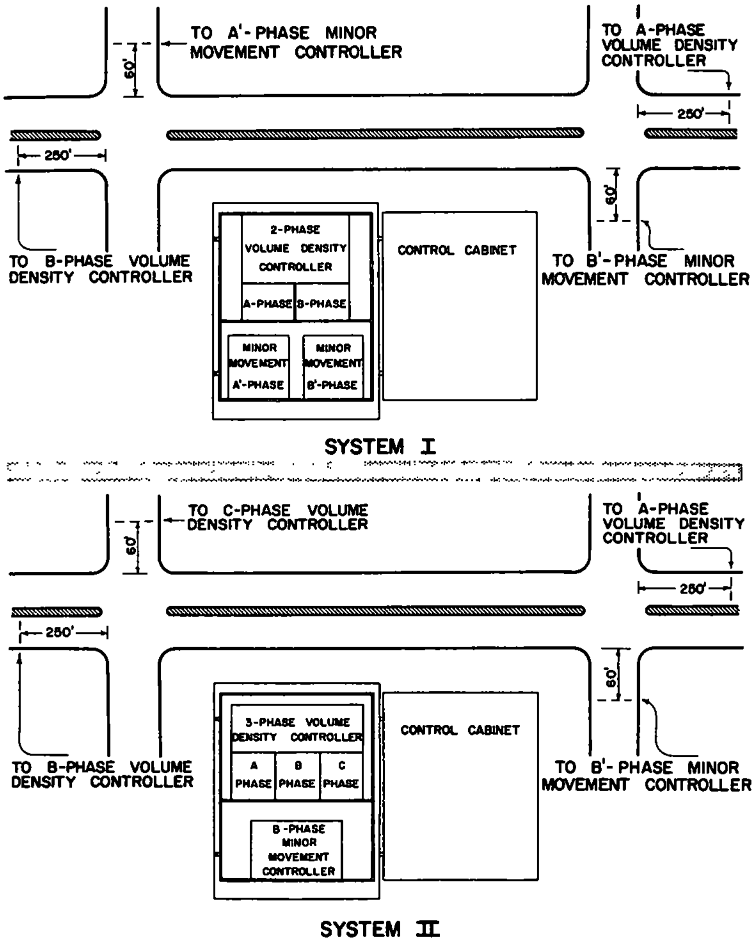


Figure 18. Basic control equipment.

System II has advantages over System I in that one of the frontage road movements can be controlled with the volume-density controller. However, either of the two systems recommended will provide a highly efficient control system for a conventional-type diamond interchange.

DESIGN ASPECTS OF DIAMOND INTERCHANGES

Design Volume

In developing the studies on diamond interchanges, it was recognized that actual traffic demand must be measured in order to determine accurately the amount of traffic that should be accommodated at a signalized intersection. This can be done by counting traffic in advance of the traffic queues on each approach as shown in Figure 19. As the traffic queues lengthen, the counting line is moved so that vehicles are counted when they are still moving at approximately 10 to 15 mph. Such counts provide an accurate measure of traffic demand at intersections.

Early in the interchange studies it became evident that demand volumes fluctuate greatly during periods of peak flow. To obtain a good measure of this fluctuation, demand volumes were recorded by 5-min periods and by each signal cycle.

Figure 20 shows typical variation of traffic demand on an approach to a diamond interchange during a peak period. This fluctuation should be considered in all designs.

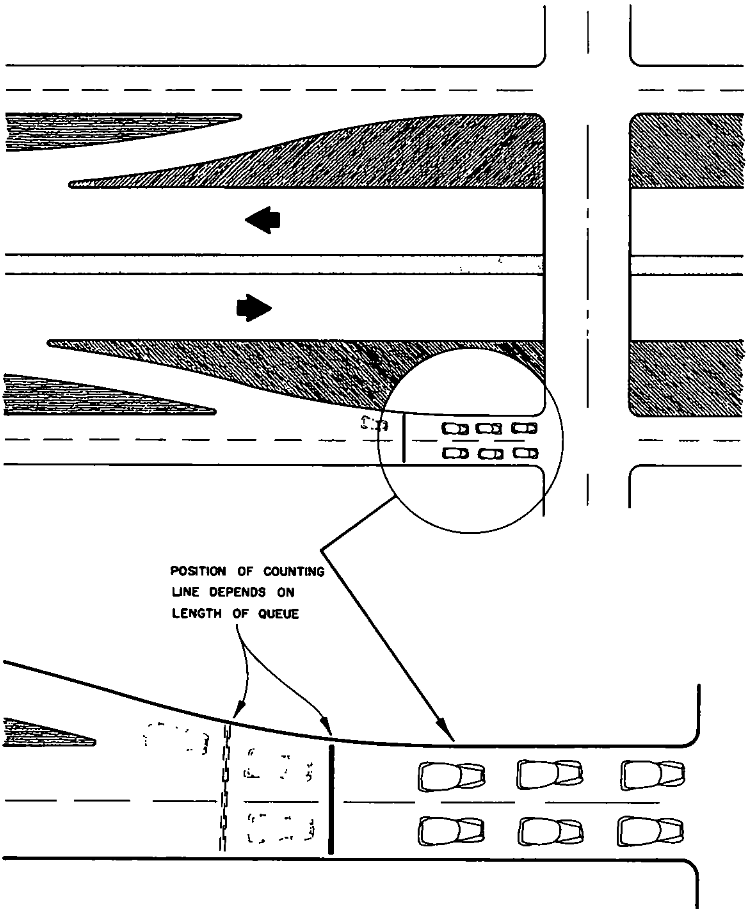


Figure 19. Procedure for determining traffic demand.

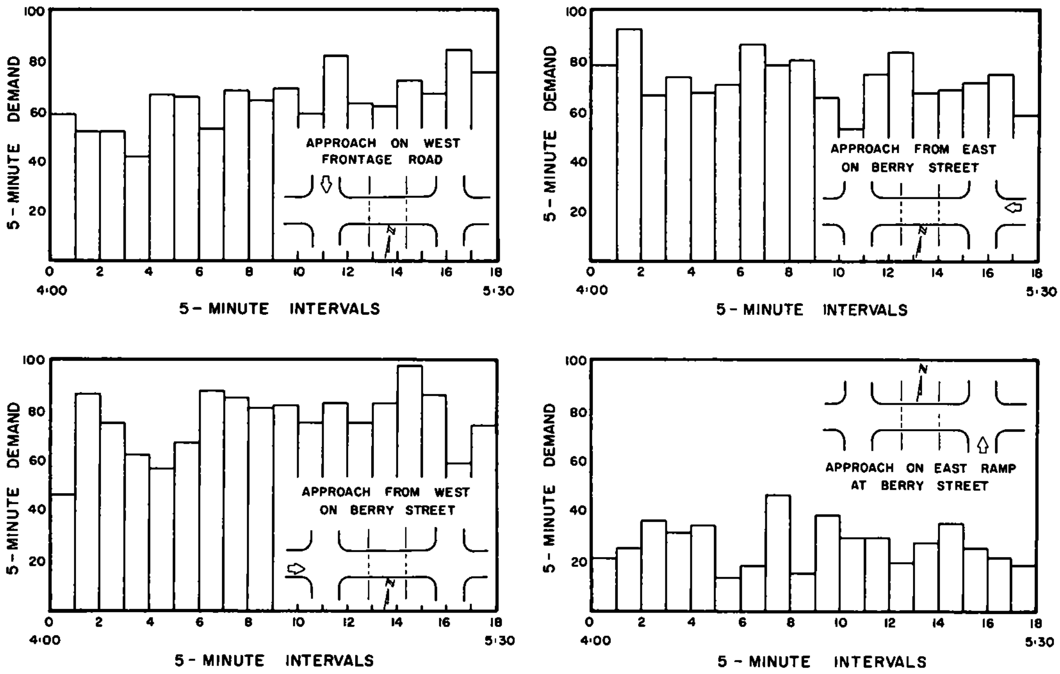


Figure 20. Demand fluctuations, Berry Street interchange, Fort Worth, Tex.

The total hourly volume is not sufficient for design because this volume is greatly exceeded during the peak 30 min of flow. Thus it becomes necessary to design on a period of less than an hour, or to adjust the hourly volume to take the peak hour fluctuations into account.

This peak hour adjustment was considered significant enough to warrant special study. A study of peak-hour traffic flow at signalized intersections was conducted and analyzed by Drew (2).

It was found that the average signalized intersection on a major artery in an urban area will experience a peak approximately 30 min in length and that 55 to 60 percent of the total hourly demand will occur during this peak 30-min period. On the basis of these studies, it was decided to increase all hourly volumes by a factor of 1.15 to obtain a proper design figure. This procedure is followed in all design examples presented in this report. Drew (2) should be consulted for a more detailed discussion of peak-hour demand fluctuation.

Lane Assignment

In working with the capacity design procedure presented in the authors' report (1), it is necessary to develop a lane assignment for the traffic volumes from each approach. Therefore, it is important that the design traffic data provide volume and turning movement information from which critical lane volumes can be determined. Studies of lane distribution at intersection approaches indicated that, in general, traffic distributes equally over the approach lanes. High-volume turning movements may require special consideration and the critical lane volume should be increased slightly to allow a factor of safety. The determination of the critical lane volumes requires a thorough study of the traffic movements on each of the approaches. No definite procedure can be established for this determination inasmuch as it is greatly dependent on engineering judgment.

After a critical lane volume is determined for an approach (based on some assumed design), this volume can be used for design because the adjacent lanes with smaller volumes will move during the same time.

Right Turn Lanes

The purpose of the diamond interchange is to move traffic between a major street and a freeway system. This interchange movement develops an inherently heavy right turn at each of the four approaches (Fig. 21). Consequently, it is important to give major consideration to these movements in the design of the interchange.

The operation of free right turn lanes at high-volume diamond interchanges has not been good and indicates that the right turn movement should be controlled by signals. Poor operation has also developed on interchanges that have inadequate turn radii for the right turn movements. Therefore, it was concluded that the right turn movement should be given special attention by a design such as that shown in Figure 22. With this design, the right turn movement is controlled by the signal but is greatly facilitated by the improved geometrics. In cases where the right turn volume is extremely heavy, provision should be made for turning two lanes simultaneously.

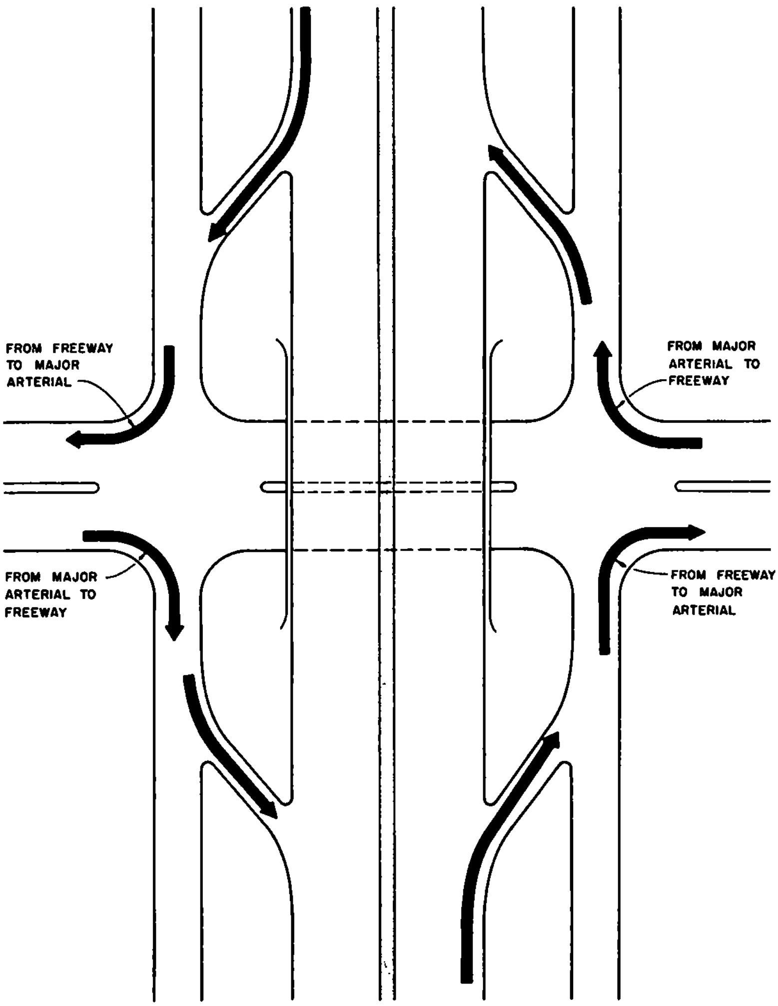


Figure 21. Interchange movements.

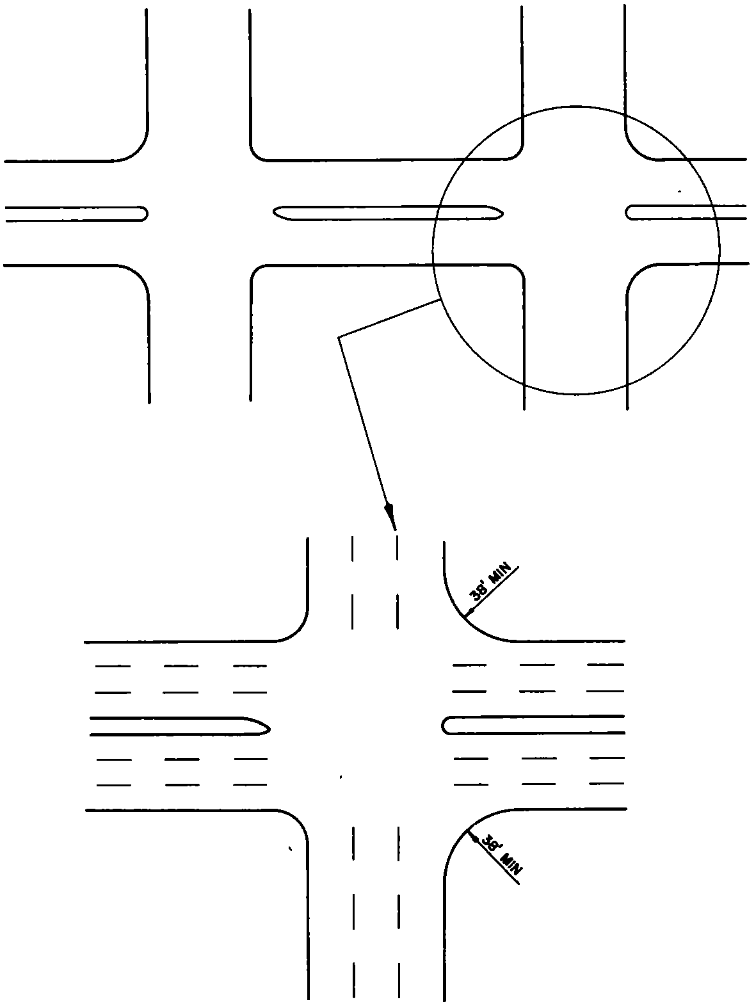


Figure 22. Desirable geometrics for right turn movements at diamond interchange.

Number of Lanes on Interior Approaches

Results of operational studies on diamond interchanges indicated that the number of lanes to be provided on the interior approaches is governed by the major street approaches. The same number of lanes should be provided on the interior approaches as will ultimately be required for the major street approaches.

Previously, there has been special consideration given to providing an additional lane on each of the interior approaches to obtain a separate left turn lane. This separate left turn lane is required if any phasing arrangement other than that shown in Figure 4 is used. This method of operation will not be satisfactory, however, if heavy left turn movements are experienced. The storage room that can be provided will not accommodate more than 7 vehicles per lane.

With the four-phase operation, left turns from the interior approaches are not critical. This is true because only U-turning vehicles arriving during the last 8 to 10 seconds of a frontage road phase are required to store on the interior approaches. Therefore, very satisfactory operation can be obtained if the same number of lanes are provided on the interior approaches as on the major street approaches.

U-Turn Lanes

A desirable feature that can be incorporated into the design of diamond interchanges is a U-turn lane. This lane provides for the free movement of traffic from one frontage road to another. This extra lane normally requires additional structure and thus additional expense. There is a question as to the justification of this additional cost.

The U-turn movement from a frontage road is the most difficult to handle of all interchange movements. This movement involves two left turns through the intersection area, and large volumes of U-turning traffic can cause a complete breakdown of traffic operation.

The warrant for a U-turn lane depends on the demand for the U-turn movement. However, a good estimate of future U-turning traffic is very difficult to predict during the design stage. The U-turn movement is created by traffic generators located ad-

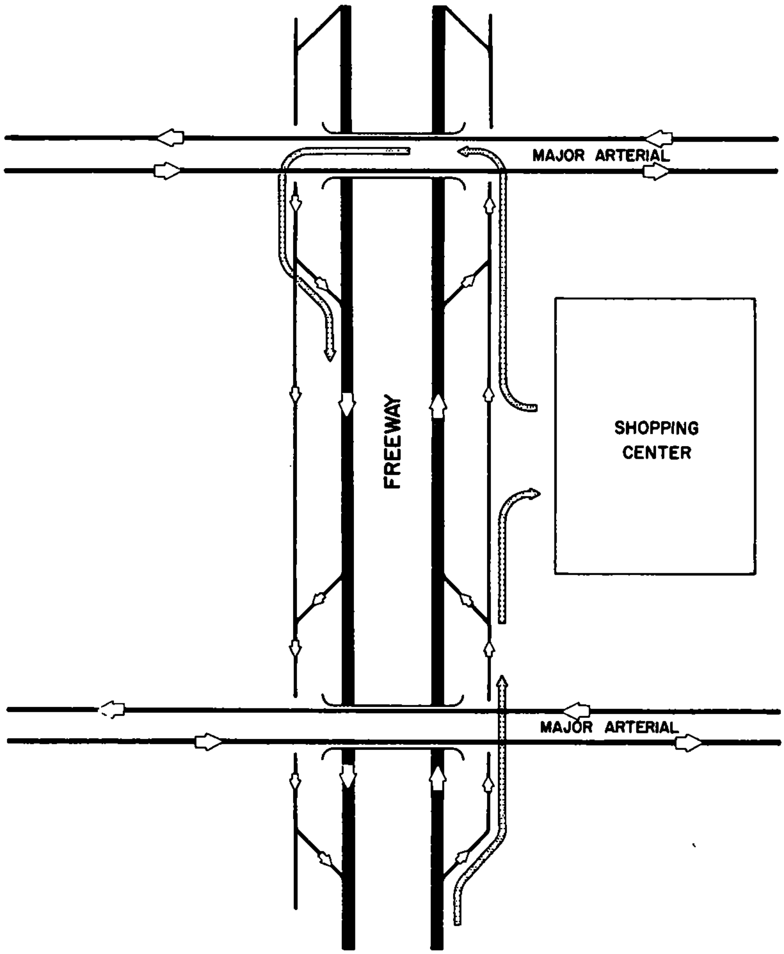


Figure 23. Generation of U-turn movements.

jacent to the frontage roads; and the location, installation date, and impact of such generators are almost impossible to predict accurately.

An excellent example of how a heavy U-turn movement can develop is shown in Figure 22. A large shopping center is being constructed adjacent to the frontage road on IH 35W in Fort Worth, Tex. This shopping center is expected to create a very heavy traffic movement of the type shown in this figure. This will generate a heavy U-turn movement at the Seminary Drive interchange and will require a modification of this interchange. An analysis of the expected volumes and their effect on the interchange is presented in the example section of this report.

Thus, in urban areas where extensive future land development is likely to occur along the frontage roads, the provision of U-turn lanes is a relatively inexpensive measure that will insure against the interchange becoming inadequate for the developing U-turn movements which cannot be predicted.

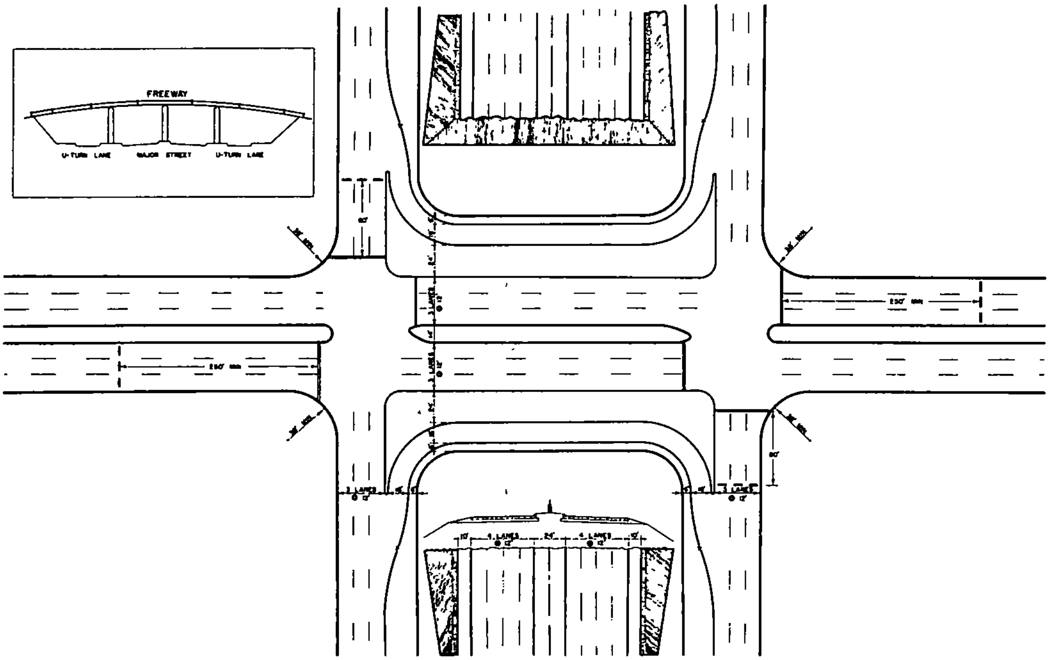


Figure 24. Desirable geometrics for conventional-type diamond interchange.

Desirable Design

Figure 24 presents a desirable design incorporating the previously discussed design factors. This design, when incorporated with the recommended traffic control equipment, will produce a highly efficient interchange. The number of lanes required on each of the approaches is a function of the design volumes and should be obtained by a capacity analysis. The geometric features (such as turn radii and island location) are applicable to a design for any number of lanes.

EXAMPLES OF DIAMOND INTERCHANGE

Regardless of the efficiency of the signal system used at diamond interchanges, satisfactory operations cannot be obtained when traffic demand exceeds the capacity of the interchange. Thus, in addition to the recommended signal controls, adequate interchange capacity must be provided by the initial design.

Many existing diamond interchanges are presently experiencing congestion with a resultant inefficiency in operation. Possible improvement could be obtained at these interchanges by a critical capacity evaluation of their present operation. Modifications of the design and/or signalization may be necessary to accommodate present or future traffic demands.

A capacity-design procedure for designing and evaluating diamond interchanges was developed and reported during the first phase of the diamond interchange studies. This report (1) is available for detailed information on diamond interchange capacity. Three examples of a capacity-design analysis for diamond interchanges are presented in this report to emphasize and explain the design procedure further.

Design of Future Interchanges

For a design example of a future interchange, the intersection of the West Loop Freeway and Richmond Road in Houston, Tex., was considered. Future ADT volume assignments for this intersection were obtained from the Houston Urban Study and are shown in Figure 25. Assuming a K-factor (percent ADT) of 10 percent and a D-factor (directional distribution) of 60 percent, the interchange volumes shown in Figure 25 were developed.

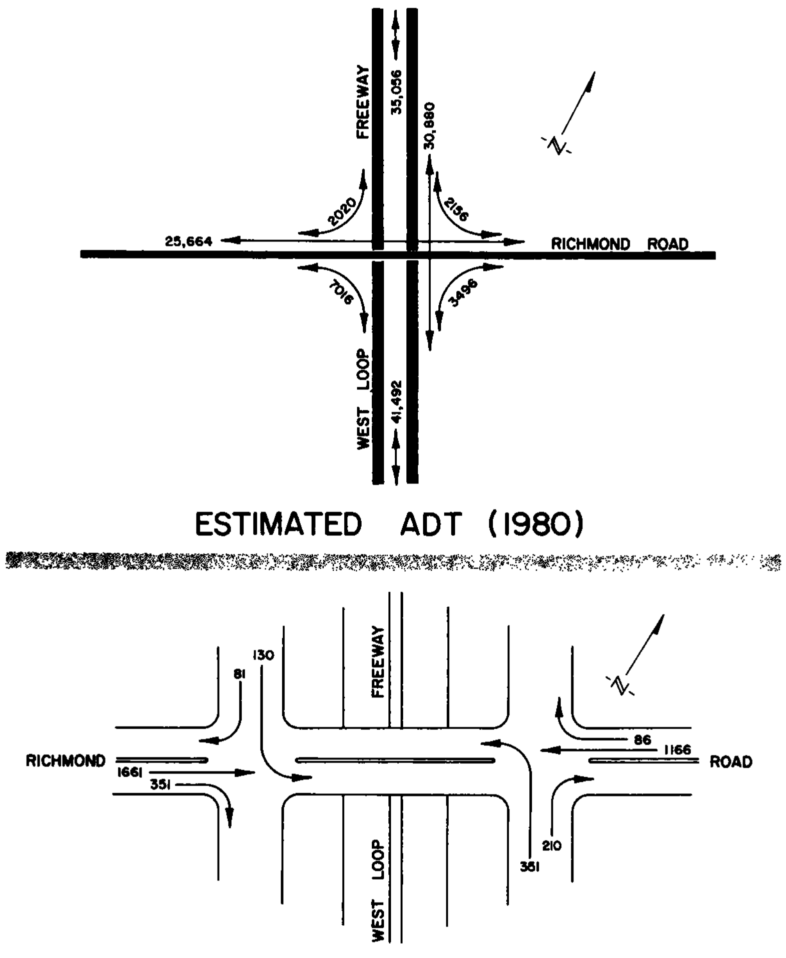


Figure 25. Estimated hourly volumes (1,980), a. m. peak.

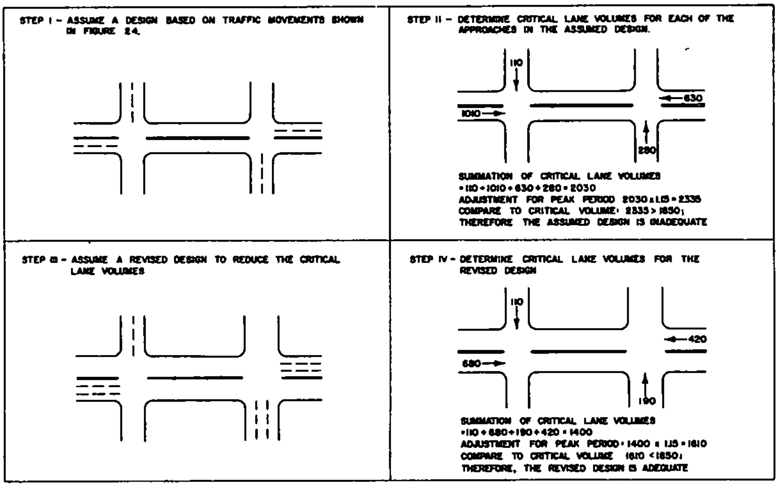


Figure 26. Sample calculations for diamond interchange, West Loop Freeway and Richmond Road, Houston, Tex.

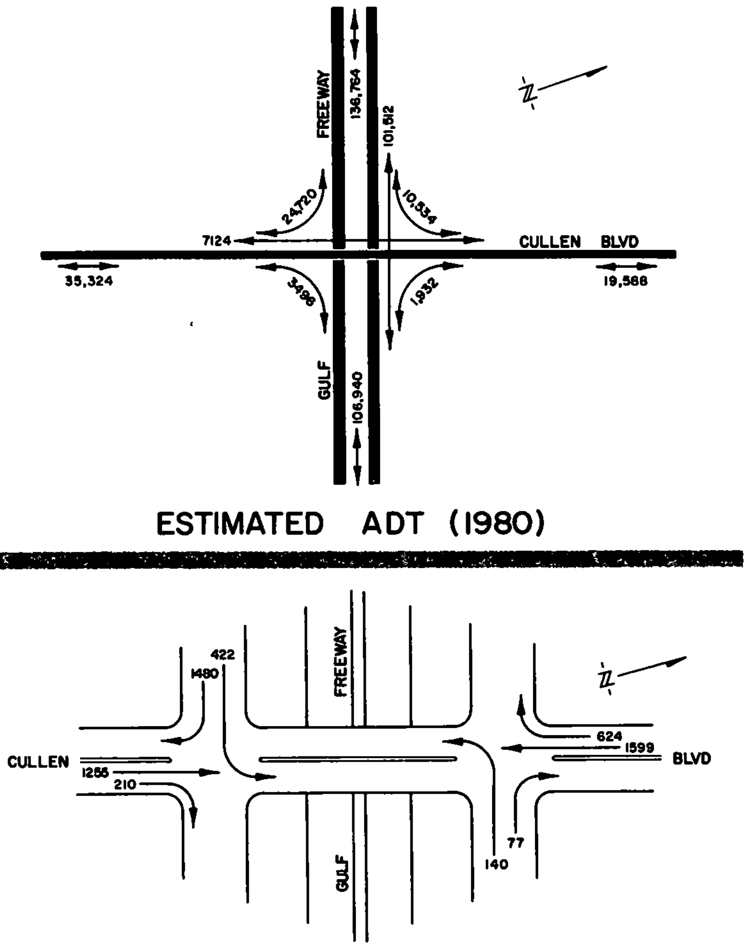


Figure 27. Estimated hourly volumes (1,980), p. m. peak.

Figure 26 shows the steps necessary to evaluate and design a conventional-type diamond interchange for use at this intersection. The calculations indicated that a conventional-type diamond interchange would be adequate for this location if the required number of approach lanes and adequate signalization were provided.

It is apparent that some interchange volumes when subjected to this type of analysis would yield unreasonable designs. Such results would indicate a higher type of direc-

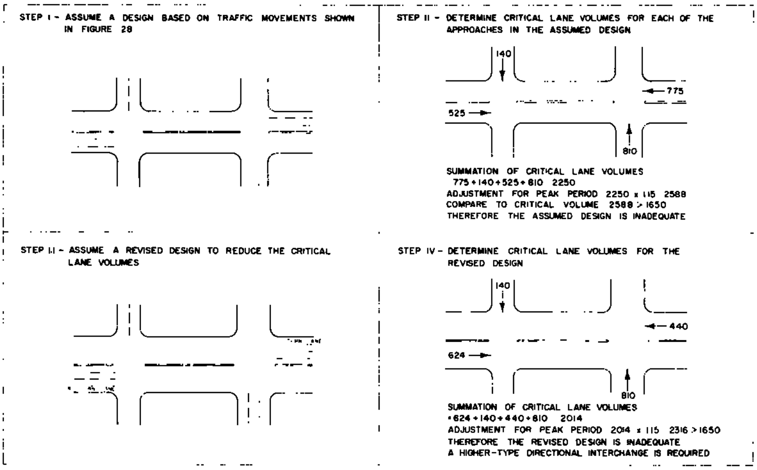


Figure 28. Sample calculations for diamond interchange, Gulf Freeway and Cullen Boulevard, Houston, Tex.

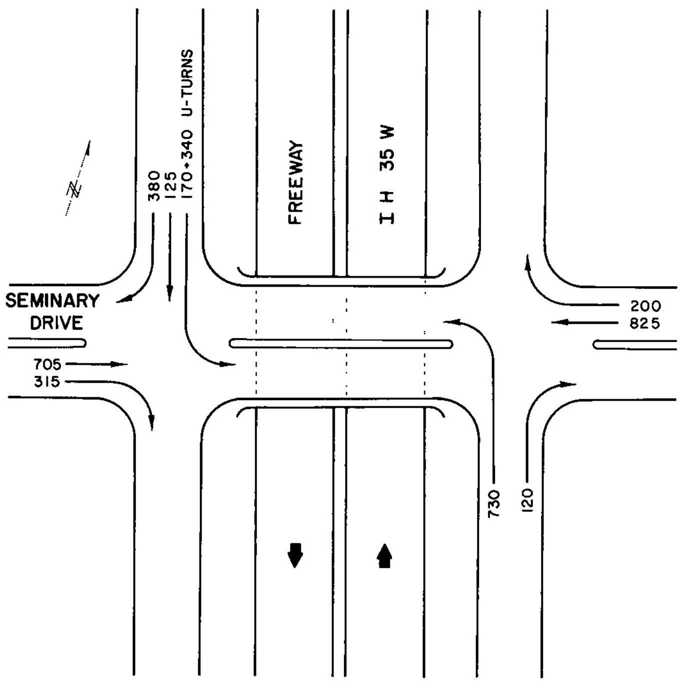


Figure 29. Estimated peak-hour volume (1,975), a. m. peak, Seminary Drive, Interstate 35W, Fort Worth, Tex.

tional interchange is warranted. An example of such a volume condition is shown in Figure 27. The analysis of these data (Fig. 28) shows that a conventional-type diamond interchange would be inadequate.

Evaluation of Existing Interchanges

The evaluation of an existing interchange is illustrated by the Seminary Drive interchange on Interstate 35W in Fort Worth, Tex. This interchange will be greatly affected by the future construction of a shopping center near the interchange, as shown in Fig. 23.

The Seminary Drive interchange is presently accommodating existing traffic volumes with little or no congestion. However, the shopping center is expected to create large U-turn movements at this interchange in the future. This is shown by the predicted volumes in Figure 29. Because there are no U-turn lanes provided at this interchange, the large U-turn movements will create congestion.

A capacity analysis for this interchange is shown in Figure 30. The modifications (indicated by this analysis) should provide the required capacity to accommodate the future traffic demand.

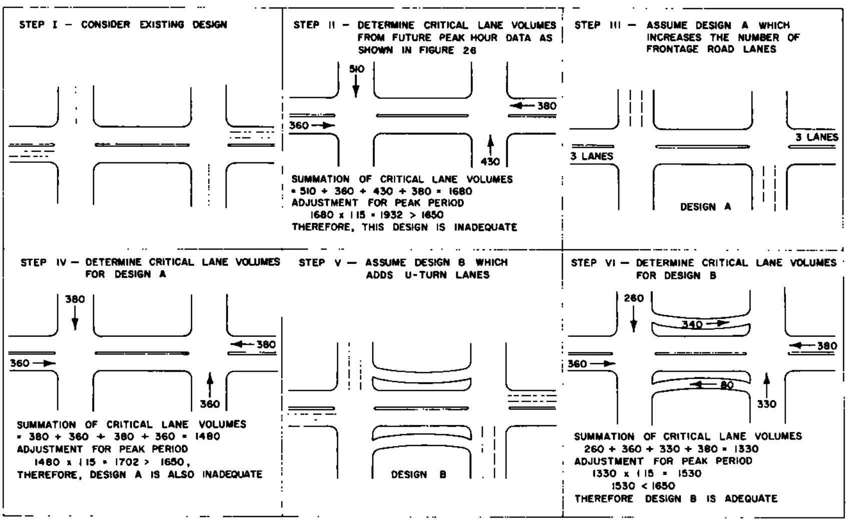


Figure 30. Seminary Drive interchange, Interstate 35W, Fort Worth, Tex.

SUMMARY

As a result of research work conducted by the Texas Transportation Institute on conventional-type diamond interchanges, it is concluded that this type of interchange has many efficient applications on freeway systems in urban areas. With adequate design and proper signalization, the diamond interchange is capable of providing a high degree of efficiency in the interchanging of major arterial and freeway traffic.

ACKNOWLEDGMENTS

Grateful acknowledgement is expressed to members of the Traffic Engineering Department of Fort Worth and the Fort Worth District of the Texas Highway Department for invaluable assistance rendered during the field studies of this project. Appreciation is also expressed to members of the staff of the Texas Transportation Institute who assisted in the collection and analysis of the field data.

REFERENCES

1. Capelle, D. G., and Pinnell, C., "Capacity Study of Signalized Diamond Interchanges." HRB Bull. 291, 1-25 (1961).
2. Drew, D. R., "An Analysis of Peak Traffic Demand at Signalized Urban Intersections." Thesis, A. & M. College of Texas (1961).
3. Leisch, J. E., "Design Geometrics for Diamond Interchanges." Proc., Inst. of Traffic Engineers (1960).
4. LeRoy, H. M., and Clinton, J. W., "Signalization of Diamond Interchanges." Proc. Inst. of Traffic Engineers (1960).
5. Moskowitz, K., "Signalizing a Diamond Interchange." Proc. Northwest Traffic Engineering Conf. (July 1959).

Some Fundamental Relationships of Traffic Flow on a Freeway

DONALD P. RYAN, and S.M. BREUNING, Michigan State University

• FUNDAMENTAL CHARACTERISTICS of traffic flow — especially the relationships among speed, volume, and density — have been given increasing attention in recent years. One reason for the current interest in these characteristics is the hope that by studying their effects on the capacity of a highway, a way may be found to alleviate or eliminate congestion on highways, especially under maximum volume conditions.

The fundamental characteristics are dependent on the geometric design of a roadway and on the traffic flow. This dependence has been expressed in the form of a "Four Friction Concept" (7, 15), in which three frictions (intersectional, medial, and marginal) can be minimized through geometric design, while the fourth (internal-friction) is dependent on the traffic flow. If it can be assumed that the geometric design of a modern expressway controls intersectional, medial, and marginal frictions, the study of the traffic flow characteristics on such a freeway will point up the effects of internal friction only.

PURPOSE

It was the purpose of this work to study the relationships among speed, volume, and density on an expressway to obtain known or new characteristics of traffic flow at high volumes. The reason for such research is the need for specific indications of impending traffic congestion which can then be used in control processes intended to maintain maximum traffic flow.

Because traffic flow in the area of maximum volume changes from stable to unstable, and because the characteristics of this change are not well known, the primary emphasis of this work had to be extended to defining this area of change and to separating data into the ranges of critical and noncritical flow.

CRITICAL VS NONCRITICAL FLOW

It has been known for a long time that traffic flow is unstable in the region beyond maximum volume. Any volume increase beyond a certain value causes complete congestion. The precise characteristics of the point where unstable flow begins are not known, although this topic has been the subject of considerable research. One might compare the situation in traffic flow with the transition between laminar and turbulent flow in hydraulics, although in the case of fluid flow the transition point does not represent the point of maximum flow capacity. Another possible comparison is that with the yield point of steel. But the maximum strength of steel, unlike traffic flow capacity, is maintained over a wide range of strain before complete failure of the material occurs. In traffic flow, failure occurs suddenly and no definite warning has as yet been found.

Critical Flow

The unstable region of traffic flow is referred to here as "critical flow." Characteristics of this type of flow are the appearance of congestion, the drastic reduction in traffic volume and speed, and the unpredictability of data variations, exemplified by the scatter of measurements obtained in this type of flow. The motorist traveling in this flow is much more restricted by the action of vehicles around him than by his own desires. This can be concluded from the greater fluctuations in data in short successive time intervals. It is also easily verified qualitatively by each driver when caught in such flow. Specific quantitative comparisons will be given in later sections.

The study of critical flow is not of primary interest, except in determining how to avoid it. Once critical flow has been established, noncritical flow can only return when the traffic volume input is reduced sufficiently so that the speed can increase to free flow speed and stable flow.

An important fact about critical flow is that its effect spreads so rapidly and becomes so dominant. If only one point on a long, heavily traveled facility converts to critical flow, the lower capacity at this point will quickly create an overload that will spread backward fast and far. The point of conversion from noncritical to critical flow travels opposite to the direction of vehicular travel quickly, and may even spread into heavily loaded feeder or cross-routes. Thus critical flow can be quite "contagious."

On the other hand, the conversion from critical to noncritical flow requires that all points along the facility can and do convert. If only one location retains critical flow, no points behind this one can change to noncritical flow.

Noncritical Flow

Noncritical traffic flow is flow in the stable region. This flow is the primary subject of this discussion because it represents the desirable traffic flow characteristics from the point of capacity and economy.

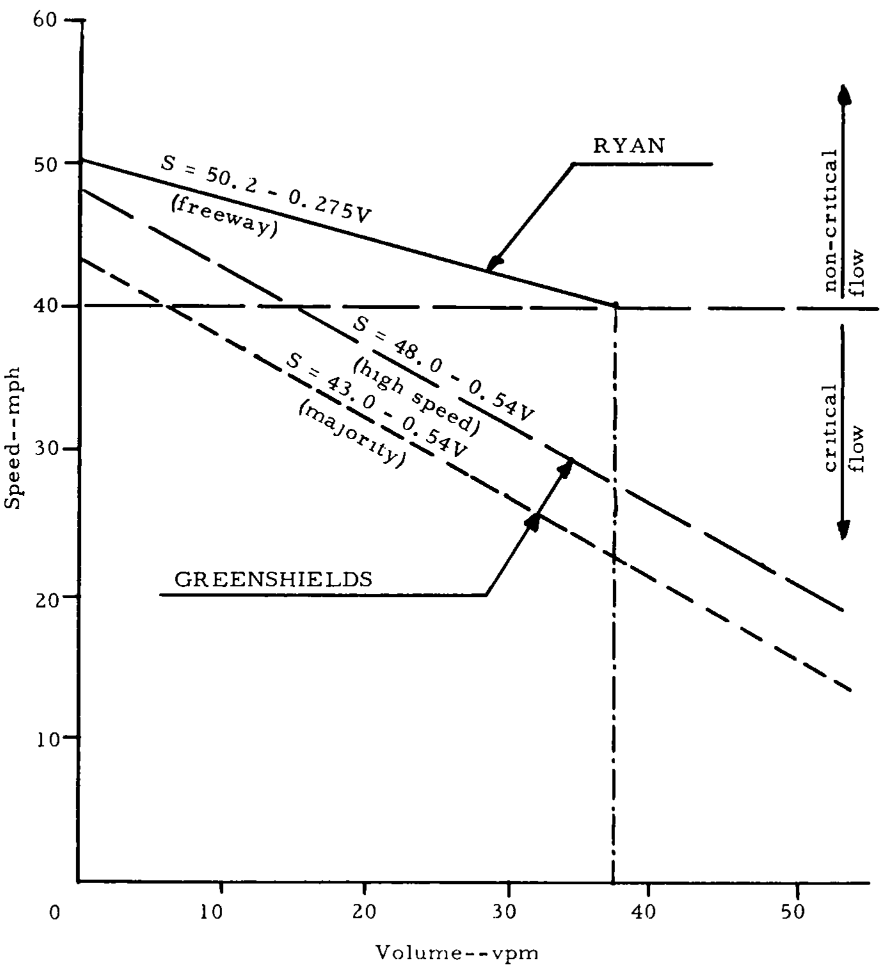


Figure 1. A comparison of speed-volume relations.

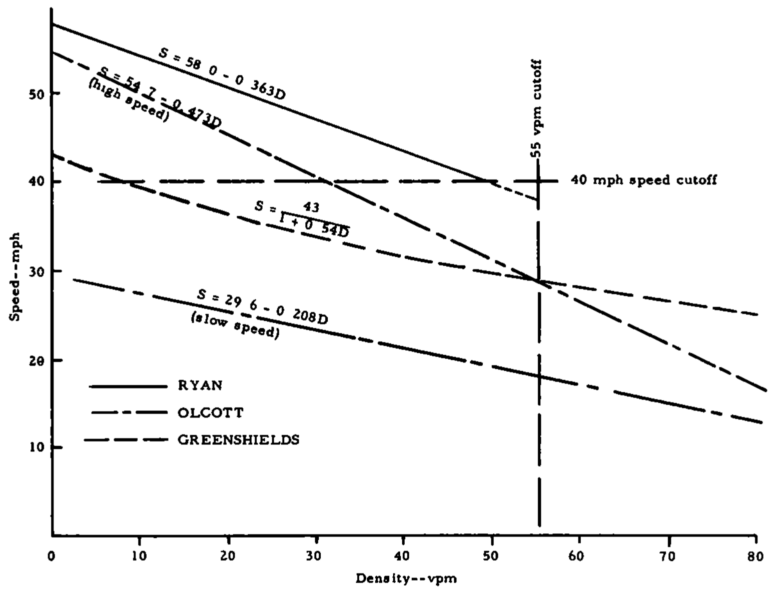


Figure 2. A comparison of speed-density relations.

Linearity of relations of the fundamental characteristics has been suggested in various and previous studies, but not necessarily for all relations. Greenshields' early study in 1934 (6) proposed a linear relation for the speed-volume relation, but slight, negligible, curvature for the speed-density relation. Linearity of volume-density relation is not generally suggested, but May and Wagner (15) present graphs in which good linearity seems to exist in the range of noncritical flow, although curvilinear regressions are proposed for the entire flow range.

Linearity of the basic relations need not extend to very low volumes, because the flow in that area is of no significance in the study of maximum flow. Furthermore, the few observations possible would not be statistically significant.

Linearity of the relations in the region of noncritical flow was assumed in this study. Regression equations were computed for the data and were tested for linearity by the F-linearity test. If linearity was rejected, as it was in all but one case for the entire peak period, a cutoff was chosen and the reduced sample again tested for linearity. In this way the limit of linearity of relationships was determined and found to agree closely with the selected boundary between noncritical and critical flow.

Another characteristic of noncritical flow is the relatively small change in measurements of the fundamental characteristics from one observation period to the next. In critical flow these changes are considerably larger. It appears that these average changes might be a measure of internal friction and be very helpful in defining the boundary between noncritical and critical flow.

The significance of change of average measurements is not evident, although a comparison to the effects of turbulence in fluid flow would seem reasonable. More study of this subject is indicated.

METHOD OF STUDY

Description of Test Area

The study site was located on the three in-bound lanes of the Edsel Ford Expressway at the Lonyo Street overcrossing in Detroit. The expressway is a six-lane, divided, grade-separated, and depressed freeway, located in an urban area approximately 2 mi west of the CBD. It is about 15 mi long and is an integral part of Interstate 94 — Detroit to Chicago.

There are two on-ramps in the vicinity of the study area. One is on the west side of the bridge about ¼ mi away. The other ramp is on the east side of the bridge some 600 ft away. In general, this section of freeway is typical of modern freeway design.

Instrumentation

The data-gathering equipment consisted of electronic radar vehicle and speed detectors, some mounted over and some alongside the roadway. The detection equipment was not discernible to the passing motorist.

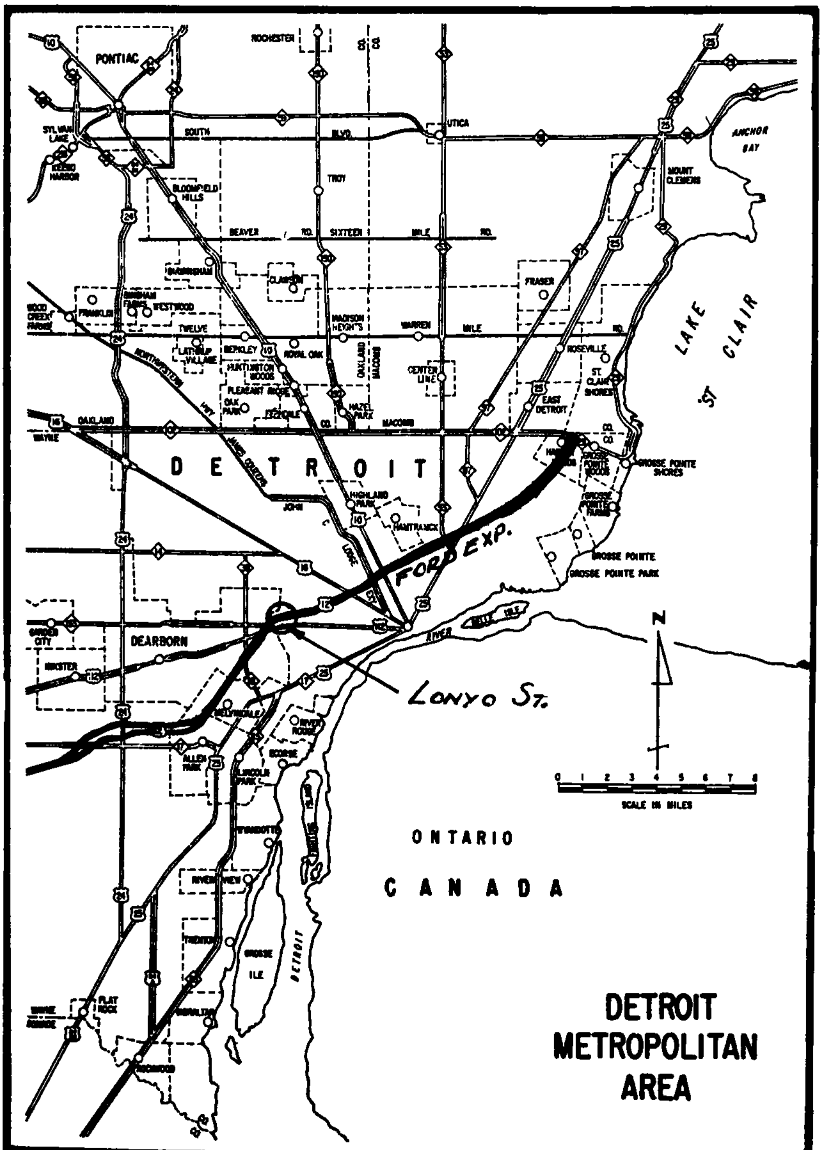


Figure 3. Map of Detroit area showing study site location.

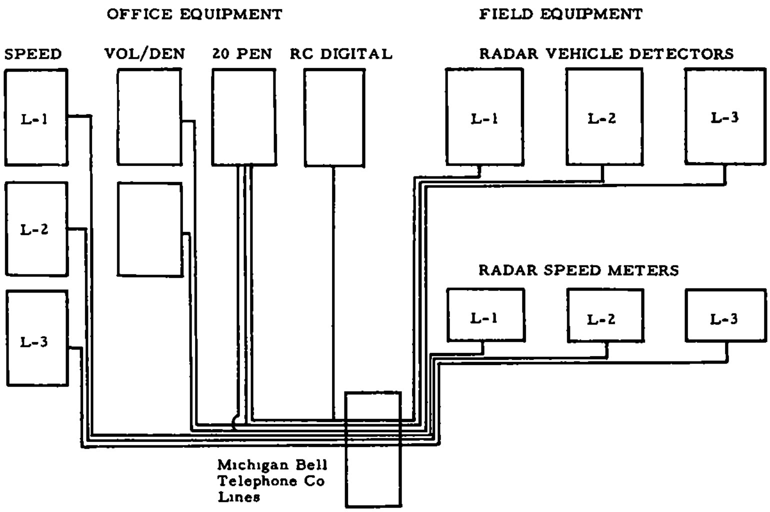


Figure 4. Instrumentation of test site.

Each vehicle that passed the field study point was detected by model RD-1A Electro-Matic overhead radar vehicle detectors and by modified models of the S-2A Electro-Matic radar speed meter. These units were connected remotely by telephone lines to an Esterline-Angus 20-pen recorder, a 1-min model of the RG Streeter-Amet printing counter, and to Esterline-Angus graphical speed and volume-density recorders.

The 20-pen recorder was used to plot volumes and time headways of vehicles by lane. The digital recorder was used as a check on the 20-pen volume record of the various lanes. The graphical speed recorders kept a plot of speeds of all vehicles according to time of occurrence, allowing for correlation of speed and volume data. The volume-density recorders produced a plot of the daily fluctuations in traffic. These could be compared for different days to see if daily characteristics were similar. All equipment was kept under constant surveillance and was checked periodically to make sure that all units were functioning properly.

Data Collected

Data for this study were taken during periods of peak traffic flow in order to have volumes sufficiently large to cause a decrease in speed and an increase in density so that a back-up of traffic would occur. Because the test siste was on the in-bound traffic lanes, data were collected in the morning from 6:45 a.m. to 8:00 a.m. This time period included peak flow and the time immediately preceding and following the peak.

All the data for this study were gathered in 1-min intervals. In the past, samples of 5-min or more intervals have generally been used and it is believed that much valuable information has therefore been leveled out and lost through averaging of data extremes over a 5-min or longer interval.

The majority of vehicles on this road were assumed to be driven by people working in or around Detroit, who were familiar with the highway, the peak flow conditions, and with the posted speed limits of 55-mph maximum and 40-mph minimum when conditions permit.

METHOD OF ANALYSIS

The analysis in this study was made on the basis of finding linear relationships in the data in the region of noncritical flow and of determining the dividing line between critical and noncritical flow. The main steps in the analysis consist of calculating several statistics, making chronological plots, and correlating the results into a set of logical deductions.

To facilitate the calculation of statistics a correlation table was used which shows numerically as well as graphically the two-way distributions of speed-volume, speed-density, or volume-density. The values in the table were used in the more refined statistical calculations. Of these calculations, the F-linearity test was used to determine whether acceptable linearity existed in a region, and in then defining noncritical flow as that region in which acceptable linearity existed.

All relationships determined were hypothesized to be linear and were tested for linearity by the F-linearity test. Calculations of the F-value had to be less than the 95 percent level values to be accepted; if larger than the 95 percent level, they were rejected. A rejection, however, is just as useful, for it helps to explain the relation as much as acceptable F-values. If the F-linearity test is rejected, a new region of noncritical flow is defined and a recalculation made. The defining of a new region is essentially the selecting of a new cutoff line between critical and noncritical flow. This process is repeated until an acceptable F-test is made.

Another statistic determined was the correlation coefficient. This statistic is used to determine whether the regression coefficient is significantly non-zero. The null hypothesis is made and tested, that is, it is believed that the regression coefficient is zero. Values of r larger than the 95 percent level values reject the hypothesis. In general, the closer the value of r to 1 the greater the significance of non-zero and of correlation of Y and X.

The standard error of estimate ($S_{y/x}$) was used to determine the relative scatter of the observed points about the regression line in the ordinate Y direction. A large standard error of estimate means that data are more widely scattered, whereas a small $S_{y/x}$ indicates the data to be relatively close to the regression line. Relating this to traffic flow then, a small $S_{y/x}$ would be more desirable as it would indicate relatively smooth-flowing traffic. A large $S_{y/x}$ would therefore indicate a wide scattering of data, which would likely be the result of congestion or interrupted flow. If the standard error of estimate is squared, the resulting value is the variance of Y left unexplained by the regression of Y on X.

To help visualize and support the statistical analysis a chronological plot of data was made. This is a successive plot of data chronologically from the beginning of the study period to the end. A plot is made of X, Y points and these points are connective successively with an arrow indicating the progression of time. Cutoff lines are also shown which help point out the differences between data in the range of critical and noncritical flow.

Analysis of Speed-Volume Relationship

In the speed-volume analysis the cutoff line was determined at approximately 40 mph, dividing flow into critical flow below and noncritical flow above the line. Figure 5 shows from the chronological plot of lane-1 data, that the character of data differs considerably for critical and noncritical flow. This difference is further demonstrated by comparing the range of speeds and the change of average speed between 1-min intervals as shown in Figure 6. Noncritical flow has a speed range of 6 mph, an average speed increase of 1.94 mph, and an average speed decrease of 1.77 mph. Critical flow has a speed range of 31 mph, an average speed increase of 3.56 mph, and an average speed decrease of 4.43 mph. These differences in flows reflect the increase in internal friction in critical flow.

When the fluctuations in speed are less pronounced, as shown in Figure 7, the apparent effect of congestion is not present and the flow soon returns into the region of noncritical flow.

Each of the lane regressions (see Table 1) were tested for linearity of relationship. The results indicated that at the 95 percent level there was no basis for rejecting the hypothesis that the speed-volume relationship was linear. The results further indicate that there is some justification for saying that speed and volume are linearly related in the area of noncritical flow but not in the area of critical flow. A calculation of the regression slope limits indicate generally that the slopes of all lanes lie within these limits, with the exception of lane 2, on Friday, which had a positive slope.

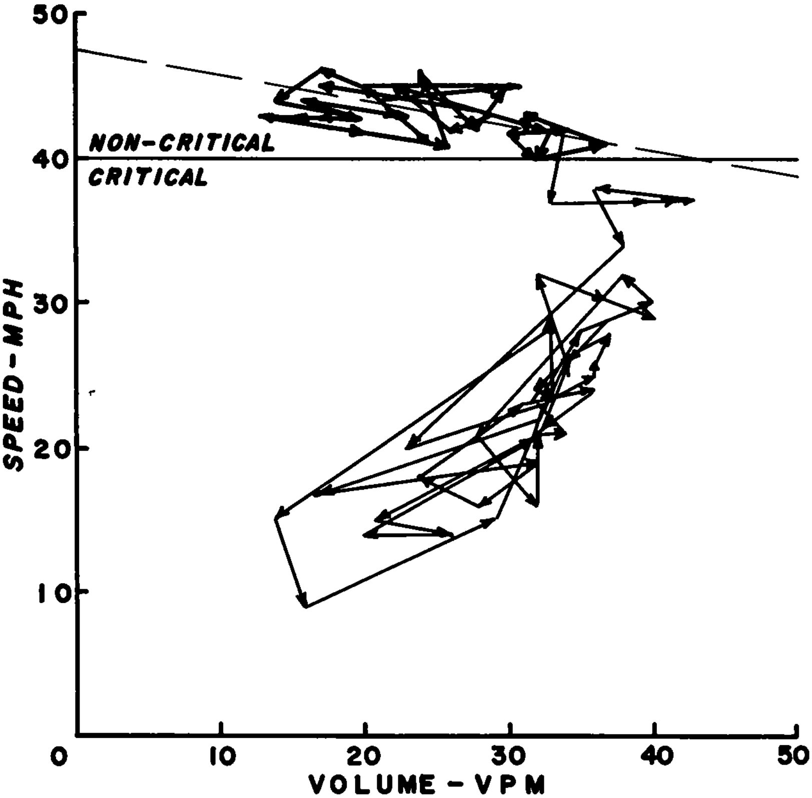


Figure 5, Chronological plot of data for lane 1, February 13, 1958.

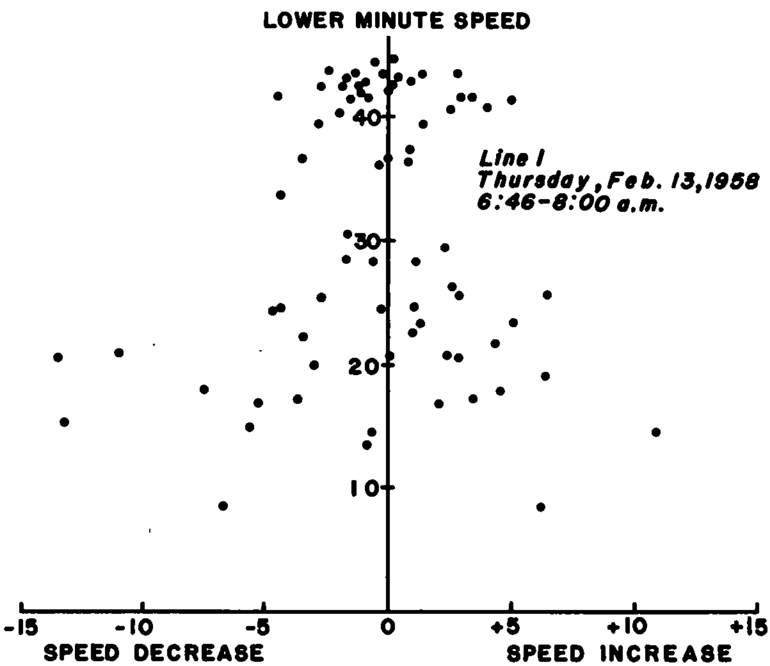


Figure 6. Change of minute average speed vs lower minute speed.

TABLE 1

REGRESSION EQUATIONS FOR FUNDAMENTAL RELATIONS

Lane		Speed Volume	Speed Density	Volume Density
1	Thur.	S = -0.179V + 47.6	S = -0.142D + 48.1	V = 0.612D + 3.7
	Fri.	S = -0.166V + 52.7	S = -0.15 D + 53.6	V = 0.695D + 3.1
2	Thur.	S = -0.231V + 54.6	S = -0.271D + 57.2	V = -0.584D + 7.6
	Fri.	S = 0.010V + 41.1	S = -0.310D + 57.8	V = 0.574D + 7.1
3	Thur.	S = -0.273V + 48.6	S = -0.323D + 54.2	V = 0.497D + 3.3
	Fri.	S = -0.231V + 49.2	S = -0.427D + 53.8	V = 0.559D + 3.5
Avg.	Thur.	S = -0.275V + 50.2	S = -0.363D + 58.0	V = 0.472D + 5.7
	Fri.	S = -0.227V + 51.1	S = -0.361D + 57.4	V = 0.459D + 7.0

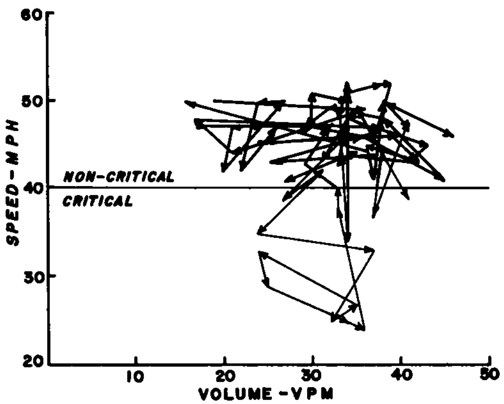


Figure 7. Chronological plot of data for lane 2, March 28, 1958.

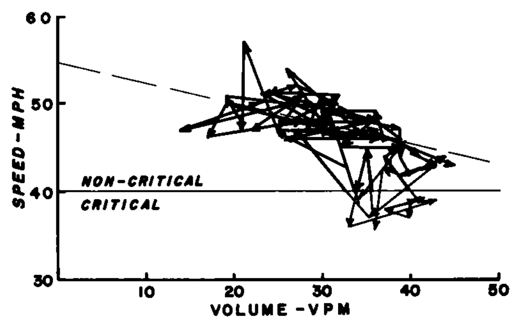


Figure 8. Chronological plot of data for lane 2, March 27, 1958.

The chronological plot of speed-volume data shows that critical flow (congestion) does not always occur immediately following peak flow. There is some delay after peak noncritical flow before congestion occurs, shown in Figures 5 through 9. Furthermore, peak-minute volumes do not necessarily occur in noncritical flow, but can also occur during critical flow when speeds are between 35 and 40 mph for lanes 1 and 2, and 30 to 35 mph for lane 3. Further, Figures 5 through 9 show that the transition to critical flow occurred on lanes 1 and 2 when speeds were less than 45 mph and when volumes were from 30 to 40 vehicles per min. On lane 3 the transition occurred when the volume range was 15 to 25 vehicles per min.

Analysis of Speed-Density Relationships

The speed cutoff line in the speed-density analysis was also 40 mph. However, in this analysis a second cutoff line for density data was determined, indicating critical flow above 55 vehicles per mile (vpm). Figure 10 shows a considerable difference in the character of data between critical and noncritical flow. In addition to the difference in speed range pointed out in the speed-volume analysis, the difference in density range, 36 vs 57 vpm, further substantiates the difference in critical and noncritical flow. Furthermore, the average density changes are 55 percent greater in critical flow than in noncritical flow. These greater changes in density reflect larger changes in headway, which points to increased internal friction and a more likely chance of congestion.

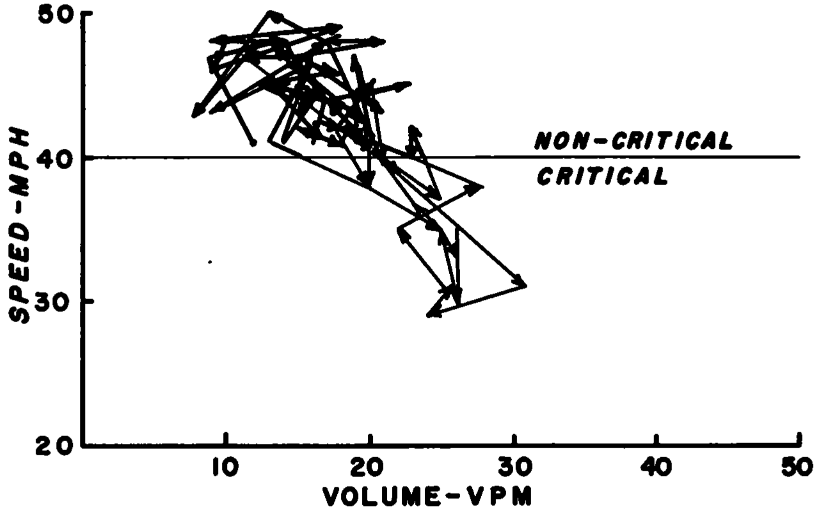


Figure 9. Chronological plot of data for lane 3, March 28, 1958.

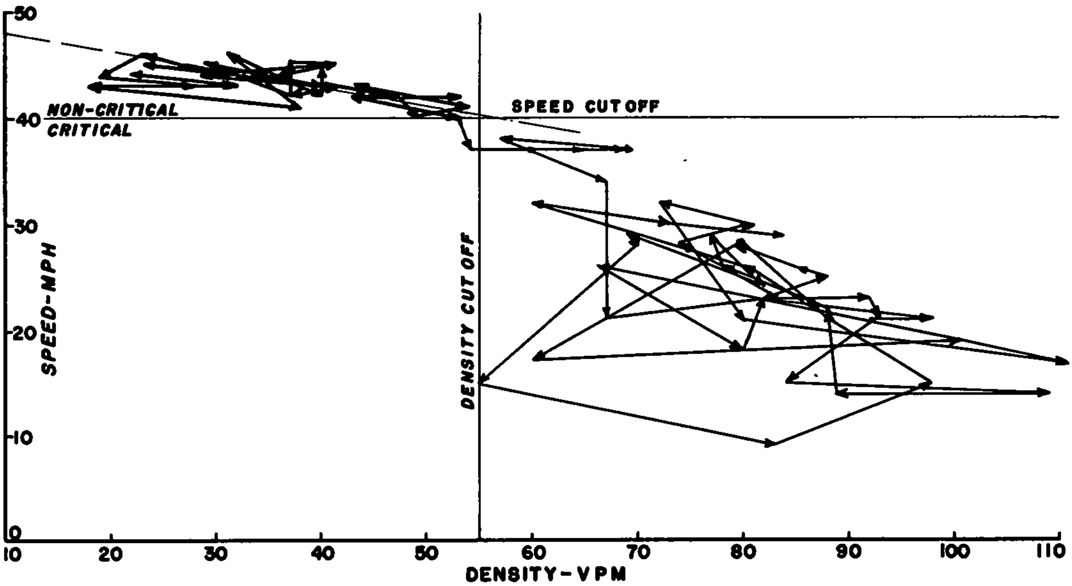


Figure 10. Chronological plot of data for lane 1, February 13, 1958.

Densities ranged from as few as 18 vpm to as many as 111 vpm. This is equivalent to a drop in headway from 293 ft to less than 48 ft. The greatest change though was only from 60 to 100.5 vpm or an increase of 40.5 vpm. This change occurred in 60 sec and decreased headways from 88 to 53 ft. Such density changes are not possible in noncritical flow, where high speeds prevail.

Lane 3 speed-density characteristics are a little different in that the plot of data (Fig. 11) indicates that the cutoff line corresponding to a speed drop below 40 mph would have to be about 30 to 35 vpm. Some of the calculations, such as the regression analysis of volume-density relation, bear this out, although the regression analysis of the speed-density relation does not.

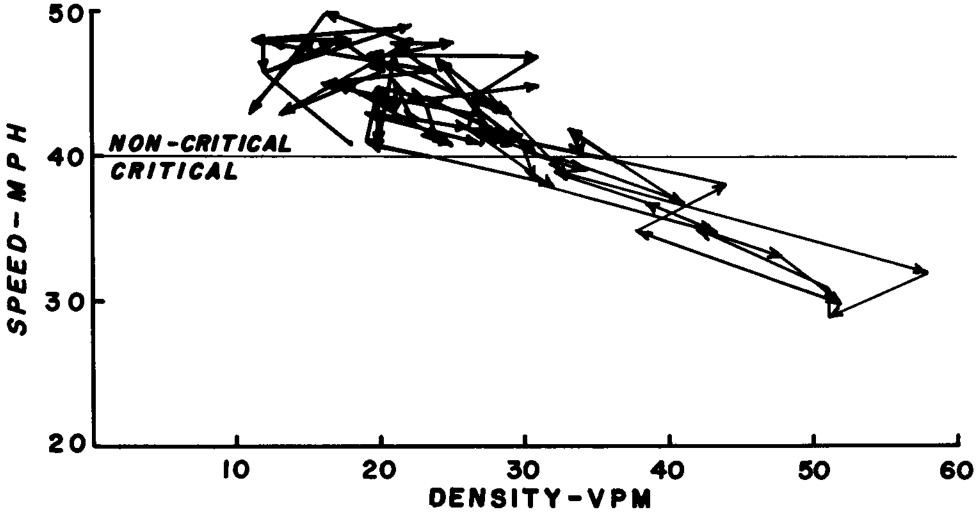


Figure 11. Chronological plot of data for lane 3, March 28, 1958.

Each of the lane regressions were tested for linearity of relationship. The results indicated that at the 95 percent level the speed-density relationship was linear with a cutoff of 55 vpm. Acceptable, although statistically less definite, linearity was determined for some higher cutoffs. A calculation of the regression slope limits indicated generally that the slopes of the speed-density relation for each lane are different from one another.

The chronological plot of speed-density data shows that maximum density does not occur with minimum speed.

Analysis of Volume-Density Relationships

The cutoff line for this analysis was a density of 55 vpm for lanes 1 and 2, and 45 vpm for lane 3. The chronological plot of data for each lane (Fig. 12) shows that the character of data differs by varying amounts in critical flow as compared to noncritical flow. Lane 1, which portrays congestion, supports the premise of difference in flows both graphically and numerically. This figure also shows the relative linearity of volume vs density up to 55 vpm.

The range in volume is not too different in critical flow as compared to noncritical flow, nor are the average volume increases and decreases widely different. The only thing pointed out here is that average decreases are larger than average increases in critical flow, whereas average increases are larger than average decreases in noncritical flow. This may be caused by the greater internal friction in congestion. Noncritical flow may have the ability to absorb these changes, whereas in critical flow there may not be enough headway to absorb an increase in density.

There are generally close similarities of operating characteristics of lanes 1 and 2, but not with lane 3. Volumes, speeds, and densities are less for lane 3, which is reflected in the lower density cutoff of 45 vpm. Inspection of the plotted data indicates a gentle curve primarily above 30 vpm. It is felt that a higher degree of linearity could be attained if the density were cut off between 30 and 35 vpm on lane 3.

The regression analysis indicated in that five of the six analyses the use of all data would not give an acceptable F-linearity test. Lanes 1 and 2 had cutoffs of 65 to 75 vpm on preliminary calculations. It was believed, however, that a higher degree of linearity could be attained with lower cutoff densities, and recalculations of data proved this in all respects. First, F-values were 0.874 for the 55 vpm cutoff and 1.21 for the 65 vpm cutoff. Second, the coefficient of correlations were considerably different, 0.956 as compared to 0.641. Third, the standard error of estimate is smaller for 55

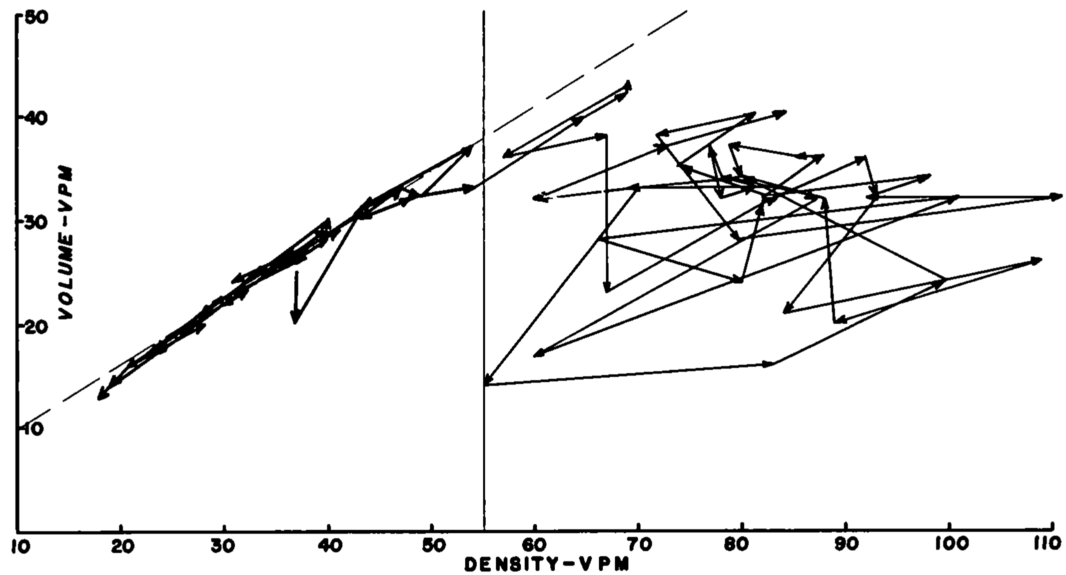


Figure 12. Chronological plot of data for lane 1, February 13, 1958.

vpm cutoff than for the higher cutoffs. These results indicate that the 55 vpm cutoff is a better estimate of the line between noncritical and critical flows.

SUMMARY AND CONCLUSIONS

A detailed analysis of traffic flow on an expressway during peak flow periods has been presented. Traffic flow was separated into noncritical and critical flow, and the analysis limited on the determination of the boundary between the two types of flows and on the characteristics of noncritical flow.

Noncritical flow is studied alone to obtain reliable data on that type of flow. 40 mph was found to be the boundary between critical and noncritical flow for the speed-volume and the speed-density relationships. A density cutoff was also chosen for the volume-density relationship, although the cutoff point is not so clearly evident for density as it seemed to be for speed.

It was found that all three relations among speed, volume, and density are linear within noncritical flow in range of observations bounded on one side by low traffic flow and on the other by the boundary with critical flow. This linearity is lost at the boundary where flow becomes unstable. Therefore, it is proposed that the end of linearity of flow marks the boundary between noncritical and critical flow.

Another significant set of data is the average change of characteristic measurements between successive 1-min observation intervals. It was found that in noncritical flow the average change in general is much smaller than in critical flow. This result, for which rational explanations may be postulated, might lead to another definition of the boundary between noncritical and critical flow. This study did not find whether this change occurs sufficiently before actual congestion to serve as an advance warning tool.

The three relationships studied showed considerable difference in correlation coefficients. The coefficient of correlation for speed is the lowest, that for density the highest. This might point to a comparative independence of speed from the other two measurements. But because density was determined from accumulations of volume and speed measurements over time, and, therefore, is not independent, this conclusion must be taken with reservation until proved by direct density measurements.

REFERENCES

1. Arkin, H., and Colton, R.R., "Tables for Statisticians." Barnes and Noble (1960).
2. Barnett, J., "Operation of Urban Expressways." Jour. Highway Div., ASCE, 83: No. HW4 (Sept. 1957).
3. Crow, E.L., Davis, F.A., and Maxfield, M W., "Statistics Manual." Dover (1960).
4. Edie, L.C., and Foote, R.S., "Traffic Flow in Tunnels." HRB Proc., 37: 334-344 (1958).
5. Forbes, T.W., "Speed, Headway, and Volume Relationships on a Freeway." Inst. of Traffic Eng., pp. 103-126 (1951).
6. Greenshields, B.D., and Weida, F.M., "Statistics with Application to Highway Traffic Analyses." Eno Foundation for Highway Traffic Control (1952).
7. Halsey, M., "Traffic Accidents and Congestion." (1941).
8. "Highway Capacity Manual." U.S. Dept. of Commerce, Bureau of Public Roads (1950).
9. Keese, C.J., and Schleider, R.H., "The Correlation of Design and Operational Characteristics of Expressways in Texas." HRB Bull. 170, 1-23 (1958).
10. Lighthill, M.J., and Whitman, G.B., "Kinematic Waves. Part II: A Theory of Traffic Flow on Long Crowded Roads." Proc., Royal Soc. of London, Series A, 229: 317-345 (1955).
11. Moroney, M.J., "Facts from Figures." Penguin (1958).
12. Moskowitz, K., "Research on Operating Characteristics of Freeways." Inst. of Traffic Engineers (1956).
13. Olcott, E.S., "The Influence of Vehicular Speed and Spacing on Tunnel Capacity." Operations Research for Management, John Hopkins Press, 2: 57-81 (1956).
14. Walker, H.M., and Lev, J., "Statistical Inference." Holt (1953).
15. May, A.D., Jr., and Wagner, F.A., Jr., "A Study of Fundamental Characteristics of Traffic Flow." Michigan State Univ. (1960).
16. Underwood, R.T., "Speed, Volume, and Density Relationships." Quality and Theory of Traffic Flow, A Symposium. Bureau of Highway Traffic, Yale Univ. (1961).

A System for the Collection and Processing of Traffic Flow Data by Machine Methods

J. H. AUER, Jr., Principal Engineer, Research Department, General Railway Signal Company, Rochester, N. Y.

• THE COLLECTION and processing of traffic flow data frequently involves many man-hours devoted to the task of correlation of measurements taken simultaneously at different locations and working the original data into a form where it can be handled by digital computers. This paper describes a new data collection system that has proven to be very practical for conducting accurate traffic surveys and traffic flow research studies. There are two significant features inherent in the system:

1. Traffic data is obtained and processed entirely by machines, thereby eliminating human errors. Because the processed data is on punched cards, digital computers may be readily applied to assist in the task of data analysis.
2. Large amounts of traffic data may be obtained and processed at relatively low expense with a minimum of manpower.

The basic type of traffic data used in this system consists of vehicle presence and vehicle classification information obtained from ultrasonic vehicle classification detectors. This information is permanently recorded on magnetic tape in the field. These tapes are then processed in a computing center using a special tape readout unit to punch IBM cards directly. A number of computer programs have been written that enable a digital computer to accept these input cards and compute minute averages of the following traffic parameters: passenger car volume, commercial vehicle volume, total volume, lane occupancy, speed, time headway, distance headway, time spacing, and distance spacing.

LANE OCCUPANCY

Lane occupancy is a relatively new traffic parameter obtainable through the use of vehicle presence detectors as opposed to conventional treadles or radar units as the source of traffic information. The nature of lane occupancy and its relationship to density (vehicles per mile) deserve some explanation. If one were to draw a line down the center of a traffic lane, the lane occupancy at any instant could be defined as the percentage of this line covered by vehicles. Vehicles packed bumper to bumper would produce a lane occupancy of 100 percent. Vehicles spaced by three vehicle lengths would produce a lane occupancy of 25 percent.

Although lane occupancy is quite similar in its behavior to density, there are some significant differences. Traffic consisting of a mixture of passenger cars and commercial vehicles can create a wide departure in the relative behavior of lane occupancy and density. This may be illustrated by the following exaggerated example. If 1 mi of lane contained 30 automobiles, density would be 30 vehicles per mi, and a typical lane occupancy reading would be approximately 10 percent. If this same mile of lane were occupied by 30 trucks, density would still be 30 vehicles per mi; however, lane occupancy would read from 20 to 30 percent depending on the length of trucks encountered. It is obvious that in the latter case the lane would be substantially more congested, although the density figures would be identical.

In actual practice it has been found that the congestion in a traffic lane may be reliably ascertained by an observation of the magnitude of lane occupancy because this figure automatically takes into account the varying lengths of vehicles encountered in typical mixed traffic.

VEHICLE CLASSIFICATION DETECTOR

The GRS Type SVDS-CD ultrasonic vehicle classification detector is used as the source of traffic information in the system. The sensing unit of this detector is mounted approximately 15 ft over the center of a traffic lane with its axis perpendicular to the surface of the roadway. The detection zone is in the form of a sharp vertical cone directly beneath the sensing unit. The electronic transceiver for the detector is located in a weatherproof housing and interconnecting cables run between the transceiver and sensing unit. The transceiver is shown in Figure 1.

The transceiver contains two relays that provide information as follows:

1. The first relay is called the all-vehicle (AV) relay. This relay provides vehicle presence information, closing its contact when the front of a vehicle first appears within the detection zone and opening its contact when the end of the vehicle leaves the detection zone. Continued presence of a vehicle within the detection zone maintains the relay contact in a closed position. Two types of information are obtained from this relay; namely, all-vehicle count and lane occupancy.
2. The second relay is called the high-vehicle (HV) relay. This relay closes its contact whenever a vehicle whose top surfaces are above a preset height is within the detection zone. The transceiver is normally adjusted so that vehicles whose top surfaces are higher than 68 in. cause this relay to close its contact. Actuation of this relay therefore indicates the presence of a high (commercial) vehicle in the detection zone. Commercial vehicle count is obtained from this relay.

SYSTEM BLOCK DIAGRAM

A block diagram of the system used to secure the typical traffic data is shown in Figure 2. Eight vehicle classification detectors were temporarily installed on bridges over a typical expressway at stated intervals. The sensing unit in each case was installed over the same lane. The two output relays of each detector were connected by low-grade telephone pairs to a central point where the encoding and tape recording apparatus was located.

The encoders consisted of two GRS Type TME-8 telemetering encoders, each encoder handling eight input channels. The function of the encoders in the system was to convert DC contact closure information into a tone format that could be recorded on magnetic tape.

The outputs of the two encoders were connected to a standard two-channel tape recorder. The encoding and tape recording apparatus is shown in Figure 3. Standard $\frac{1}{4}$-in. magnetic tape was used, running at a speed of 7 $\frac{1}{2}$ in. per sec. With 4,800-ft reels of tape, approximately 2 hr of traffic data could be collected on a single tape. During this period of time, on some occasions over 3,600 vehicles passed by each of the eight detectors providing for a 28,000-vehicle sample.

The tapes were then taken to the computing center and played back on a second tape recording machine used as a tape reader. The output of the tape reader was connected to a GRS Type ES-100-TME-8 decoder whose output, by suitable channel selection, recreated the original sixteen channels of HV and LV relay contact closure information.

The decoder was connected to a volume and lane occupancy computer and counter, which served as summation and punch control equipment. This equipment was connected to an IBM 514 card punch machine which automatically punched an IBM card at the end of each minute with the time and date, together with minute totals of vehicle count, commercial vehicle count, and lane occupancy in digital form from all eight vehicle classification detectors. Each card, therefore, contained traffic data taken simultaneously over the same minute interval at all eight detector locations.

The punched cards were then fed to an IBM 650 computer which computed the desired traffic parameters from the input cards and furnished a set of output cards containing time and date together with minute averages of the parameters at the eight detector locations. The computer output cards were then fed to an IBM 407 tabulator which provided tabulations of the parameters as a function of time. The tape reader, decoder, volume, and lane occupancy computer, and counter together with the IBM 514 card punch machine are shown in Figure 4.

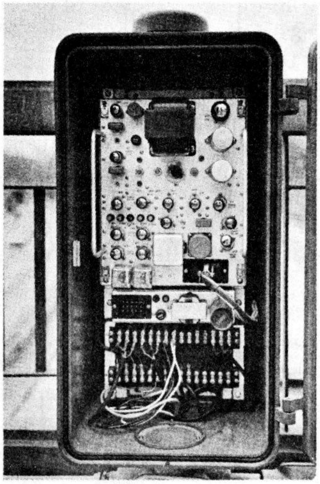


Figure 1. Classification detector, Type SVDS-CD.

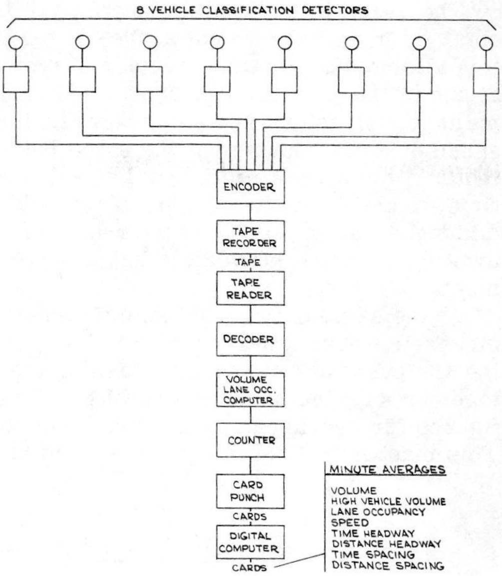


Figure 2.

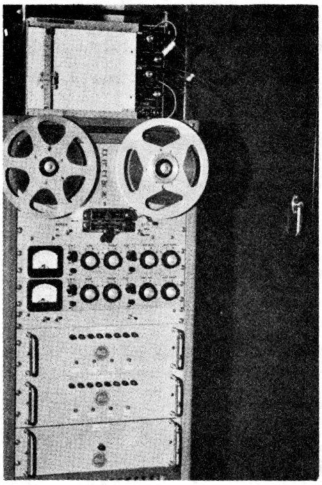


Figure 3. Encoding and recording equipment.

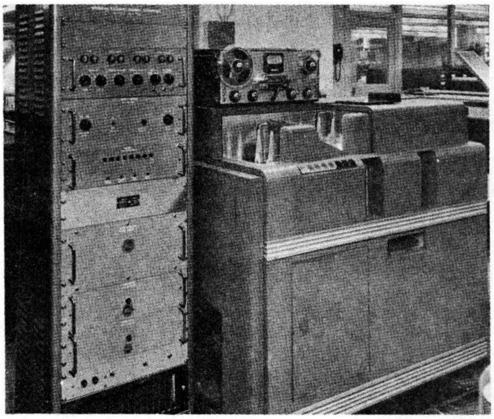


Figure 4. Tape-to-card information, transfer equipment.

DATA PROCESSING

Figure 5 shows the traffic parameters obtained directly from each vehicle classification detector as well as those computed by the digital computer. Data on the original tapes consisted of nothing more than vehicle passage time intervals and whether the vehicle was low (passenger car) or high (commercial). By processing the original tapes through the tape reader, decoder, volume, and lane occupancy computer, counter, and IBM 514 card punch machine; minute averages of vehicle volume (V), high-vehicle volume (N_h), and lane occupancy (Occ) were punched in digital form together with time and date on computer input cards. Vehicle volume and high-vehicle volume were obtained by counting the number of actuations of the all-vehicle (AV) and high-vehicle (HV) relays respectively during each 1-min interval. Lane occupancy in digital form was obtained by gating a constant frequency signal with the AV relay, and counting the number of cycles that passed through the gate within each 1-min interval.

The average low-vehicle length (ALVL) and average high-vehicle length (AHVL) were obtained in an independent survey at the time the original data was recorded. To secure this information, a large number of typical vehicles on the expressway were measured at one of the detector locations. The individual speed of each vehicle was obtained simultaneously with vehicle passage time using a GRS Type SVDS-MD ultrasonic doppler speed detector mounted at the same location as the ultrasonic vehicle classification detector. Knowing the individual speed of each vehicle and simultaneously its passage time, together with the type of vehicle (i.e., passenger car or commercial), the individual length of each vehicle was computed. The average passenger car length and average commercial vehicle length were then computed from the individual measurements.

At the same time actual vehicle length measurements were being made by automatic means, several observers at the location of the detectors noted the year and make of the various types of vehicles passing the detectors. On the basis of this information, together with the manufacturers' published lengths of vehicles, an independent set of figures for average passenger car and commercial vehicle lengths could be obtained. This independent set of figures was in close agreement with the actual measured lengths

OBTAINED DIRECTLY FROM CLASSIFICATION DETECTOR

V - VOLUME	VEHICLES PER MINUTE
NH - HIGH VEHICLES	VEHICLES PER MINUTE
OCC - LANE OCCUPANCY	PCTG. OF TIME OCCUPIED

MEASURED

ALVL - AVERAGE LOW VEHICLE LENGTH FEET

AHVL - AVERAGE HIGH VEHICLE LENGTH FEET

COMPUTED AVERAGES

NL - LOW VEHICLES	VEHICLES PER MINUTE
SP - SPEED	MILES PER HOUR
HT - TIME HEADWAY	SECONDS
ST - TIME SPACING	SECONDS
HD - DISTANCE HEADWAY	FEET
SD - DISTANCE SPACING	FEET

$$N_L = V - N_H$$

$$S_P = \frac{ALVL \times N_L + AHVL \times N_H}{60 \times Occ.} \times \frac{60}{88}$$

$$H_T = \frac{60}{V}$$

$$S_T = (1 - Occ)\, H_T$$

$$H_D = \frac{88}{60} \times S_P \times H_T$$

$$S_D = \frac{88}{60} \times S_P \times S_T$$

Figure 5. Traffic parameters.

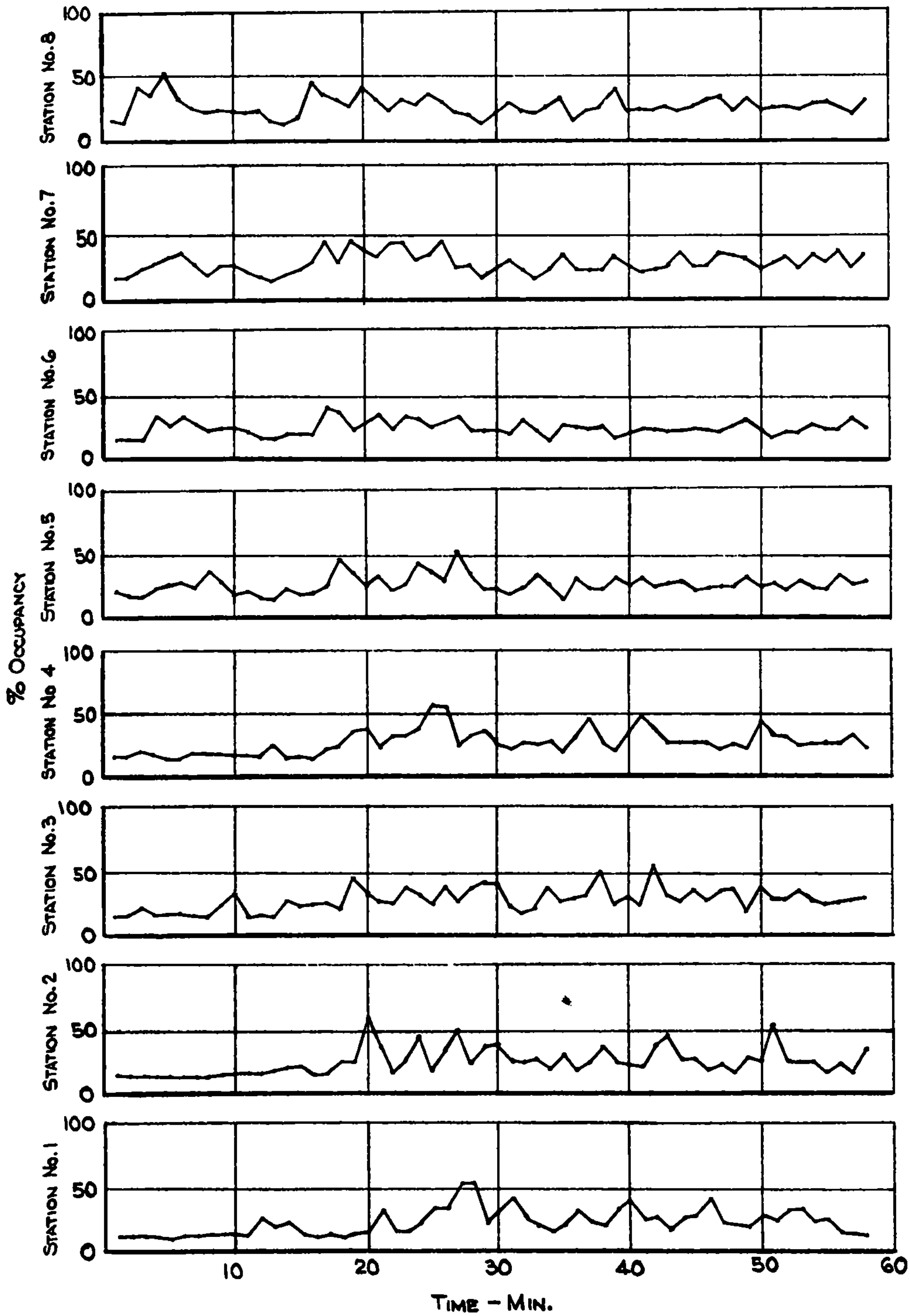


Figure 6. Lane occupancy vs time.

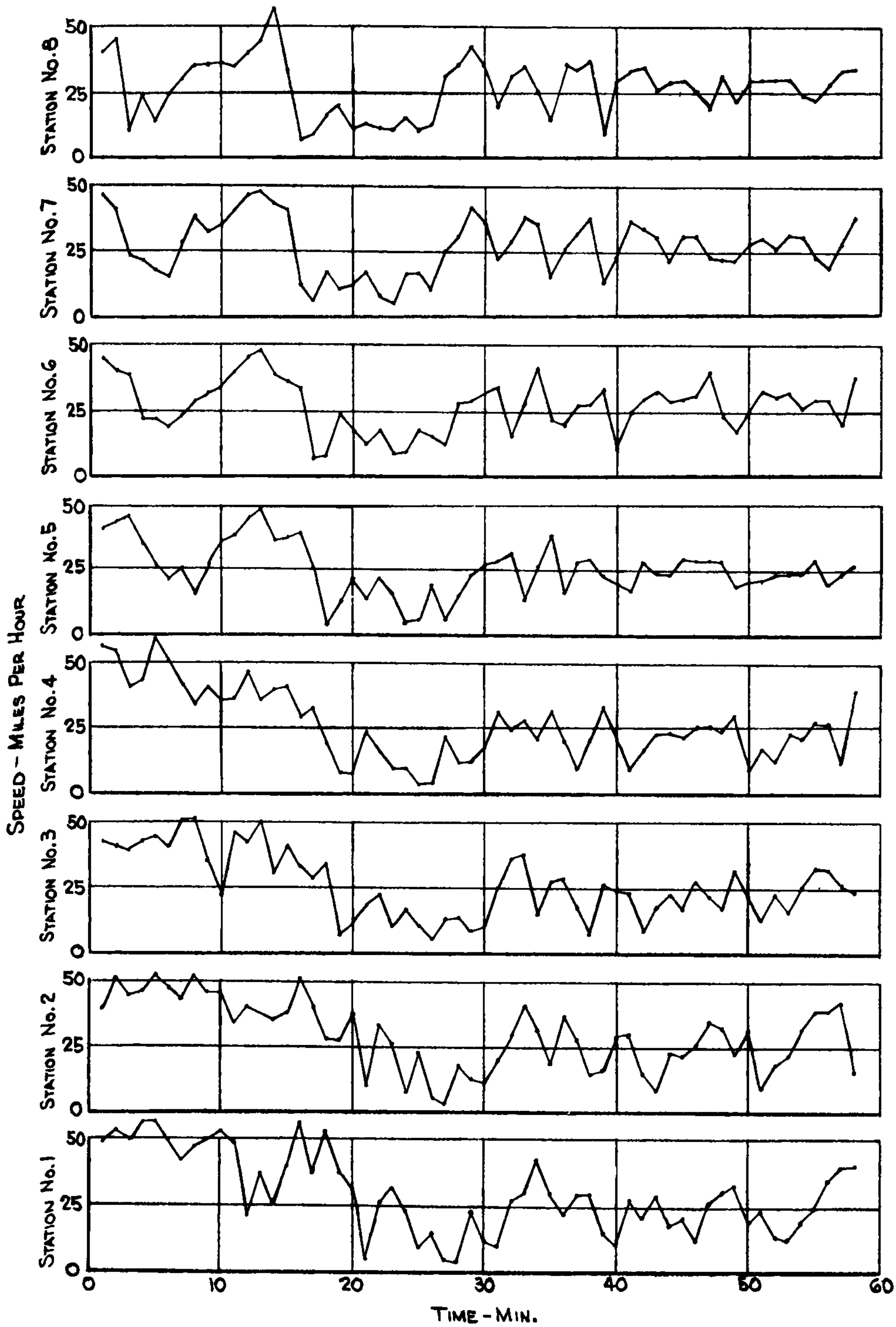


Figure 7. Average speed vs time.

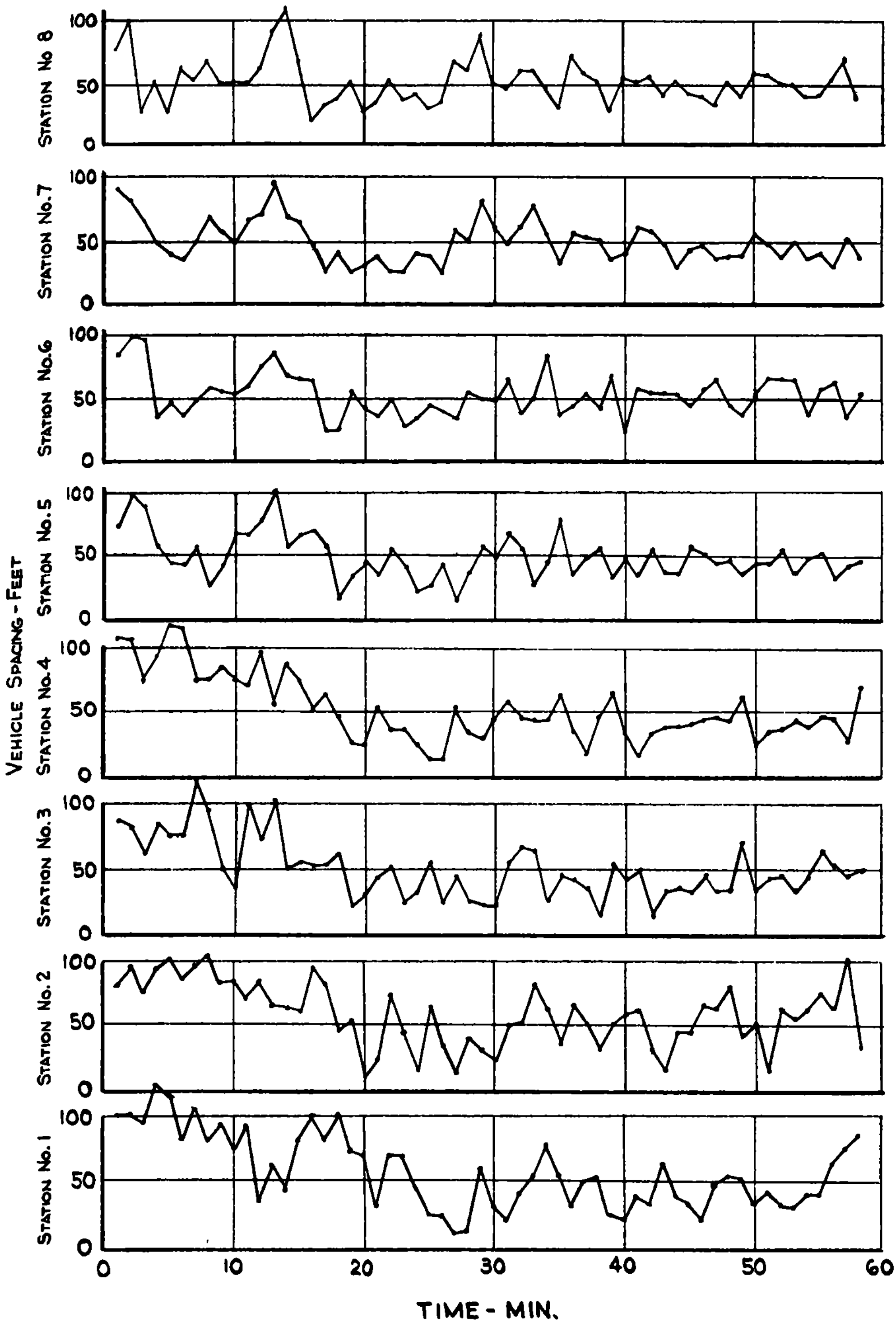


Figure 8. Average vehicle spacing vs time.

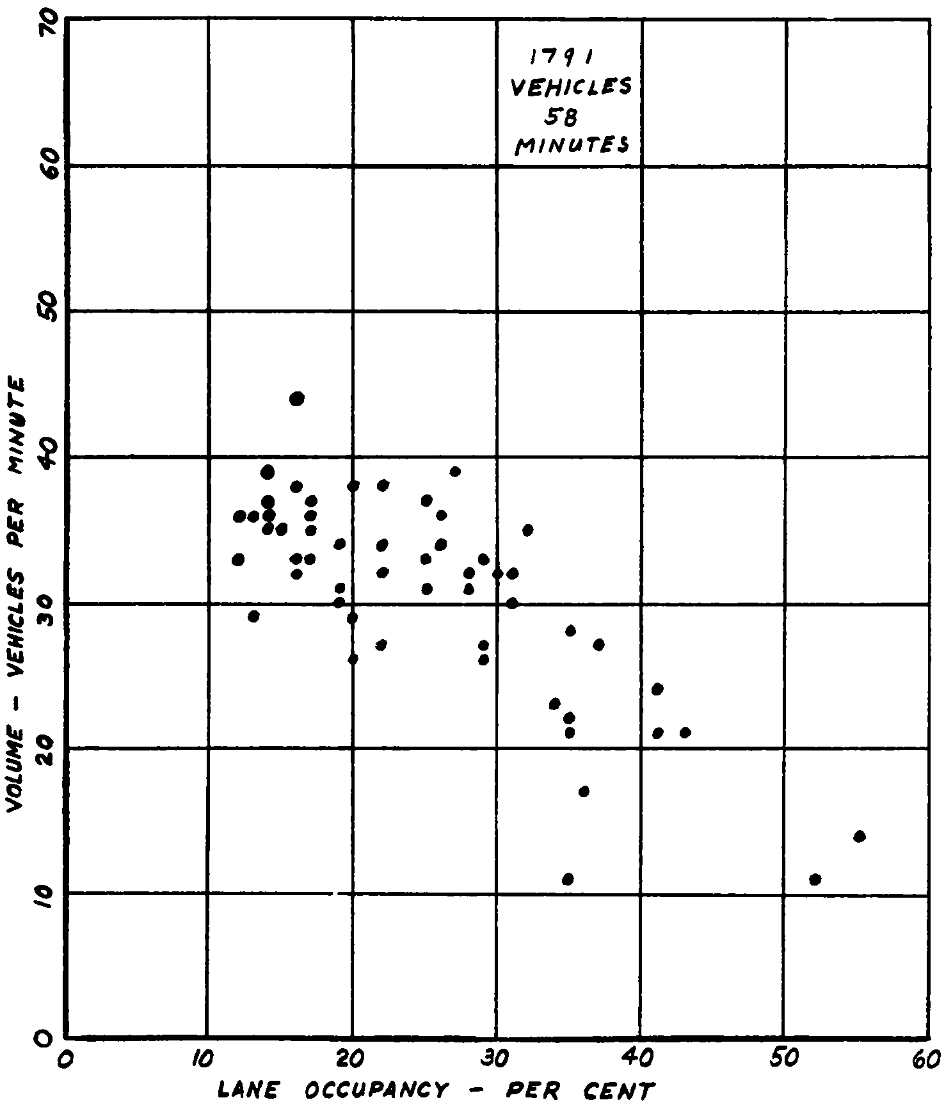


Figure 9. Volume vs lane occupancy.

obtained automatically. It may be concluded, therefore, that the average length of vehicles encountered on a typical expressway can be ascertained either by automatic means or by visual observation to a comparable degree of accuracy.

The IBM 650 computer was employed to compute the remaining traffic parameters using the formulas in Figure 5. The number of passenger cars within each minute interval was obtained by subtracting the number of commercial vehicles from the volume. Average speed within a minute interval was obtained by dividing the total length of vehicles passing the detector by the total time of vehicle passage. The total length of vehicles passing was computed as the sum of the total length of low vehicles (ALVl x N_l) plus the total length of high vehicles (AHVL x N_h). The total time of vehicle passage was computed by multiplying lane occupancy in percent by 60 sec. Multiplying this quotient by 60/88 provided average speed in miles per hour for the minute interval.

The average time headway was computed by dividing 60 sec by the minute volume. The average time spacing was computed by multiplying the average time headway by the percentage of the minute during which the detection zone was unoccupied by vehicles. Average distance headway and average distance spacing were obtained by multiplying the average time headway and the average time spacing respectively by the average speed.

TABULATED RESULTS

To illustrate the type of data obtainable with the system, a few curves were hand plotted from computer output cards for a typical 58-min sample of traffic. The original data were taken during a typical evening rush hour period when congestion on the expressway caused stoppages to occur within the test section. These curves are shown in Figures 6 through 10.

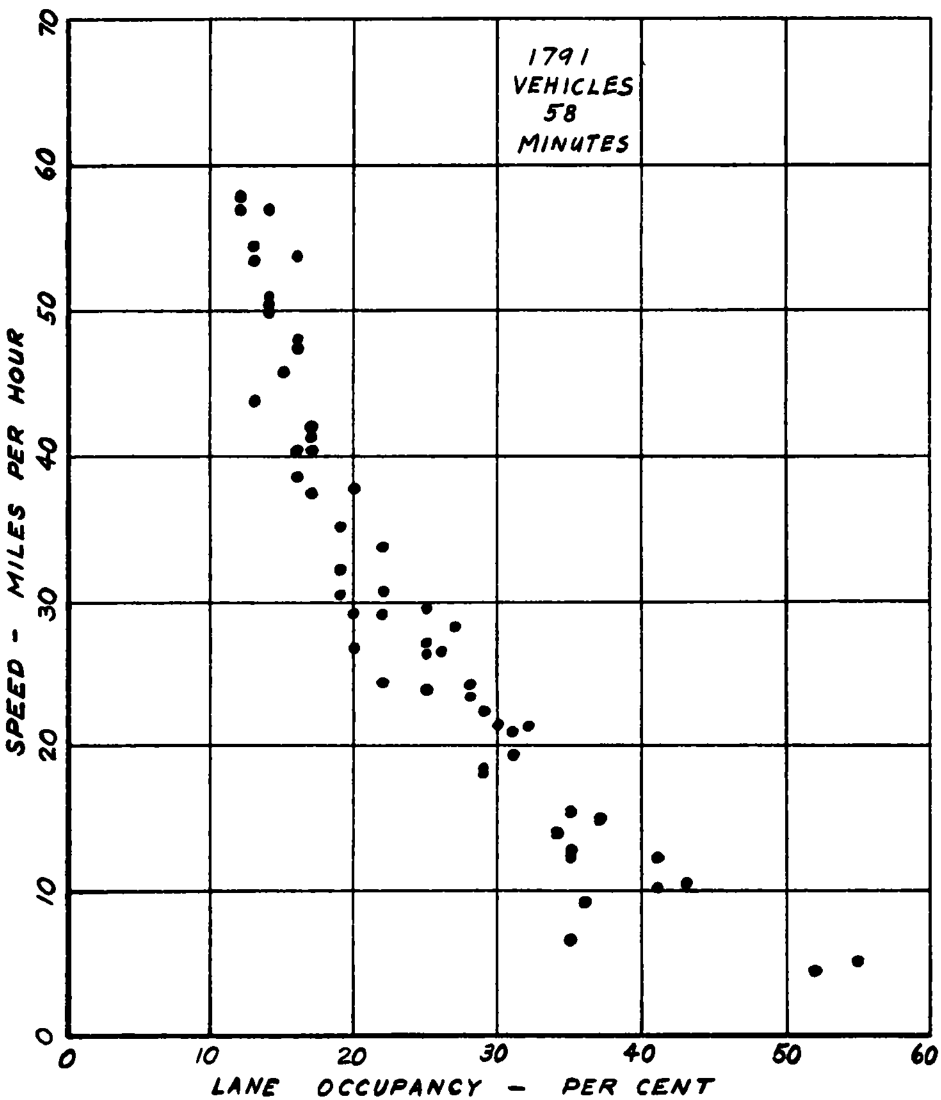


Figure 10. Speed vs lane occupancy.

Detector Station 1 was located at the entering end of the test section and Station 8 was located at the leaving end of the test section. The eight detectors were all mounted over the same lane. The distance between stations is given in Table 1.

TABLE 1

Between Stations	Distance (ft)
1 and 2	1,240
2 and 3	705
3 and 4	825
4 and 5	1,255
5 and 6	1,060
6 and 7	860
7 and 8	845

In Figure 6 lane occupancy measured simultaneously at the eight stations is plotted vs time. The shock waves composed of lane occupancy peaks are propagated against the direction of traffic flow; i. e., from Station 8 (downstream) to Station 1 (upstream). At time 16, for instance, a peak of lane occupancy is evident at Station 8. This peak may be traced as it moves backwards at a speed of ap-

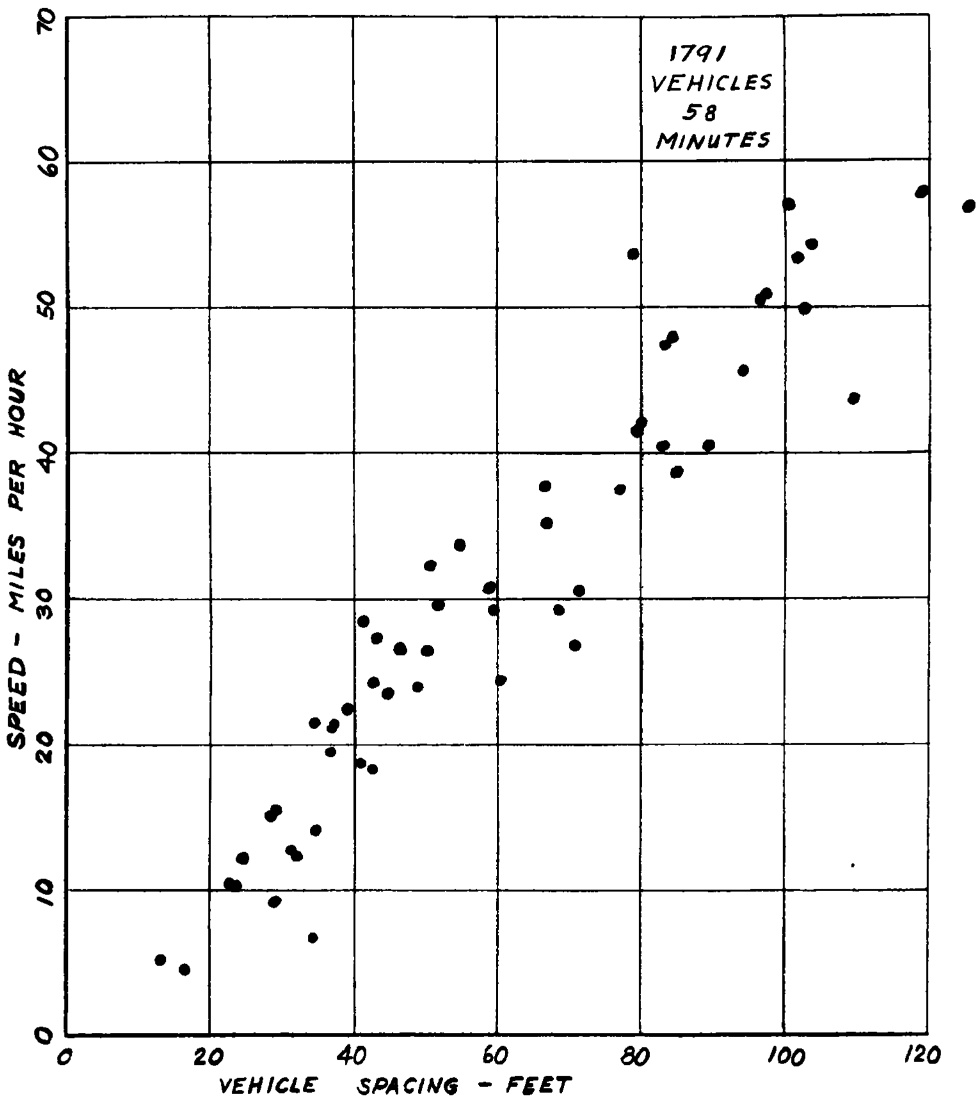


Figure 11. Speed vs vehicle spacing.

proximately 15 mph through the test section, occurring at time 17 at Station 7, time 17.5 at Station 6, time 18 at Station 5, time 19 at Stations 3 and 4, time 20 at Station 2, and time 21 at Station 1. In the absence of congestion, the peaks in lane occupancy would be expected to move with the direction of traffic and at approximately the speed of traffic.

Figure 7 shows a similar plot of average speed vs time for the eight stations. Here again the direction of propagation of the shock wave can be seen, along with the marked similarity between the speed curves taken at adjacent Stations; for example, at Stations 6 and 7 between times 10 and 30.

Figure 8 shows a plot of average vehicle spacing vs time for the eight stations. This information is significant because it involves one of the factors that directly influences driver behavior.

Figure 9 shows a plot of volume vs lane occupancy for Station 1. There is a sharp reduction in volume as lane occupancy increases above 20 percent. A curve drawn through the center of these points would have a negative slope because the congestion on the highway at the time the data were taken had already passed the maximum volume point. This curve would have normally started at the origin under light traffic conditions and would have reached a peak at approximately 15 percent lane occupancy. After this point, as lane occupancy increased, volume decreased as shown.

In Figure 10 average speed is plotted vs lane occupancy at Station 1. Here, the essentially horizontal left end of the curve is absent because congestion on the highway had already reached the point where a rapid decrease in speed occurred during the period of measurement. The speed is already undergoing a rapid reduction within the 10 to 20 percent region of lane occupancy.

Figure 11 shows a plot of speed vs vehicle spacing for Station 1. This curve indicates the tendency of drivers on this particular highway to maintain a spacing of approximately 20 ft for each 10 mph of speed.

The curves in Figures 6 through 11 were plotted by hand in order to complete this paper for presentation in January. Since that time, a number of programs have been worked out for machine plotting the data by feeding computer output cards into an IBM 407 tabulator, and thereby completing an additional important step in the automatic processing of highway traffic data.

ACKNOWLEDGMENTS

The first trial of this system was made in November 1961 on the John C. Lodge Expressway in Detroit, Mich., in connection with the Freeway Traffic Surveillance and Control Research Project. This project is jointly sponsored by the U.S. Bureau of Public Roads, Michigan State Highway Department, Wayne County Road Commission, and the City of Detroit.

In September 1960, the General Railway Signal Company was invited by the project to furnish vehicle detectors and expressway surveillance and control computers for long-term evaluation by the project. A representative amount of this equipment was furnished by the Company in November 1960. This equipment has been in continuous service by the project since that time as an adjunct to the television monitoring facilities in the collection of traffic data. From time to time, additional equipment has been furnished to the project by the Company for the purpose of making specialized measurements. The cooperation extended by the project in the installation of all equipment furnished, and the many courtesies extended to the Company by members of the project staff in the evaluation of this equipment, and in permitting the Company to conduct its own evaluations of new devices and systems that have been developed as a result of the contact with the project is hereby gratefully acknowledged.

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY–COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.