HIGHWAY RESEARCH RECORD

Number 199

Mathematical and Statistical Aspects of Traffic

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Foreword

The five papers, three discussions and two abridgments in this RECORD show the many-faceted attempts to study the dynamics of traffic flow by using mathematical models for traffic situations. In addition, methods of measuring existing traffic flow are described which provide some idea of the statistical parameters involved when specified counting patterns are used by the researchers.

Traffic researchers, statisticians, mathematicians and others concerned with the measurement of traffic flow will find this RECORD interesting. Federal and state personnel seeking a more scientific approach to traffic counting on the state highway systems will find the papers by Fox and by Parrish particularly informative.

Treadway and Oppenlander discuss some of the significant factors influencing travel speed and delay, and develop statistical models to estimate these factors. They found that the most significant factors accounting for variations in travel speed were types of roadside development and stream friction. At intersections, delays were chiefly caused by signal timing, traffic volume and occurrence of stops.

May and Keller investigated car-following models to develop deterministic flow models describing relationships between flow characteristics. They found that selection of the appropriate mathematical traffic model is sensitive to the critical level of the free speed, jam density and maximum flow.

Fox and Lehman report on a digital simulation model of the single-lane no-passing driving situation. Close validation of the factors used in describing the model found that it compared with those experimentally determined by actual stable driving patterns.

Parrish, Peterson and Threlkeld describe Georgia's unique automatic traffic data telemetry system which uses a centrally located computer connected by telephone to read out counters at each remote location. After a year of use, this system proved superior to previously used systems for obtaining continous traffic counts. The report outlines the advantages of such a computer based system as well as the redesign of techniques needed to achieve the advantages. Three discussions highlight this paper and bring out significant comments. A closure by Parrish completes the work.

Bodle indicates the range of differences that can be expected when using various samplings of traffic to obtain ADT traffic figures. Standard deviations are relatively high for low-volume roads and small samples. Because of the great amount of traffic data obtained by sampling for use as a basis for highway planning, knowledge of the data limitations is vital.

Reilly and Radics present an abridgment of their paper concerning 30th peak hour factor trend. Another abridgment of a paper about vehicle headway by Gwynn completes this RECORD.

Contents

Statistical Modeling of Travel Speeds and Delays on a High-Volume Highway

T. B. TREADWAY, Graduate Assistant, and
J. C. OPPENLANDER, Associate Professor, Joint Highway Research Project, Purdue University

•VEHICULAR travel has increased at a tremendous rate in recent years, and the construction of new highways and the improvement of existing facilities have failed to keep pace with the growth of motor-vehicle travel. The problem is especially acute in urban areas where major arterial highways lack needed capacity for handling large movements of intracity travel. Inadequate planning and improvement of these facilities have resulted in congestion and delays which are costly and irritable to the road users.

Limited-access freeways are being constructed in large urban areas to accommodate major flows of through and intracity travel. Existing arterial highways continue to play an important role in the movement of traffic, however, and they serve as collectors and distributors for the new expressways. The improvement of these arterial facilities is necessary for the efficient and safe functions of the complete urban area transportation system. As a result of the emphasis placed on the construction of new roads, the continuing renovation of existing highways has been largely neglected.

This project was undertaken by the Joint Highway Research Project of Purdue University, the Indiana State Highway Commission, and the U.S. Bureau of Public Roads to evaluate the effectiveness of traffic engineering as applied to the improvement of a congested urban arterial highway. The purpose of this research investigation, as a portion of that project, was a detailed analysis of travel speeds and delays. The specific objectives of this study were (a) to determine the significant factors and variables which influence travel speeds and delays; and (b) to develop statistical models using these significant variables to predict travel speeds and delays.

The various statistical models developed to express travel speeds and delays as functions of factors and variables that are descriptive of the roadway and its environment afforded an insight into the characteristics of traffic flow on the study route. The relationships permitted the determination and evaluation of appropriate improvements in the existing roadway and in traffic control devices to minimize travel delays. The planning and design of new facilities are also benefited by the development of estimating equations to predict travel speeds and delays.

REVIEW OF LITERATURE

Travel-time studies have been performed for various purposes, all of which are related to the evaluation of the level of service afforded by a highway section. Because the driver often considers total time in reaching his destination as the criterion for selecting a certain route, travel times are given consideration in the evaluation of a highway system (4).

Previous investigations have been performed to determine those variables that have significant effects on travel speed. These variables are generally classified in the categories of traffic stream, roadway geometry, roadside development, and traffic controls.

Overall travel speed appears to be related closely to traffic volume. W. P. Walker found that for a highway section on which all variables were controlled except volume,

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the average speed of traffic decreased with an increase in volume. In rural areas, a straight-line relationship occurred between volume and average travel speed when the critical density of the highway was not exceeded. Beyond this density, speed continued to decrease but volume also decreased because of congestion (13). In the Chicago area, travel speeds were observed to decrease continually with increasing volumes without a break signifying the critical density (7).

The characteristics of the traffic stream have important effects on travel speed, but these influences have not been conclusively substantiated by field investigations (13). The character of traffic includes such items as through traffic, local traffic, driver residence, trip purpose, and trip destination. In one study, the percentage of commercial vehicles had a negligible influence on travel speed (2).

Little information is available concerning the relationship of overall travel speed with highway geometry. A linear correlation of travel time with street width was made by R. R. Coleman. The width alone did not affect travel time significantly (2).

The effects of various types of impedances on the average overall speeds of test vehicles were studied in North Carolina. Many of these impedances were related to commercial development. They included various types of turning movements, slow-moving vehicles, marginal friction such as parked cars and pedestrians, and vehicles passing in the opposing direction. The presence of slow-moving vehicles had the most significant influence in reducing speeds. Left and right turns from the direction of travel of the test car were also important causes of speed reductions. The remaining impedances examined in the study were both individually and collectively insignificant (3).

Investigations have been made to evaluate and compare the performance of different types of traffic signals and their relationships to travel speeds and delays. W. N. Volk reported that stopped-time delays to vehicles which were required to stop were much greater at fixed-time signals than for traffic-actuated signals and for two-way and four-way stopped-controlled intersections. In the same study, intersections exhibiting similar relationships between delays and volumes were grouped together. Simple linear regression equations were developed to predict delay from traffic volume with an acceptable degree of reliability (12).

A straight-line relationship between mean travel time and signal density was established for urban areas in Pennsylvania. Regression equations developed for various volume-to-capacity ratios were reasonably precise for uncongested conditions. Travel times for test sections with coordinated signals were compared with times for a series of non-coordinated signals. The sections with coordinated signals had reduced travel times, but the difference was not statistically significant (2).

PROCEDURE

The highway analyzed in this investigation was the U.S. 52 Bypass at Lafayette, Indiana. A variety of traffic functions served by this two-lane facility include the following:

1. Through traffic between Indianapolis, Chicago, and intermediate points;
2. Terminal traffic from throughout Tippecanoe County to Lafayette, an industrial center and the county seat, and to Purdue University in adjoining West Lafayette; and
3. Local traffic to commercial and industrial establishments abutting the bypass.

Study Design

The bypass was divided into 18 homogeneous study sections by considering geometry, speed limit, roadside development, and location of traffic signals (Fig. 1). Signalized intersections were separated from the other sections of this route. These intersections, which were categorized as "interrupted flow," represented a special condition where traffic was required to stop for a red signal. A distance of 500 ft on each side of the center of the intersection was established to define the zone of influence of the traffic signal. Sections 3, 8, 11, 13, and 15 were classified in this category of interrupted flow. The traffic signal in section 3 was semi-actuated, and the other four signals had fixed-time cycles. The remaining portion of the two-lane bypass was

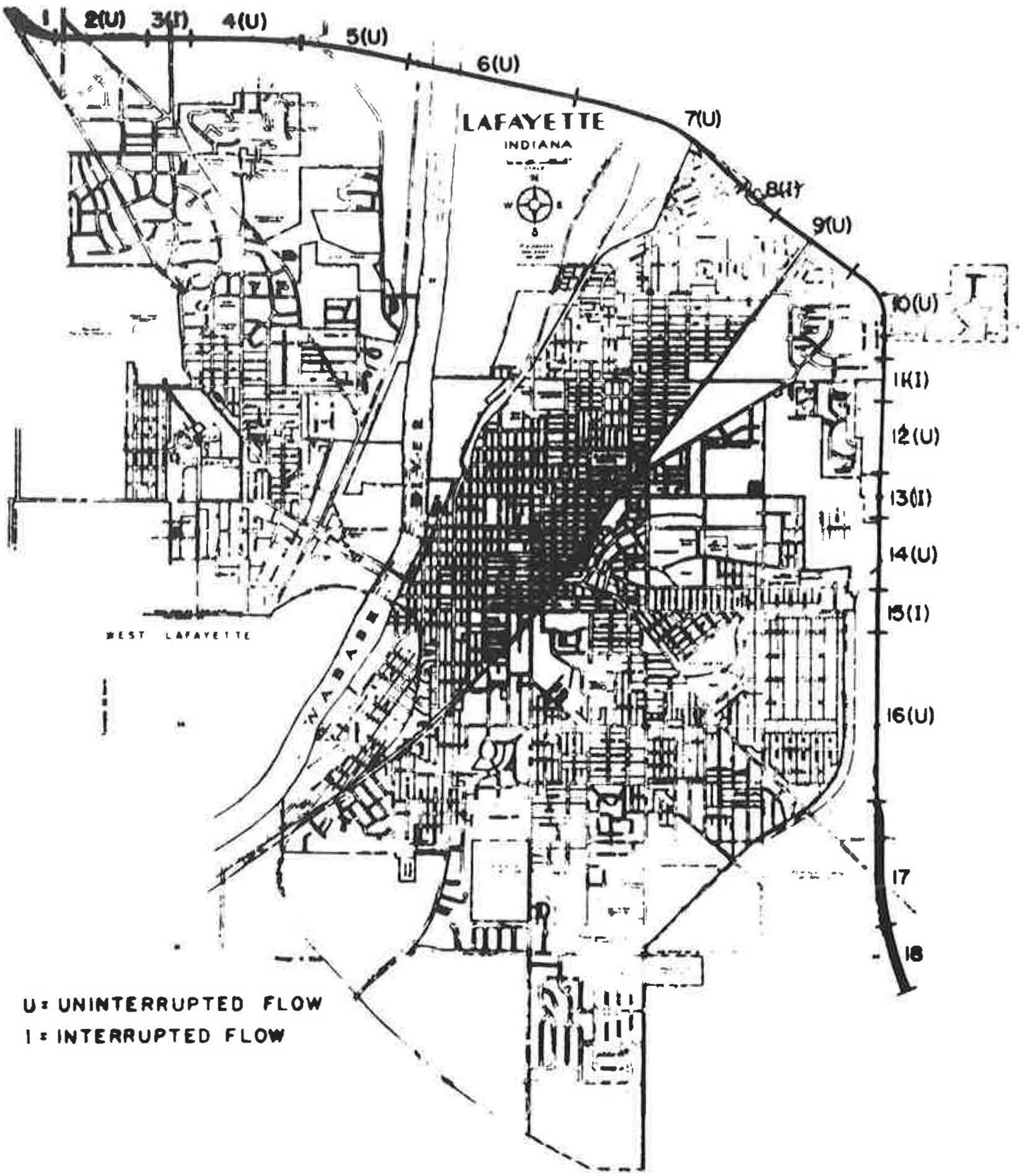

Figure 1. Test sections of U.S. 52 Bypass.

designated and analyzed as "uninterrupted flow." This category included sections 2, 4, 5, 6, 7, 9, 10, 12, 14, and 16.

Three sections of the bypass were not considered in the multivariate analysis of the interrupted and the uninterrupted flows. Sections 1 and 17 included transitions from a four-lane divided highway to a two-lane roadway; section 18 was entirely a four-lane facility.

The selection of the variables to be included in the multivariate analyses was dependent on an examination of those variables considered in previous investigations and on the availability and ease of collecting data. The following variables were included in the analysis of uninterrupted flow by direction of travel:

1. Intersecting streets on the right, number per mile;
2. Intersecting streets on the left, number per mile;
3. Intersecting streets on both sides, number per mile;
4. Access drives on the right, number per mile;
5. Access drives on the left, number per mile;
6. Access drives on both sides, number per mile;
7. Commercial establishments on the right, number per mile;
8. Commercial establishments on the left, number per mile;
9. Commercial establishments on both sides, number per mile;
10. Posted speed limit, mph;
11. Average shoulder width on the right, ft;
12. Average shoulder width on the left, ft;
13. Portion of section length where passing was not permitted, percent;
14. Average absolute grade, percent;
15. Average algebraic grade, signed percent;
16. Average curvature, deg;
17. Geometric modulus—based on gradient, lane width, sight distance, and curvature—(11);
18. Average safe stopping sight distance, ft;
19. Practical capacity, vph;
20. Possible capacity, vph;
21. Advertising signs, number per mile;
22. Warning signs, number per mile;
23. Information signs, number per mile;
24. Regulatory signs, number per mile;
25. Presence of a truck climbing lane (0 if no, 1 if yes);
26. Presence of a signal in the next section (0 if no, 1 if yes);
27. Presence of a signal in the preceding section (0 if no, 1 if yes);
28. Monday (0 if no, 1 if yes);
29. Tuesday (0 if no, 1 if yes);
30. Wednesday (0 if no, 1 if yes);
31. Thursday (0 if no, 1 if yes);
32. Friday (0 if no, 1 if yes);
33. 8:00 a.m. to 10:00 a.m. (0 if no, 1 if yes);
34. 10:01 a.m. to 12:00 n. (0 if no, 1 if yes);
35. 12:01 p.m. to 3:00 p.m. (0 if no, 1 if yes);
36. 3:01 p.m. to 6:00 p.m. (0 if no, 1 if yes);
37. Traffic volume in direction of travel, vehicles per 15 min;
38. Traffic volume in the opposing direction of travel, vehicles per 15 min;
39. Commercial vehicles (larger than a pickup truck), percent;
40. Southeast direction of travel (0 if no, 1 if yes);
41. Northwest direction of travel (0 if no, 1 if yes);
42. Total traffic volume, vehicles per 15 min;
43. Volume to practical capacity ratio;
44. Volume to possible capacity ratio; and
45. Overall travel speed, mph.

The remaining variables, included in the analysis of interrupted flow, are as follows:

46. Presence of a semi-actuated signal (0 if no, 1 if yes);
47. Presence of a signal indication for left-turn movement (0 if no, 1 if yes);
48. Presence of a right-turn lane (0 if no, 1 if yes);
49. Length of approach to turning lane, ft;
50. Length of exit for merging lane, ft;
51. Average algebraic grade of approach, percent;
52. Average algebraic grade of exit, percent;
53. Intersecting streets, excluding that street with the signal, on the right, number;
54. Intersecting streets, excluding that street with the signal, on the left, number;
55. Intersecting streets, excluding those streets with the signal, on both sides, number;

56. Access drives on the right, number;
57. Access drives on the left, number;
58. Access drives on both sides, number;
59. Commercial establishments on the right, number;
60. Commercial establishments on the left, number;
61. Commercial establishments on both sides, number;
62. Cycle length of traffic signal, sec per cycle;
63. Green time in direction of flow, sec per cycle;
64. Practical approach capacity, vph;
65. Advertising signs, number;
66. Warning signs, number;
67. Information signs, number;
68. Regulatory signs, number;
69. Southeast direction of flow (0 if no, 1 if yes);
70. Northwest direction of flow (0 if no, 1 if yes);
71. Vehicles making left turns from the direction of travel, percent;
72. Vehicles making right turns from the direction of travel, percent;
73. Vehicles making left turns from the opposing direction of travel, percent;
74. Average shoulder width on the right, ft;
75. Average shoulder width on the left, ft;
76. Monday (0 if no, 1 if yes);
77. Tuesday (0 if no, 1 if yes);
78. Wednesday (0 if no, 1 if yes);
79. Thursday (0 if no, 1 if yes);
80. Friday (0 if no, 1 if yes);
81. 8:00 a.m. to 10:00 a.m. (0 if no, 1 if yes);
82. 10:01 a.m. to 12:00 n. (0 if no, 1 if yes);
83. 12:01 p.m. to 3:00 p.m. (0 if no, 1 if yes);
84. 3:01 p.m. to 6:00 p.m. (0 if no, 1 if yes);
85. Traffic volume approaching the intersection in the direction of travel, vehicles per 15 min;
86. Traffic volume approaching the intersection in the opposing direction of travel, vehicles per 15 min;
87. Total traffic volume entering the intersection on all approaches, vehicles per 15 min;
88. Commercial vehicles (larger than a pickup truck), percent;
89. Green time to cycle length ratio;
90. Approach volume to total volume entering intersection ratio;
91. Approach volume to practical capacity ratio;
92. Overall travel speed, mph; and
93. Delay (total delay for the test vehicle traveling through the intersection), sec.

Variables comprising street, access drive, and commercial densities were expressed in a "per-mile" form for the uninterrupted flow sections because of the variation in section lengths. The lengths of the interrupted flow sections were uniform, and similar variables for this analysis were retained as an absolute value. Because all traffic lanes of the bypass were 11 ft wide, lane width was not included as a variable.

Collection of Data

An inventory of the physical characteristics for the bypass was made from construction plans and aerial photographs. In some cases, actual measurements were performed in the field. Section lengths measured by a fifth-wheel odometer were checked with the control points located on the construction plans.

Possible and practical capacities were computed in accordance with methods described in the "Highway Capacity Manual" (6). Volumes were recorded simultaneously with the measurement of travel times. Counts were taken at four points along the test route for 15-min intervals. The control stations, located in sections 2, 6, 10, and 16, were used to expand the volumes by hour and by direction for the remaining sections. All volumes were obtained with recording counters actuated by pneumatic hoses.

The result of a traffic composition analysis at representative sections was that the percentage of vehicles larger than a small two-axle pickup truck was constant for all sections of the bypass. Hourly fluctuations did occur, and ratios were established for different periods of the day. The percentages of vehicles turning right and left at a given signalized intersection did not vary significantly for different periods of the day. Average values for turning movements were established for each intersection.

Travel times were measured by the average-car technique. The driver operated the test car at a speed which in his opinion was representative of the average speed of the traffic stream. During periods when the test car was not influenced by other vehicles, the driver observed the speed limit. Travel times at the section boundaries were recorded with a stop watch by an observer in the car. Whenever the vehicle was forced to stop, the duration of this stop was measured with a second stop watch.

Forty runs were made in each direction to assure a good estimate of the mean travel speed for each section (1, 10). This procedure provided a sample size of 800 observations for the ten sections representing uninterrupted flow. Five sections provided a sample size of 400 observations for the analysis of interrupted flow.

All test runs were made over the entire length of the bypass. The test vehicle entered the traffic stream about 0. 5 mile before the first section and continued for approximately the same distance after the last section. The data collections were made on weekdays between 8:00 a. m. and 6:00 p. m., and during clear, dry weather conditions. Trips were made during peak and off-peak hours to insure a variation in traffic volumes.

Analysis of Data

The data were first processed and summarized before the multivariate analyses were initiated. Travel times for each run and section were converted to overall travel speeds as follows:

$$S = \frac{L(3600)}{T} \tag{1}$$

where

S = overall travel speed—mph,
L = length of test section—miles, and
T = travel time—sec.

The mean travel speed and stop time for each section and direction were calculated.

The travel delay for each run at signalized intersections was computed as follows:

$$D = T - \left[\frac{L\,(3600)}{0.5\,(\bar{S}_B + \bar{S}_A)} \right] \tag{2}$$

where

D = travel delay—sec,

T = travel time—sec,

L = length of section—miles,

$\bar{S}_B$ = average overall travel speed of adjacent section before intersection—mph, and

$\bar{S}_A$ = average overall travel speed of adjacent section after intersection—mph.

The term in the brackets in Eq. 2 was considered as the hypothetical travel time if the intersection had not existed. In a few cases where the computed delay was a negative value, these delays were assumed to be zero. The delays were averaged for each intersection by direction.

The average delay per vehicle for each signalized intersection was again calculated by a theoretical method which depends on the red interval of the cycle, the average arrival headway in the traffic stream, and the starting performance of the queue. The average delay per vehicle is

$$\bar{d} = \frac{A}{C}\left[nR - \frac{n^2A}{2} + \frac{2.1(n)(n+1)}{2} + 3.7n - Q\right] \tag{3}$$

where

$\bar{d}$ = average delay per vehicle—sec,
A = average arrival headway—sec,
C = cycle length—sec,
n = total number of vehicles stopped in R,
R = length of stop time in cycle—sec, and
Q = constant (depending on the value of n).

Complete details of this derivation are presented in the textbook, "Traffic Engineering" (8).

The first step in each multivariate analysis was the calculation of a correlation matrix for the study variables. Both factor analysis and multiple linear regression techniques were utilized in this statistical modeling of travel speeds and delays on a high-volume highway. Before the factor analysis was performed, the dependent variables were deleted from the correlation matrix. This procedure permitted later correlations between the dependent variables and the generated factors.

Orthogonal factors were generated so that a maximum contribution to the residual communality was provided. The generation of the factors was terminated when the eigenvalue became less than 1.00. The factor matrix was then rotated with the varimax method to aid interpretation of each factor. An examination of the rotated-factor matrix resulted in the identification of the generated factors.

Coefficients were developed to express each factor in terms of the original variables. Thus, the factors were evaluated from the values of the variables that were significantly related to each factor. The final step in the factor analysis was the correlation of the generated factors with the dependent variables. The resulting multiple linear regression equation expressed the dependent variable as a function of the significant factors (9).

A build-up regression analysis was then performed on the study variables (5). The following criteria were used in rating the variables for inclusion in the final multiple linear regression equations:

1. Each significant factor was represented by at least one closely related variable;
2. The final model involved a minimum of computations with readily obtainable data; and
3. The multiple coefficient of determination did not increase significantly by including additional variables.

RESULTS

The results of the multivariate analyses of travel speeds and delays are discussed in this section. The data were first summarized by computing mean travel speeds and delays for each study section. A factor analysis was performed to gain an insight into the relationships among the study variables. Multiple linear regression equations were developed to predict mean travel speeds and delays in terms of the factors and the variables. The results of these analyses were then applied in recommending improvements to minimize delays on the bypass location. All variables are identified by the numbers which are listed in the discussion of the experimental design. Each factor is labeled with a letter in the evaluation of the results of the factor analysis.

Uninterrupted Flow

The overall travel speeds for each test section in the analysis of uninterrupted flow were averaged for both directional flows and the combined flows. These mean travel

TABLE 1

AVERAGE OVERALL TRAVEL SPEEDS, UNINTERRUPTED FLOW

Section	Average Overall Travel Speed, mph		
	SE Flow	NW Flow	Combined Flows
2	41.4	40.6	41.0
4	42.0	47.7	44.9
5	51.0	52.5	51.8
6	52.8	53.9	53.4
7	45.1	45.2	45.2
9	40.3	42.0	41.2
10	40.8	42.6	41.7
12	34.4	39.3	36.9
14	30.4	33.5	32.0
16	35.3	35.3	35.3

speeds are given in Table 1. The highest speeds occurred in sections 5, 6, and 7 where the commercial roadside development was sparse. In sections 12, 14, 16, where heavy commercial strip development occurred, the lowest speeds were recorded.

Factor Analysis

A correlation matrix was calculated for variables 1 to 45. Variables 2, 5, 8, and 38 were deleted from the matrix to avoid singularities. Variables 40 and 41, which identified the directional flows, and variable 45, overall travel speed, were also removed. The speed variable was later correlated with the generated factors. The revised correlation matrix was factorized with unities inserted in the main diagonal of the matrix. The 38 variables were reduced to 13 factors which accounted for 88 percent of the total variance of the variables.

The 13 factors were then rotated to aid in their identification. The signed factor coefficients indicate the relative importance of each variable in the explanation of the generated factors. The plus and minus signs are indicative, respectively, of the increasing or decreasing presence of the variables in the composition of the factors. Each factor, along with its major component variables and their respective coefficients, is included in the following list.

Commercial Development—This factor includes a concentration of commercial establishments, access drives, and related conditions indicating a high degree of commercial development:

6. Access drives on both sides, +0.9294;
9. Commercial establishments on both sides, +0.9287;
10. Speed limit, -0.4930;
11. Shoulder width on right, +0.2341;
12. Shoulder width on left, +0.5259;
26. Signal in next section, +0.4114; and
27. Signal in preceding section, +0.5888.

Horizontal Resistance—Horizontal roadway features influencing traffic movement are included in this group:

13. No-passing zone, +0.9244;
16. Average curvature, +0.7644;
17. Geometric modulus, -0.8693;
18. Stopping sight distance, -0.7443;
19. Practical capacity, -0.7638; and
20. Possible capacity, -0.7556.

Evening Shopping Travel—This category describes late afternoon shopping trips on the evenings when local stores are open:

28. Monday, +0.3523;
31. Thursday, -0.6170;
32. Friday, +0.4392;
33. 8:00 to 10:00, -0.2464;
34. 10:01 to 12:00, -0.7637; and
36. 3:01 to 6:00, +0.8724.

Flat Topography—A level roadway alignment is reflected in this factor:

15. Algebraic grade, -0.9151; and
25. Truck climbing lane, -0.6860.

Time Variations—This factor, which is not completely defined, expresses variations in the time periods and the days when the data were collected:

30. Wednesday, -0.7612; and
35. 12:01 to 3:00, -0.8616.

Urban Development—This category indicates that the highway is located in an urban area:

3. Intersecting streets on both sides, +0.7510;
10. Speed limit, -0.4368; and
24. Regulatory signs, +0.4697.

Driver Distractions—This group includes items which distract the driver's attention from the highway:

21. Advertising signs, +0.7895;
26. Signal in next section, +0.5416; and
27. Signal in preceding section, -0.4861.

Further Time Variations—Additional variations in times are reflected in this undefined factor:

31. Thursday, -0.4723;
33. 8:00 to 10:00, -0.8820; and
34. 10:01 to 12:00, +0.4830.

Outbound Traffic—Traffic heading away from the urban area is described by this factor:

23. Information signs, -0.8789;
24. Regulatory signs, -0.5969; and
37. Volume in direction of travel, -0.2154.

Day-of-Week Variations—This factor, generated by daily variations, is not completely discernible:

28. Monday, +0.8559;
30. Wednesday, -0.2779; and
32. Friday, -0.6026.

Rural Development—This group of variables describes a rural-type highway with little roadside development:

3. Intersecting streets on both sides, -0.2194;
9. Commercial establishments on both sides, -0.2030;
11. Shoulder width on right, -0.9113; and
26. Signal in next section, -0.2891.

Stream Friction—Conditions which cause congestion within the traffic stream ar indicated by this factor:

20. Possible capacity, -0.5313;
25. Truck climbing lane, -0.5902;
26. Signal in next section, +0.4616;
37. Volume in direction of travel, +0.3986; and
44. Volume to possible capacity ratio, +0.4952.

Additional Day-of-Week Variations—This undefined facto for different days of the week:

28. Monday, -0.2780;
29. Tuesday, +0.9610; and
32. Friday, -0.3467.

The factors were readily identified e day-of-week characteristics. Th different days and time perio

The next execution in the factor-analysis procedure was the computation of the factor-score matrix. The coefficients in this matrix permit the factors to be evaluated as functions of the original variables which are expressed in terms of multiple linear regression equations. Examples of these equations are presented later in the results.

The final step was the correlation of each factor with the mean overall travel speed to determine those factors which significantly accounted for the variation in travel speeds (Table 2). The four dominant factors were, in their order of importance, commercial development, stream friction, urban development, and rural development. The following multiple linear regression equation was evolved to predict mean travel speeds from the significant factors:

$$S_1 = 42.30 + 9.185\,(-0.5507F_A - 0.1874F_F + 0.1744F_K - 0.2674F_L) \qquad (4)$$

where

S_1 = mean travel speed, mph;
F_A = commercial development;
F_F = urban development;
F_K = rural development; and
F_L = stream friction.

The multiple correlation coefficient of this expression was 0.664. Approximately 44 percent of the total variation in travel speeds was explained by the four factors. The precision of the estimate was measured by the standard error of estimate of 6.87 mph. The factors of commercial development, urban development, and stream friction were negatively related to travel speed, whereas the remaining factor of rural development was positively associated with travel speed. Eq. 4 is most useful in an explanatory sense rather than for actual computations.

Multiple linear regression equations were developed to evaluate the significant factors in terms of those variables which predominantly explained each factor. The following equations were written from the coefficients in the factor-score matrix:

$$\begin{aligned} F_A = {} & -0.1070Z_3 + 0.2498Z_4 + 0.2064Z_6 + 0.2438Z_7 + 0.2068Z_9 \\ & + 0.1930Z_{27} \end{aligned} \qquad (5)$$

$$\begin{aligned} F_F = {} & 0.3878Z_1 + 0.2954Z_3 - 0.1012Z_9 - 0.1190Z_{10} + 0.2558Z_{12} \\ & -0.1444Z_{22} - 0.1214Z_{23} + 0.2535Z_{24} - 0.1106Z_{26} - 0.1049Z_{43} \end{aligned} \qquad (6)$$

$$\begin{aligned} F_K = {} & + 0.1134Z_1 + 0.1870Z_4 + 0.1688Z_7 + 0.1179Z_{10} - 0.5580Z_{11} \\ & + 0.1456Z_{15} + 0.1800Z_{18} - 0.1575Z_{19} - 0.1256Z_{20} - 0.2860Z_{22} \\ & - 0.1460Z_{26} + 0.1384Z_{43} \end{aligned} \qquad (7)$$

$$\begin{aligned} F_L = {} & -0.1102Z_1 - 0.1193Z_{10} - 0.3897Z_{15} + 0.1130Z_{15} - 0.2064Z_{16} \\ & + 0.1564Z_{17} - 0.2553Z_{20} - 0.1513Z_{21} - 0.2502Z_{25} + 0.2362Z_{26} \\ & - 0.2523Z_{27} + 0.1135Z_{37} + 0.1144Z_{42} + 0.1719Z_{44} \end{aligned} \qquad (8)$$

where

F_i = common factor, and
Z_i = standard score of variable.

The values of the dependent and independent variables in these equations are expressed standard-score form. Standard scores are computed by the following relationship:

$$Z_i = \frac{X_i - \overline{X}_i}{s_i} \qquad (9)$$

TABLE 2

CORRELATION OF MEAN TRAVEL SPEED WITH FACTORS, UNINTERRUPTED FLOW

Factor	Correlation Coefficient
A	-0.5507[a]
B	-0.0525
C	-0.0928
D	+0.0049
E	-0.0659
F	-0.1874[a]
G	+0.0956
H	-0.0920
I	+0.0535
J	+0.0289
K	+0.1744[a]
L	-0.2674[a]
M	-0.0400

[a]Significant at the 5 percent level.

where

Z_i = standard score of variable,
X_i = observed value of variable,
$\overline{X}_i$ = grand mean of variable, and
s_i = standard deviation of variable.

Multiple Linear Regression and Correlation Analysis

The second phase of the multivariate analysis of uninterrupted-flow conditions was the development of a multiple linear regression equation to predict mean travel speed from the significant variables. The 38 variables in the revised correlation matrix were included in a build-up regression technique.

The following multiple linear regression equation was selected as the most valid functional relationship for the estimation of overall travel speed:

$$S_2 = 68.60 - 0.4541X_3 - 0.1775X_9 - 0.1007X_{13} - 0.0150X_{19} - 0.0301X_{42} \tag{10}$$

where

S_2 = mean travel speed, mph;
X_3 = intersecting streets on both sides, number per mile;
X_9 = commercial establishments on both sides, number per mile;
X_{13} = portion of section length where passing was not permitted, percent;
X_{19} = practical capacity, vph; and
X_{42} = total traffic volume, vehicles per 15 min.

The various statistics of this regression equation are given in Table 3. The measure of correlation was expressed by a multiple correlation coefficient of 0.704. The variables of intersecting streets, commercial establishments, no-passing zone, practical capacity, and total volume accounted for 50 percent of the total variation in overall travel speeds for the uninterrupted flow sections of the bypass. These five variables were negatively related to travel speed. The standard error of estimate of 6.55 mph was a measure of the precision of the equation.

A significant portion of the unexplained variation in overall travel speeds was probably caused by individual driver behavior. Variations were evident in the driving habits of vehicle operators as the test-car driver attempted to relate his speed to the average speed of the traffic stream. In addition, variations occurred within the test driver in his reactions to the many conditions influencing his speed.

TABLE 3

MULTIPLE LINEAR REGRESSION AND CORRELATION ANALYSIS, UNINTERRUPTED FLOW[a]

Variable	Net Regression Coefficient	Standard Error
3	-0.4541	0.1214
9	-0.1775	0.0211
13	-0.1007	0.0135
19	-0.0150	0.0022
42	-0.0301	0.0044

[a]Dependent variable: travel speed; intercept = 68.60 mph; multiple correlation coefficient = 0.704; and standard error of estimate = 6.55 mph.

Interrupted Flow

The analysis of interrupted flow followed the same pattern as the investigation of uninterrupted flow. Mean overall travel speeds and mean running speeds were computed for directional flows and for the combined flows in each section (Table 4).

TABLE 4

AVERAGE TRAVEL SPEEDS, INTERRUPTED FLOW

Section	Average Travel Speed, mph					
	SE Flow		NW Flow		Combined Flows	
	Overall Speed	Running Speed	Overall Speed	Running Speed	Overall Speed	Running Speed
1[a]	26.8	29.5	42.4	42.4	34.6	36.0
3	30.1	31.9	29.3	32.2	29.7	32.1
8	21.7	26.4	24.1	28.2	22.9	27.3
11	19.9	25.3	27.4	30.0	23.7	27.7
13	23.6	25.9	24.8	27.8	24.2	26.9
15	19.7	23.5	21.1	25.7	20.4	24.6
17[a]	35.0	38.0	32.0	35.7	33.5	36.9
18[a]	29.2	32.9	24.1	31.9	26.7	32.4

[a]Not included in the multivariate analysis.

The overall speed equaled the running speed in the northwest flow of section 1 because no stop was required in this direction. The mean speeds in sections 17 and 18 were higher than for the other sections; these sections were longer and the delays caused by the signal were distributed over a greater distance. Of the five sections included in the multivariate analysis, section 3, which had a semi-actuated traffic signal for the traffic on the road crossing the bypass, had the highest overall travel speeds.

The stopped times for each section were summarized by computing the mean stopped time of each run, the mean duration of the stop, and the percent of the runs when stops occurred. These results are given in Table 5. Because a stop sign existed in the southeast flow of section 1, the test vehicle was always forced to stop. The stopped times were less at section 3 with the semi-actuated signal than at any other signal. In section 11 the test vehicle encountered fewer stopped times in the northwest flow, because there was a 10-sec advance green time for left turns and through movements in that direction.

The average delays per vehicle for both bypass approaches to each intersection included in the multivariate analysis were computed by the two methods described. These total delays, including both stopped and running delays, are given in Table 6. The delays computed were very similar. A hypothesis test was performed to determine whether the mean of the differences of the computed and the theoretical mean delays at each

TABLE 5

AVERAGE STOPPED TIMES, INTERRUPTED FLOW

Section	SE Flow			NW Flow		
	Avg. Stopped Time per Run (sec)	Avg. Length of Stop (sec)	Percent of Runs When Stops Occurred	Avg. Stopped Time per Run (sec)	Avg. Length of Stop (sec)	Percent of Runs When Stops Occurred
1[a]	5.3	5.3	100.0	—	—	—
3	3.7	12.4	30.0	4.1	15.3	27.5
8	10.0	16.6	60.0	8.1	15.0	52.5
11	12.1	18.7	65.0	4.2	10.5	40.0
13	4.8	11.4	42.5	5.7	12.8	45.0
15	9.2	17.5	52.5	8.6	16.5	55.0
17[a]	5.3	16.4	32.5	8.0	16.3	60.0
18[a]	8.8	17.6	50.0	15.8	19.6	72.5

[a]Not included in the multivariate analysis.

TABLE 6

AVERAGE DELAYS, INTERRUPTED FLOW

Section	Avg. Delay per Vehicle (sec)			
	SE Flow		NW Flow	
	Calculated	Theoretical	Calculated	Theoretical
3	7.0	6.4	7.4	7.9
8	11.0	15.7	15.1	12.9
11	15.5	16.4	8.3	8.5
13	8.3	7.9	10.6	8.9
15	13.5	14.2	13.0	12.7

approach was equal to zero. The hypothesis was accepted at a 5 percent level of significance. Therefore, the results of the two computational methods did not differ significantly for each intersection.

Factor Analysis

The correlation matrix including variables 46 to 93 was computed and examined. Variables 53, 57, 59, 69, and 70 and the dependent variables 92 and 93 were deleted, and the resultant matrix was factorized by the principal-axes method. The factor analysis reduced the 41 variables to 11 factors which accounted for 90 percent of the total variance of the variables.

An examination of the rotated-factor matrix permitted the identification of each factor. The following identified factors are listed with their important component variables and respective coefficients.

High Through Volume on Major Street—This factor describes a signal designed to handle a predominantly through movement of traffic for the major direction of flow:

55. Intersecting streets on both sides, -0.9117;
62. Cycle length, +0.6592;
63. Green time per cycle, +0.8961;
64. Practical approach capacity, +0.8350; and
89. Green to cycle ratio, +0.7013.

Off-Peak Period—This condition indicates an off-peak volume period of the day:

79. Thursday, +0.5827;
80. Friday, -0.4199;
81. 8:00 to 10:00, +0.5865;
84. 3:01 to 6:00, -0.7629;
85. Approach volume, -0.8230;
86. Opposing volume, -0.7167;
87. Total intersection volume, -0.8031; and
91. Approach volume to capacity ratio, -0.8525.

Flat Topography—This factor describes a level type of topography:

51. Approach grade, -0.6335; and
52. Exit grade, -0.3926.

Commercial Development—A high degree of commercial development adjacent to the intersection is indicated by this grouping of variables:

58. Access drives on both sides, +0.7022;
61. Commercial establishments on both sides, +0.7244; and
68. Regulatory signs, +0.5504.

Low Minor-Street Traffic—This factor describes an intersection with a relatively minor street intersecting the major traffic flow:

46. Semi-actuated signal, +0.8646;
62. Cycle length, -0.6240;
87. Total intersection volume, -0.2913; and
90. Approach to total volume ratio, +0.4257.

Concentrated Turning Movements—This factor indicates a large percentage of turning movements from both streams of the major traffic flow to the right side of the direction of travel of the test vehicle:

71. Left turns from directional travel, -0.7392;
72. Right turns from directional travel, +0.8801; and
73. Left turns from opposing travel, +0.8243.

Time Variations—Variations in the times and days when the data were recorded are reflected in this factor, which is not completely defined:

78. Wednesday, -0.8220;
79. Thursday, +0.5977;
81. 8:00 to 10:00, +0.4812; and
83. 12:01 to 3:00, -0.7767.

Vertical Resistance—This group describes the vertical alignment affecting the traffic flow:

50. Length of exit merge lane, +0.7288;
51. Approach grade, +0.6978; and
52. Exit grade, +0.7365.

Long-Distance Travel—Through traffic traversing the entire length of the bypass is reflected in this factor.

81. 8:00 to 10:00, -0.3519;
82. 10:01 to 12:00, +0.8699;
84. 3:01 to 6:00, -0.4207;
88. Commercial vehicles, +0.4160; and
90. Approach to total volume ratio, +0.3943.

Day-of-Week Variations—The variation in days for which travel times were obtained contribute to this partially defined factor:

76. Monday, +0.8456;
78. Wednesday, -0.2492; and
80. Friday, -0.6065.

Other Day-of-Week Variations—Further variations within the week are evident in this group:

77. Tuesday, -0.9226;
79. Thursday, +0.2653; and
80. Friday, +0.3217.

After the factor-score matrix was computed, the factors were correlated with both mean travel speed and mean delay (Table 7). The same three factors were significant in accounting for the variations of both dependent variables. These factors, which were off-peak period, flat topography, and low minor-street traffic, were associated with increased speeds and decreased delays. Multiple linear regression equations were developed to predict travel speed and delay from these significant factors. The following relationship was derived to estimate travel speed for interrupted flow:

$$S_3 = 24.16 + 10.186\,(0.2022F_0 + 0.1404F_P + 0.2626F_R) \qquad (11)$$

where

S_3 = mean travel speed, mph;
F_0 = off-peak period;

TABLE 7

CORRELATION OF MEAN TRAVEL SPEED AND DELAY WITH FACTORS, INTERRUPTED FLOW

Factor	Correlation Coefficient	
	Travel Speed	Delay
N	-0.0278	-0.0646
O	+0.2022[a]	-0.1455
P	+0.1404[a]	-0.1778[a]
Q	-0.0703	+0.0470
R	+0.2626[a]	-0.2044[a]
S	-0.0194	+0.0399
T	+0.0137	+0.0120
U	-0.0540	+0.0224
V	-0.0413	+0.0164
W	+0.0567	-0.0636
X	+0.0388	-0.0583

[a] Significant at the 5 percent level.

F_P = flat topography; and
F_R = low minor-street traffic.

The degree of correlation of this equation was expressed by a multiple correlation coefficient of 0.360. Approximately 12 percent of the total variation in travel speed was reflected in the three significant factors. The standard error of estimate was 9.49 mph.

Delay was related to the significant factors by the following:

$$D_1 = 16.49 + 14.23\,(-0.1455F_0 - 0.1778F_P - 0.2044F_R) \qquad (12)$$

where

D_1 = mean delay, sec;
F_0 = off-peak period;
F_P = flat topography; and
F_R = low minor-street traffic.

The multiple correlation coefficient of 0.307 measured the degree of linear association between delay and the three significant factors. The three factors explained only 9 percent of the total variation in delays. An index of precision was provided by the standard error of estimate of 13.54 sec.

The significant factors were evaluated in terms of the original study variables. The following multiple linear regression equations were developed in standard-score form to express these factors:

$$F_0 = 0.1177Z_{79} - 0.1225Z_{80} + 0.1969Z_{81} - 0.1390Z_{84} - 0.1907Z_{85} - 0.1200Z_{86} - 0.1416Z_{87} + 0.1514Z_{88} - 0.2080Z_{91} \qquad (13)$$

$$F_P = -0.1765Z_{48} - 0.1406Z_{49} - 0.1765Z_{66} - 0.1690Z_{75} \qquad (14)$$

$$F_R = 0.2790Z_{46} + 0.1080Z_{60} - 0.1904Z_{62} + 0.1265Z_{64} - 0.2305Z_{67} + 0.1071Z_{68} - 0.1234Z_{71} + 0.1608Z_{89} + 0.1694Z_{90} \qquad (15)$$

where

F_j = common factor, and
Z_i = standard score of variable.

The standard scores of each variable are computed from Eq. 9.

Multiple Linear Regression and Correlation Analysis

Multiple linear regression equations were developed to estimate travel speeds and delays for interrupted flow as functions of the significant variables. The techniques for deriving these relationships were similar to the standards followed in the uninterrupted flow analysis.

The multiple linear equations expressing overall travel speed and delay as functions of the significant variables are given in Table 8. The speed relationship has the following form:

$$S_4 = 28.595 - 0.4165X_{51} - 0.2118X_{62} - 0.0120X_{85} - 0.0170X_{87} + 29.4800X_{89} \qquad (16)$$

where

S_4 = mean travel speed, mph;

TABLE 8

MULTIPLE LINEAR REGRESSION AND CORRELATION ANALYSIS, INTERRUPTED FLOW

Variable	Net Regression Coefficient	Standard Error
	Part I[a]	
51	-0.4165	0.3235
62	-0.2118	0.0587
85	-0.0120	0.0280
87	-0.0170	0.0104
89	+29.4800	7.4789
	Part II[b]	
49	+0.0052	0.0024
62	+0.2299	0.0833
85	+0.0135	0.0401
87	+0.0168	0.0154
89	-35.7935	12.7107

[a]Dependent variable: travel speed; intercept = 28.59 mph; multiple correlation coefficient = 0.368; and standard error of estimate = 9.53 mph.
[b]Dependent variable: travel delay; intercept = 11.95 sec; multiple correlation coefficient = 0.326; standard error of estimate = 13.544 sec.

X_{51} = average algebraic grade of approach, percent;
X_{62} = cycle length of traffic signal, sec;
X_{85} = traffic volume approaching the intersection in the direction of travel, vehicles per 15 min;
X_{87} = total traffic volume entering the intersection on all four approaches, vehicles per 15 min; and
X_{89} = green time to cycle length ratio.

The degree of linear correlation was indicated by a multiple correlation coefficient of 0.368. The significant variables (approach grade, cycle length, approach volume, total intersection volume, and green-to-cycle ratio) accounted for 14 percent of the variation in travel speeds. All variables except green time to cycle length ratio were negatively related to travel speeds. The reliability of the estimate was expressed by a standard error of 9.53 mph.

The following multiple linear regression equation for travel delay was evolved:

$$D_2 = 11.951 + 0.0052X_{49} + 0.2299X_{62} + 0.0135X_{85} + 0.0168X_{87} - 35.7935X_{89} \quad (17)$$

where

D_2 = mean travel delay, sec;
X_{49} = length of approach to special turning lane, ft;
X_{62} = cycle length of traffic signal, sec;
X_{85} = traffic volume approaching the intersection in the direction of travel, vehicles per 15 min;
X_{87} = total traffic volume entering the intersection on all four approaches, vehicles per 15 min; and
X_{89} = green time to cycle length ratio.

The correlation coefficient of 0.326 measured the degree of the functional relationship of the variables. Approximately 11 percent of the variability in delay was explained by the independent variables. The variables of length of approach to turning lane, cycle length, approach volume, and total intersection volume were correlated with delay in a positive manner, while the green time to cycle length ratio had a negative relationship. The standard error of estimate was 13.54 mph. The sign of the regression coefficient of the length of approach to turning lane variable was contrary to expectation. The plus sign indicated that delay increased as the length of the approach increased in combination with the other variables in the model. The length of the approach, however, was interrelated with a high-volume intersection and with a relatively high number of turning movements. These conditions contributed to the increased delays.

The multiple correlation coefficients of these two regression equations were lower for the analysis of the interrupted flow vs those for the uninterrupted flow. Overall travel speeds and delays at signalized intersections depended greatly on whether or not the vehicle was required to stop. This condition of chance was not accounted for in the analysis. In addition, those variables which were significant in the final models exhibited little variation among the study intersections. The unexplained variability with individual drivers was again evident in the analysis.

SUMMARY OF RESULTS AND CONCLUSIONS

The following conclusions were derived from the results of the multivariate analyses of overall travel speeds and delays on the U.S. 52 Bypass located in Lafayette, Indiana. The movements of traffic on the bypass were classified into two categories. Uninterrupted flow was distinguished from interrupted flow at signalized intersections where traffic was required to stop for a red signal. These conclusions are valid only for the flow of traffic on the bypass, but the findings also serve as generalizations of the significant determinants of travel speeds and delays on similar type facilities.

1. The overall travel speeds of the uninterrupted-flow portions of the bypass were influenced by four significant factors. Commercial development, urban development, and stream friction were negatively related to speed; the remaining factor, rural development, was associated with travel speed in a positive manner. Commercial development accounted for 30 percent of the variation in travel speed.
2. Five variables were significant in the prediction of mean overall travel speeds for the uninterrupted flow sections. These variables, which were total number of street intersections per mile, total number of commercial establishments per mile, percent of section where passing was not permitted, practical capacity, and total volume, were all negatively related with travel speed.
3. For the interrupted-flow portions the factors which significantly explained both overall travel speeds and delays were off-peak period, flat topography, and low minor-street traffic. These three factors were associated with increased travel speeds and decreased delays.
4. The variables of cycle length, traffic volume approaching the intersection in the direction of travel, and total intersection volume contributed to decreased speeds and increased delays. The green time to cycle length ratio accounted for significant variations in travel speeds and delays in a positive and negative manner, respectively. The approach grade of the intersection was negatively related to speed, and the length of the approach to the turning lane was positively associated with delay.
5. Multiple linear regression equations were developed to estimate mean travel speeds and delays from the significant factors and variables for both flows. Approximately 50 percent of the variation in speed of uninterrupted flow was explained and 10 to 15 percent of the variation in travel speeds and delays at signalized intersections was accounted for. The reliability of these relationships was limited by the unknown effects of driver behavior which was not included in the analysis. In addition, delays at traffic signals were largely dependent on whether or not a stop occurred.

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Non-Integer Car-Following Models

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•SEVERAL authors have originated and developed models to describe the traffic flow on highways. There are at least two approaches to this problem. The microscopic approach, sometimes referred to as the car-following theory, takes as its elements individual vehicular spacing and speed. The macroscopic approach deals with traffic-stream flows, densities, and average speeds. In recent years it has been shown that the two approaches are interrelated.

This paper consists of four major parts. First, a brief background is given of microscopic and macroscopic theories of traffic flow, with special emphasis on their interrelationship. Second, a comprehensive matrix is developed which results in a set of steady-state flow equations, which includes the major macroscopic and microscopic theories. Third, analytical techniques are developed for evaluating the various theories on the basis of experimental data. The last section deals with the investigation of a continuum of non-integer car-following models for the development of deterministic flow models, which describe interrelationships between flow characteristics.

BACKGROUND

The challenge to describe vehicular flow in a microscopic manner led Reuschel (1) and Pipes (2) to formulate the phenomena of the motion of pairs of vehicles following each other by the expression

$$x_n - x_{n+1} = L + S(\mathring{x}_{n+1}) \tag{1}$$

This relation can be derived from Figure 1. In this formulation it is assumed that each driver maintains a separation distance proportional to the speed of his vehicle ($\mathring{x}_{n+1}$) plus a distance L. The factor L is the distance headway at standstill ($\mathring{x}_n = \mathring{x}_{n+1} = 0$), including the length of the lead vehicle. The constant S has the dimension of time, and the differentiation of Eq. 1 gives

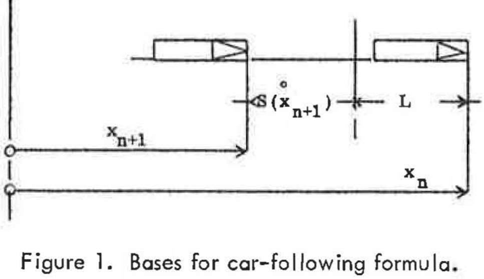

Figure 1. Bases for car-following formula.

$$\overset{\circ\circ}{x}_{n+1} = \frac{1}{S}[\mathring{x}_n - \mathring{x}_{n+1}] \tag{2}$$

This differential equation is generally referred to as the basic equation of the car-following models.

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It is the basic stimulus-response relation which was further investigated by a research group from General Motors Corp. (3). Their linear mathematical model showed surprisingly good results for high-density conditions when tested against actual data:

$$\ddot{x}_{n+1}(t + T) = \lambda [\dot{x}_n(t) - \dot{x}_{n+1}(t)] \tag{3}$$

where T = the time lag of response to the stimulus.

This formulation was refined in 1959 by Gazis, Herman, and Potts (4) by letting the sensitivity factor, λ, be inversely proportional to the distance of separation (distance headway):

$$\lambda = \frac{a_1}{x_n(t) - x_{n+1}(t)} \tag{4}$$

$$\ddot{x}_{n+1}(t + T) = \frac{a_1}{x_n(t) - x_{n+1}(t)} [\dot{x}_n(t) - \dot{x}_{n+1}(t)] \tag{5}$$

In 1961, Gazis, Herman, and Rothery (5) proposed a more general expression for the sensitivity factor, λ:

$$\lambda = a \frac{\dot{x}^m_{n+1}(t + T)}{[x_n(t) - x_{n+1}(t)]^\ell} \tag{6}$$

The general expression for these microscopic theories thus becomes

$$\ddot{x}_{n+1}(t + T) = a \frac{\dot{x}^m_{n+1}(t + T)}{[x_n(t) - x_{n+1}(t)]^\ell} [\dot{x}_n(t) - \dot{x}_{n+1}(t)] \tag{7}$$

It can be seen that when $m = 0$ and $\ell = 0$, the general equation becomes Eq. 3, while the condition $m = 0$ and $\ell = 1$ converts the general equation to Eq. 5. Eq. 7 and the exponents m and ℓ will be shown to be significantly important in later portions of this paper.

Macroscopic theories of traffic flow date back to 1935. Greenshields (6), after inspection of a set of speed-density measurements, hypothesized that a linear relationship existed between speed and density:

$$u = u_f [1 - \frac{k}{k_j}] \tag{8}$$

where

u_f = free-flow speed,
k_j = jam density,
u = speed, and
k = density.

Based on the developments of Lighthill and Witham (7), Greenberg (8), in 1959, proposed a macroscopic flow model by using the analogy of the traffic-flow situation with the problem of one-dimensional fluid flow. By using the equation of continuity and the equation of motion, a relationship between speed and density was developed:

$$\frac{\partial k}{\partial t} + \frac{\partial q}{\partial x} = 0 \tag{9}$$

$$\frac{du}{dt} = -c^2(\frac{1}{k})\frac{\partial k}{\partial x} \tag{10}$$

$$u = c \ln(\frac{k_j}{k}) \tag{11}$$

where $c = u_0$ = speed at maximum flow.

In 1960, Underwood (9) proposed an exponential speed-density relation:

$$u = u_f e^{-k/k_0} \tag{12}$$

where k_0 = density at maximum flow.

Edie (10) after careful study of q - k curves, hypothesized that there were two regimes of traffic flow: free flow and congested flow. He proposed that an exponential speed-density relation be used for the free-flow regime, Eq. 12, and the Greenberg equation be used for the congested-flow regime, Eq. 11.

Using the fluid-flow analogy approach proposed earlier by Greenberg but employing a more general derivation of this problem, Drew (11) proposed the following speed-density relation:

$$\frac{du}{dt} = -c^2 k^n \frac{\partial k}{\partial x} \tag{13}$$

$$u = u_f\left[1 - (\frac{k}{k_j})^{\frac{n+1}{2}}\right] \quad \text{for } n > -1 \tag{14}$$

Drake, May, and Schofer (12) report on the application of a bell-shaped curve which gave satisfactory results when compared to speed-density measurements:

$$u = u_f e^{-\frac{1}{2}(\frac{k}{k_0})^2} \tag{15}$$

The speed-density relationships shown in Eqs. 8, 11, 12 as well as two linear regimes, three linear regimes, and a modified Greenberg model were evaluated in this study.

In 1961, a paper (5) of major significance was published that placed emphasis on the steady-state flow equations which result from various microscopic theories of traffic flow. It was shown that several proposed macroscopic theories are mathematically equivalent to the generalized microscopic expression given by Eq. 7, provided proper integers are selected for the exponents m and ℓ. For example, if m and ℓ are assumed to have the values of 0 and 2, respectively, the microscopic expression and Greenshields' macroscopic expression will give identical speed-density relationships. Thus this paper offered the first evidence of the bridge between microscopic and macroscopic approaches of traffic-flow theory.

Further analysis of the interrelationships between the two different approaches was given by Haight (13). Drew also investigated this interrelationship and showed that by setting m = 0 and varying the exponent, ℓ, Eq. 7 can be transformed into the steady-state flow Eq. 14, for $n = 2\ell - 3$ (11).

MATRIX DEVELOPMENT AND RELATIONSHIP OF MACROSCOPIC AND MICROSCOPIC THEORIES

The use of the general expression for the sensitivity factor in the stimulus-response equation (Eq. 6) as formulated by Gazis, Herman, and Rothery (5) gives a very powerful tool for an evaluation of existing models. All previously mentioned models can be described by the generalized equation (Eq. 7) by using appropriate m and ℓ values. Gazis et al have shown that by integration of the generalized equation the following expression is obtained:

$$f_m(u) = c' + cf_\ell(s) \tag{16}$$

where

u = steady-state speed of a stream of traffic,
s = constant average spacing, and
c and c′ = some appropriate constants consistent with physical restrictions.

The integration constant c′ is related to a free speed, u_f, or a jam spacing, s_j, depending on the values of m and ℓ. The jam spacing, s_j, can be transformed to jam density, k_j, by $s_j = 1/k_j$.

By using this general solution of Gazis et al, a matrix of steady-flow equations for different m and ℓ values was developed. The general expressions are shown in Figure 2, and the expressions in macroscopic model format are shown in Figure 3. An inspection of these two matrices reveals that all of the previously reported microscopic and macroscopic models and several other possible models can be located in terms of m and ℓ combinations. For example, the models of Pipes (2), Reuschel (1), and Chandler, Herman, and Montroll (3) are obtained when m = 0 and ℓ = 0. The Greenberg model (8) and the Gazis, Herman, and Potts model (4) is obtained when m = 0 and ℓ = 1. When m = 0 and ℓ = 3/2, the Drew model (11) is obtained. The Greenshields model (6) results when m = 0 and ℓ = 2, whereas the Edie (10) and Underwood (9) model results when m = 1 and ℓ = 2. The bell-shaped curve proposed by Drake, May, and Schofer (12) is obtained when m = 1 and ℓ = 3.

The matrix of m and ℓ values not only shows that the existing traffic-flow models can be reduced to the generalized car-following model, but also that, by choosing particular m and ℓ combinations, a wide variety of shaped curves for the speed-density relation can be selected (Fig. 4). One also can recognize certain trends in the shape of the curves by keeping one of the exponents, m or ℓ, constant. It should be noted that

ℓ \ m	m < 1	m = 1	m > 1
$\ell < 1$	$u^{1-m} = ck_j^{\ell-1} + ck^{\ell-1}$	Boundary conditions not satisfied	$u^{1-m} = u_f^{1-m} + ck^{\ell-1}$
$\ell = 1$	$u^{1-m} = c\ell n(1/k_j) + c\ell n(1/k)$	$\ell nu = c\ell n(1/k_j) + c\ell n(1/k)$	$u^{1-m} = c\ell n(1/k_j) + c\ell n(1/k)$
$\ell > 1$	$u^{1-m} = ck_j^{\ell-1} + ck^{\ell-1}$	$\ell nu = \ell nu_f + ck^{\ell-1}$	$u^{1-m} = u_f^{1-m} + ck^{\ell-1}$

Figure 2. Matrix of steady-state flow equations for different m, ℓ values in $f_m(u) = c' + cf_\ell(s)$.

ℓ \ m	m = 0	m = 1
$\ell = 0$	$u = \frac{1}{s}\left[\frac{1}{k} - \frac{1}{k_j}\right]$	-
$\ell = 1$	$u = u_0 \ell n \left(\frac{k_j}{k}\right)$	-
$\ell = 1.5$	$u = u_f\left[1 - \left(\frac{k}{k_j}\right)^{1/2}\right]$	-
$\ell = 2$	$u = u_f\left[1 - \left(\frac{k}{k_j}\right)\right]$	$u = u_f e^{-(k/k_0)}$
$\ell = 3$	-	$u = u_f e^{-\frac{1}{2}(k/k_0)^2}$

Figure 3. Matrix of existing traffic-flow models.

non-integer m and ℓ values can be utilized, and consequently an expression can be determined which more closely represents actual speed-density relations. This is shown in Figure 5, where for a constant exponent m = 1 the exponent ℓ is changed in steps of 2/10. One can see the gradual change from the exponential model (m = 1, ℓ = 2) to a bell-shaped model (m = 1, ℓ = 3).

In examining the matrix, one should remember that the m value is the exponent of the following vehicle's speed $[\dot{x}_{n+1}(t+T)]$ and the ℓ value is the exponent of the spacing of the two vehicles $[x_n(t) - x_{n+1}(t)]$. Consequently, the fundamental difference between models is the weight given to the following vehicle's speed and the spacing between vehicles.

This matrix of m and ℓ values has permitted the development of analytical techniques for evaluating deterministic traffic-flow models using speed-density measurements.

ANALYTICAL PROCEDURE FOR EVALUATING DETERMINISTIC INTEGER AND NON-INTEGER TRAFFIC-FLOW MODELS

The speed-density relation rather than the flow-density or speed-flow relation was selected as the relationship for evaluation. Once this equation is evaluated, the other relationships can be obtained by using the steady-state equation q = uk. The speed-

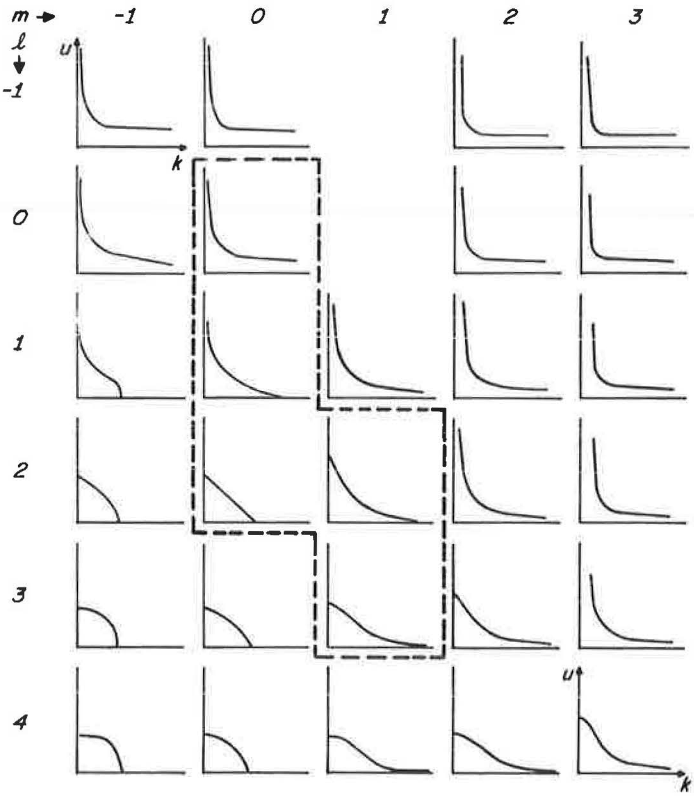

Figure 4. Matrix of speed-density relations for various m, ℓ combinations of the general car-following equation.

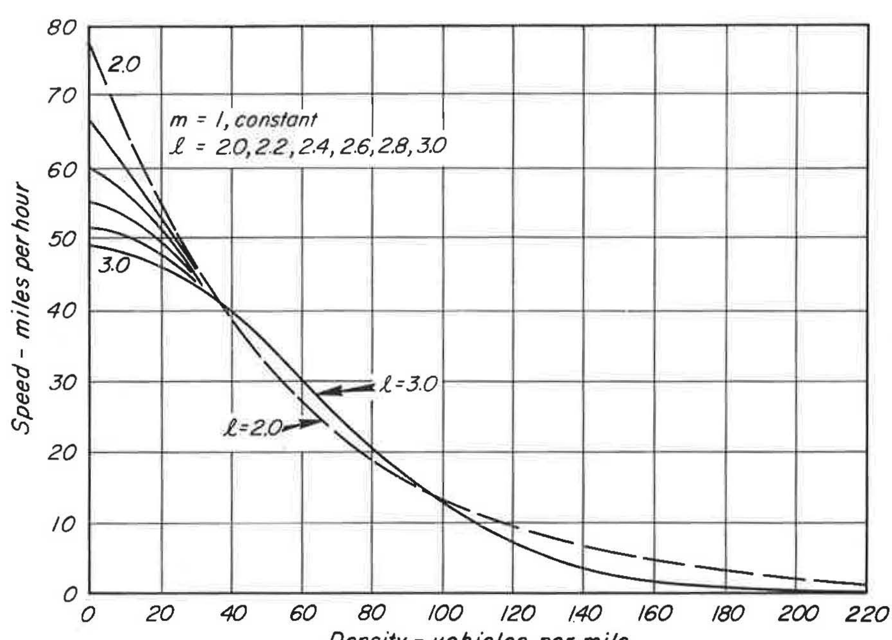

Figure 5. Influence of the use of non-integer exponents on the speed density relation.

density relation has the advantage of being easier to handle mathematically. For example, the equation resulting from the integration of Eq. 7 is in the form of a linear speed-density relationship depending on the scale of the coordinate system.

A variety of different models can be obtained through the selection of different m, ℓ values. To define a specific equation, the 4 parameters m, ℓ, c, and c' must be determined. One method to obtain these parameters is the use of regression analysis or least squares fit to compute c and c', and to determine the parameters m and ℓ by maximizing or minimizing the correlation coefficient or the standard error of estimate in cooperation with close approximation to significant traffic-flow characteristics.

Statistical Procedure of Evaluation

As shown in Figure 4, the speed-density relation is truly a linear equation with a regular coordinate system, when m = 0 and ℓ = 2. For the speed-density relationships with other exponents m and ℓ, the speed, u, and/or density, k, have to be transformed (depending on the parameters m and/or ℓ), if linear regression analysis is to be employed. This is equivalent to contracting or extending the scale on the ordinate and/or abscissa in order to arrange the data points for a linear analysis. The regression analysis is used to obtain an estimate of the dependent variable (in this case, speed, u) from an independent variable (in this case, density, k). The regression equation is determined by minimizing the squared deviations of the data points from the regression curve. However, the correlation coefficient obtained from the regression analysis for the transformed scale (where each speed-density relation is made linear) is not the same as for the real scale (where each speed-density relation except one is not linear).

One possible solution to the problem would be to compute a new correlation coefficient for the real-scale situation similar to the linear correlation coefficient, which was obtained from the transformed-scale situation. However, the linear regression and correlation analysis is based on the supposition that the mean values of the dependent

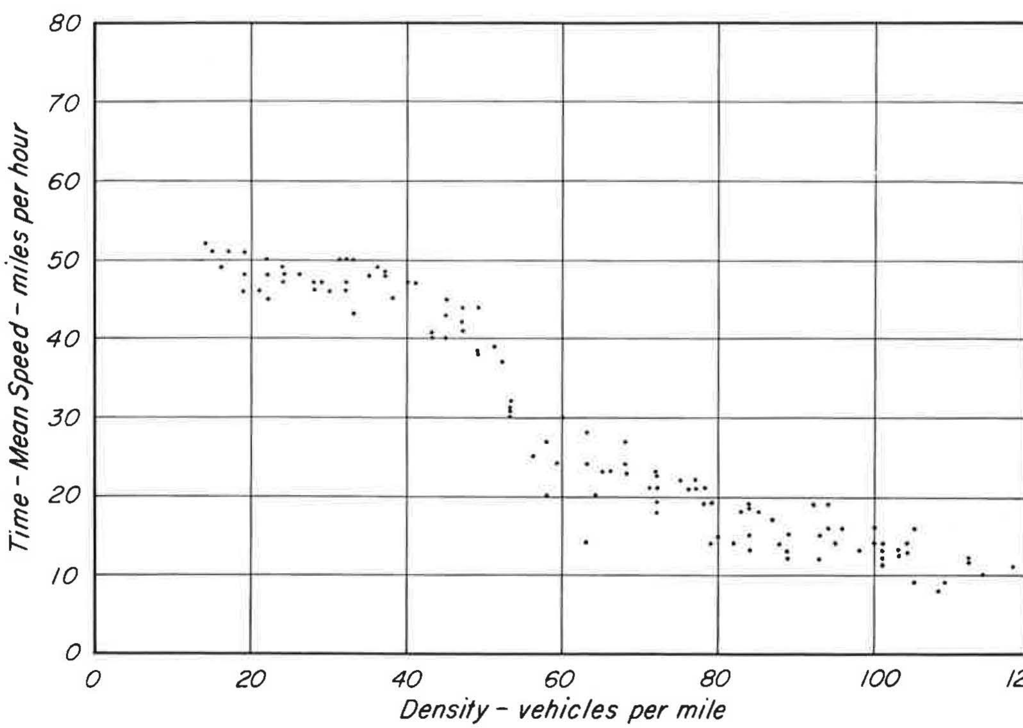

Figure 6. Speed-density data collected on the Eisenhower Expressway.

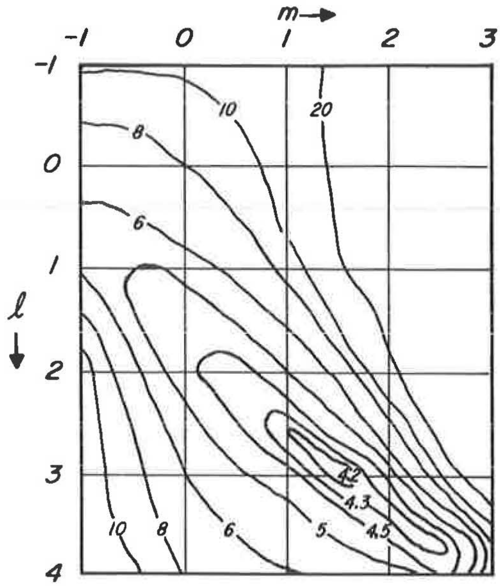

Figure 7. Curves of equal mean deviations (mpm).

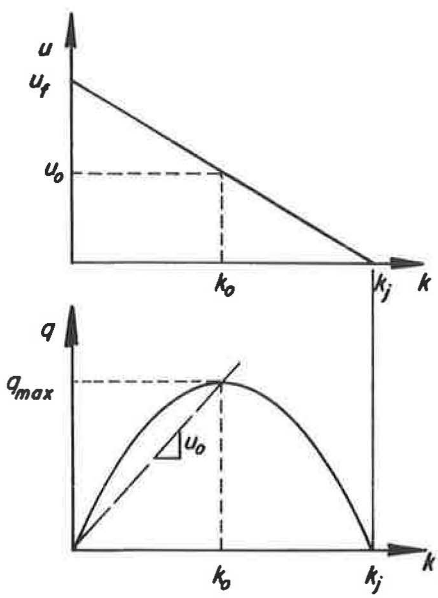

Figure 8. Traffic-flow characteristics shown for the linear speed-density model of Greenshields ($m = 0, \ell = 2$).

and independent variable are one point on the regression line. This is no longer the case for the retransformed regression curve in the real-scale system.

This and other reasons lead to the idea of computing the sum of the squared deviations of the data points from the regression curve for the dependent variable $\Sigma(u - u_{estimated})^2$ and taking this as a measurement of the goodness of fit. By dividing this sum by the number of data points, N, and taking the square root of this value, one obtains a mean deviation, s, for the curve considered.

This method was applied to a set of 118 one-minute samples of time-mean speeds and mean densities recorded with the pilot detection system of the Chicago Area Expressway Surveillance Project. The data were collected in the middle lane of the three-lane westbound roadway on the Eisenhower Expressway, at Harlem Avenue. The highest measured mean speed was 52.3 mph, the lowest 8.2 mph; the highest computed density was 118.4 vpm, the lowest 14.2 vpm (Fig. 6). A justification of using the time-mean speed instead of the space-mean speed with the corresponding density is given by Drake, May, and Schofer (12).

The results were plotted in an m, ℓ plane and values of equal mean deviation were combined to trace a contour map of equal deviations. Figure 7 shows this plot for the most significant values of the exponents m ($-1 \leq m \leq 3$) and ℓ ($-1 \leq \ell \leq 4$). The minimization of the mean deviation leads to an area of m and ℓ values, which gives a first indication about the goodness of fit of selected speed-density relations with particular m and ℓ combinations.

Traffic-Flow Characteristics as Criteria for Evaluation

The statistical procedure, minimizing of the mean deviations, alone is not satisfying. There is very little difference in the mean deviations for different sensitivity factors or m, ℓ combinations. On the basis of these small differences, it would be difficult to give the preference to certain speed-density relations as the best fit to the data with any assurance. The choice of a model or equation only by the criteria of the minimum mean deviation of the data points from the estimated curve may also give misleading results. The chosen equation may fit the data points very nicely, but the speed-density curve may, for example, have no limit value for the free speed. Physical restrictions and field observations, however, indicate that there are limit values for certain traffic-flow characteristics.

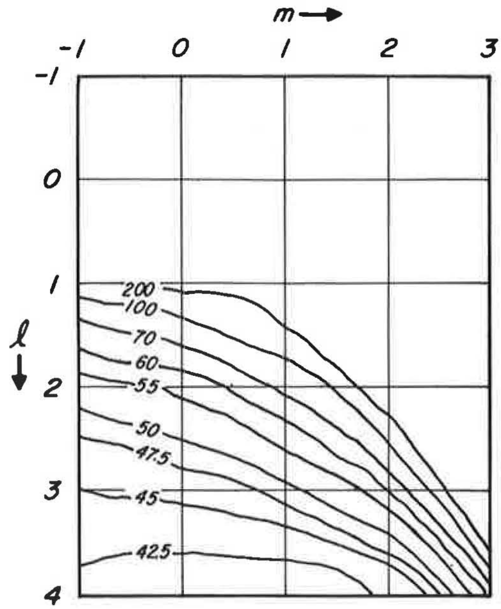

Figure 9. Curves of equal free speed (mph).

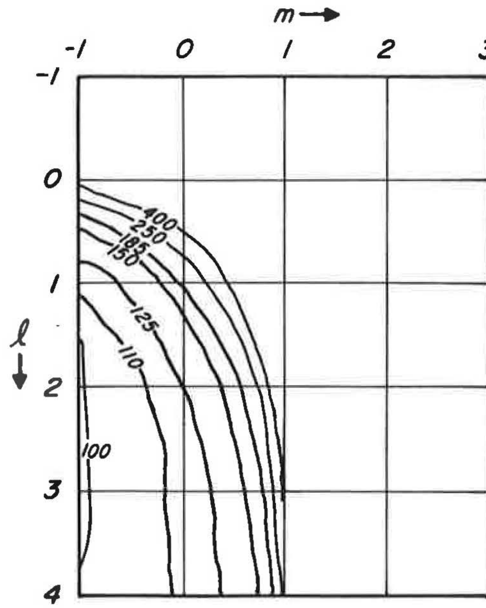

Figure 10. Curves of equal jam density (vpm).

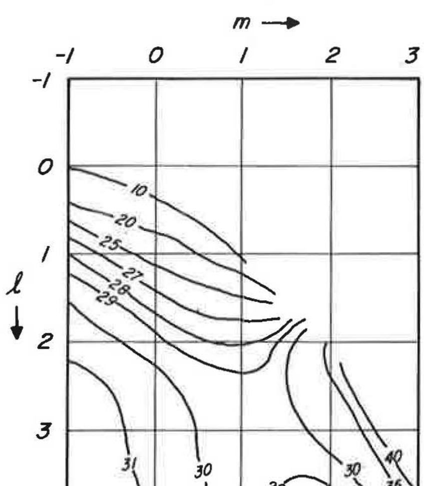

Figure 11. Curves of equal optimum speed (mph).

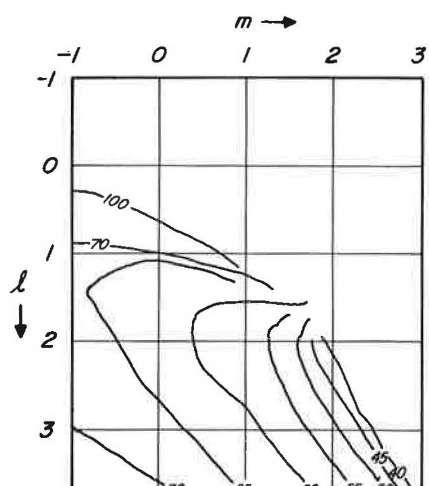

Figure 12. Curves of equal optimum density (vpm).

Important traffic-flow conditions for a speed-density relationship are the free speed, u_f, where the density $k = 0$ and the jam density, k_j, where the speed $u = 0$. Significant points of the flow-density relation are the optimum density, k_0, and the optimum speed, u_0, at which the maximum flow, q_{max}, occurs (Fig. 8). These flow characteristics can be derived from the steady-state flow equation (Eq. 16, see also Fig. 2) by setting the density $k = 0$ to obtain the free speed, u_f, and by setting the speed $u = 0$ to obtain the jam density, k_j. By differentiating the flow-density equation $q = uk$ with respect to the density, k, or the speed, u, and setting the results equal to zero $\partial q/\partial k = 0$ or $\partial q/\partial u = 0$, one

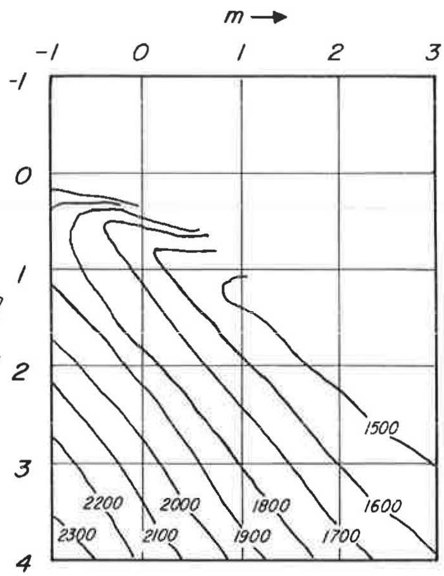

Figure 13. Curves of equal maximum flow (vph).

can obtain the optimum density, k_0, and the optimum speed, u_0. The flow-density relation $q = uk$ gives for $k = k_0$ and $u = u_0$ the maximum flow, q_{max}.

These traffic-flow characteristics, which can be determined for each m, ℓ combination, allow a judgment about the value of the specific model considered and provide a good means of evaluation. For the evaluation, it is helpful to plot the results in an m, ℓ plane and to develop curves of equal levels of flow characteristics. The trends of these curves show how specific m, ℓ combinations or models fit the flow requirements. The model which fits the flow characteristics best can be determined by superimposing the curves of the flow characteristics graphically and by locating in this way that area of m, ℓ combinations which fulfills all or most flow requirements as well as statistical measures of deviation.

This procedure, the use of the flow characteristics as a means of evaluation, was applied to the previously mentioned data from the Eisenhower Expressway. Figures 9 and 10 show the results of the computations for free speed, u_f, and jam density, k_j. The pictures show clearly that finite values for the free speed exist only for $\ell > 1$ and for the jam density only for $m < 1$. The calculations for the other areas give either unrealistic results or, for example, infinite values for the jam density when $m \geq 1$. The curves of equal free speed and/or jam density indicate that only very small bands give reasonable flow characteristics in the m, ℓ plane.

The investigation of the significant points of the flow-density equation is shown in Figures 11, 12, and 13. Realistic values for the optimum density, the optimum speed, and the maximum flow occur only in a limited area of the m, ℓ plane. This area

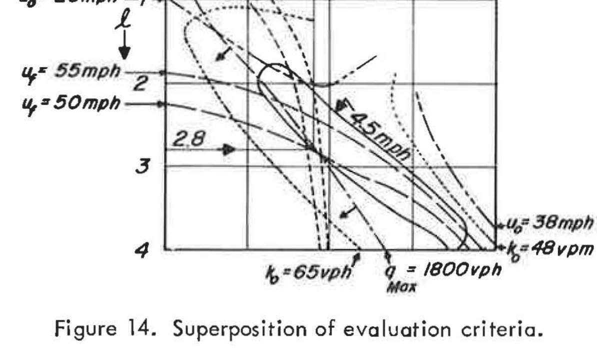

Figure 14. Superposition of evaluation criteria.

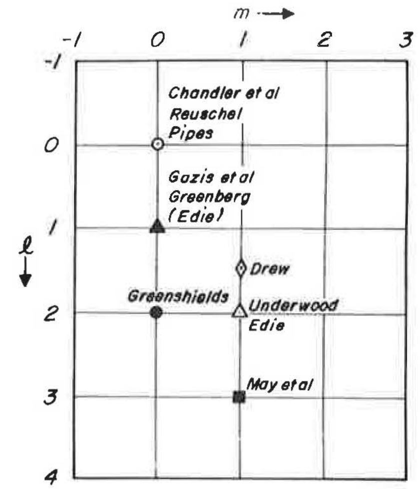

Figure 15. Existing traffic-flow models.

is the part below the bisecting diagonal through the origin of the coordinate system. The curves of equal optimum speed and optimum density having reasonable values cover a relatively wide range of the m, ℓ plane, whereas the curves of equal maximum flow show a trend of increasing flow parallel to the bisecting diagonal.

While the previously mentioned contour maps of traffic-flow characteristics can be used individually to check the applicability of a certain model, the graphical superposition of all contour maps gives a tool to determine those in m, ℓ combinations, which fit best the assumed flow requirements. Figure 14 shows the superposition of these parameters for certain reasonable values. According to the situation at the Eisenhower Expressway, the following values for the flow characteristics have been assumed to be reasonable: (a) free speed, u_f = 50-55 mph; (b) jam density, k_j = 185-250 vpm; (c) optimum speed, u_0 = 28-38 mph; (d) optimum density, k_0 = 48-65 vpm; and (e) maximum flow, q_{max} $\geq$ 1,800 vph. The strongest requirements are imposed by the free speed and the jam density shown as narrow bands on the m, ℓ chart (Fig. 14). The other limitation is given through the requirement of a greater maximum flow rate than 1,800 vehicles per hour. While the optimum density, optimum speed, and mean deviation are of less influence, these three criteria limit the solution to a very small area around m = 0.8 and ℓ = 2.8, where all traffic-flow criteria are fulfilled.

	Selected model for integer solution	Selected model for non-integer solution
m	1.0	0.8
ℓ	3.0	2.8
mean deviation	4.6 mph	4.5 mph
free speed	48.7 mph	50.1 mph
jam density	∞	220 vpm
optimum speed	29.5 mph	29.6 mph
optimum density	60.8 vpm	61.1 vpm
maximum flow	1,795 mph	1,810 mph
macroscopic equation	$u = u_f e^{-\frac{1}{2}\left(\frac{k}{k_0}\right)^2}$	$u = u_f \left[1 - \left(\frac{k}{k_j}\right)^{1.8}\right]^5$
microscopic equation	$\ddot{x}_{n+1}(t + T) = (a) \frac{\dot{x}_{n+1}(t + T)^1}{[x_n(t) - x_{n+1}(t)]^3} [\dot{x}_n(t) - \dot{x}_{n+1}(t)]$	$\ddot{x}_{n+1}(t + T) = (a) \frac{\dot{x}_{n+1}(t + T)^{0.8}}{[x_n(t) - x_{n+1}(t)]^{2.8}} [\dot{x}_n(t) - \dot{x}_{n+1}(t)]$
(a) value	1.35×10^{-4}	1.33×10^{-4}

Figure 16. Comparison of the results for the selected integer and non-integer traffic-flow models.

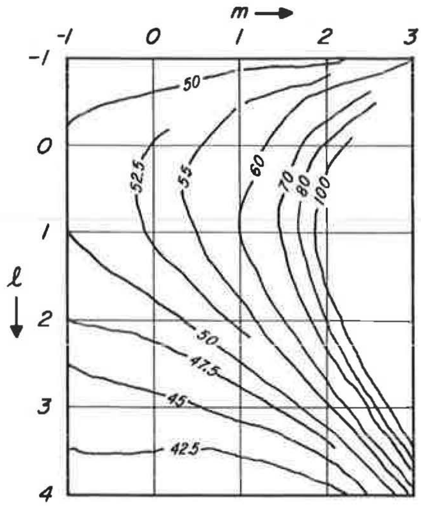

Figure 17. Curves of equal speed (mph) at density $k = 20$ vpm.

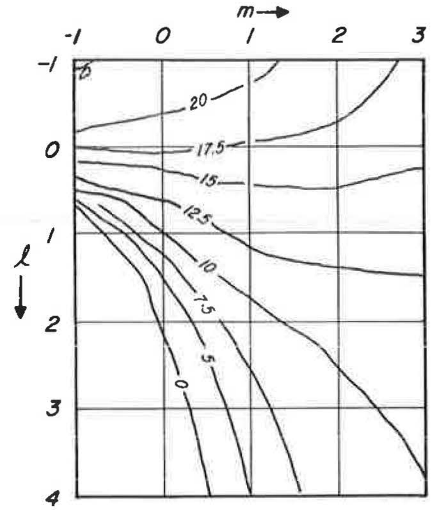

Figure 18. Curves of equal speed (mph) at density $k = 120$ vpm.

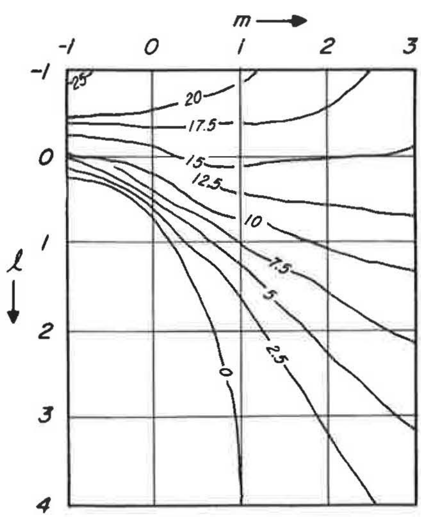

Figure 19. Curves of equal speed (mph) at density $k = 220$ vpm.

The superposition of the evaluation criteria in Figure 14 shows that the use of a continuum of non-integer m, ℓ combinations allows a very good adjustment to the different levels of the evaluation requirements. The speed-density equation is

$$u = u_f [1 - (\frac{k}{k_j})^{1.8}]^5$$

where
u_f = 50.1 mph, and
k_j = 220 vpm.

Figure 15 gives, for reasons of comparison, the location of existing traffic-flow models in the m, ℓ plane.

It can be seen that of the existing models tested with the data set, the bell-shaped curve for m = 1, ℓ = 3 shows the best fit to the criteria. A comparison of the selected model for the integer and non-integer m, ℓ combination is made in Figure 16. The advantage of the selected non-integer model over the integer model for the tested data set is that it has a finite jam density. It also has a slightly higher free speed and maximum flow.

As shown before, the free speed and the jam density require a very strong limitation in the application of certain models. An investigation of the speeds near these limit values has been undertaken to extend the procedure of evaluation (Figs. 17, 18, 19). Figure 17 shows that the area limited by the levels of acceptable free speed in Figure 14 can be considerably extended, if one takes the speed at low densities, for example, k = 20 vpm as criterium. A similar effect can be shown for speeds at densities of k = 120 vpm (Fig. 18). The investigation of speeds which occur at the density close to the assumed jam density, here 220 vpm, shows the contour map in Figure 19. If one allows a speed of 2 or 3 mph at this assumed density, the area limited by the jam density levels in Figure 14 can be extended to larger values of m (2.0 to 2.5). This allows the inclusion of more models, which still fulfill all flow characteristic criteria requirements, except for the desired maximum flow rate. But, as has been mentioned before, the maximum flow is an important criterium, and it cannot be neglected.

The superposition of the criteria of the flow characteristics with that of the statistical analysis is also shown in Figure 14. A mean deviation of 4.5 mph was considered as the limit for the evaluation procedure. The graph shows that the minimization of the mean deviation already gave a good means of evaluation. This is especially true if the extended criteria shown in Figures 16 and 18 are included. The area indicated by the statistical criterium coincides with all of the criteria for the flow characteristics. This stresses that already very small differences in the mean deviations investigated are of great significance. The only criterium which is not included is the maximum flow rate of more than 1,800 vph. The curve of 1,800 vph is just at the border of the 4.5-mph mean deviation curve. This justifies the use of the flow characteristics criteria as a means of evaluation, because only in this way is attention given to the desired and reasonable shape of the speed-density relationship.

SUMMARY

Study Results

This paper describes a procedure on how deterministic microscopic and macroscopic traffic-flow models can be evaluated. It has been shown that all reported macroscopic and microscopic traffic-flow models can be reduced to the general car-following equation (Eq. 7) by selection of appropriate exponents m and ℓ, representing the influence of the speed of the following vehicle, respecitvely, of the distance headway between vehicles on the sensitivity factor. The use of an m, ℓ matrix gives the possibility of comparison of existing models due to given criteria. The m, ℓ plane is the basis for the method of evaluation applied. Two different procedures have been used. The statistical analysis is based on the minimizing of the mean deviations of the data points from the determined regression curve. The preference to this method was given because of its clearness. Although there are very little differences in the mean deviations for particular m, ℓ combinations, this method has been shown to be very effective. The other procedure introduces the traffic-flow characteristics as evaluation criteria. This provides a very helpful tool, because certain physical restrictions or limitations of the flow characteristics are considered as a control. The graphical superposition of the results of both evaluation procedures allows a judgment about the goodness of fit of existing traffic-flow models to given criteria and an estimate of those m, ℓ combinations (or models) which best fit the investigated data. The introduction of a continuum of non-integer exponents m and ℓ implies a considerably greater variety of possible models and a more flexible adjustment to the assumed criteria of evaluation.

The combination of m = 0.8 and ℓ = 2.8 fulfills all requirements of the assumed evaluation criteria, mean deviation, free speed, jam density, optimum speed, optimum density, and maximum flow. The evaluation indicates that the area around the line between m = 0.5, ℓ = 2.5 and m = 2.5, ℓ = 3.5 covers models of very good fit, but with a maximum flow rate of less than 1,800 vph. Models with a maximum flow rate greater than 1,800 vph appear in the area below this line. As can be seen from Figure 4, the speed-density relation tends to be bell-shaped in that area.

The investigation of the flow-density relation in these optimum combinations of m, ℓ values indicates a unique shape of the flow-density curve, i.e., in the high-density regime where the relationship exhibits a reversed curve.

The analysis shows that the general car-following equation implies a series of models in addition to known existing models, which describe actual data quite well. An extension to the area of existing models in the m, ℓ matrix can be proposed mainly for ($1 \leq m \leq 2.5$) and ($2.5 \leq \ell \leq 4.0$).

Future Directions

The use of electronic computers facilitates the procedure of evaluation considerably and an evaluation program has been written in Fortran language. The application of a plotter would have been helpful for the tracing of the contour maps.

The use of the developed evaluation procedure opens a wide field for the application of this method to other typical data sets. The evaluation of data from highways, tunnels, or urban streets may result in different m, ℓ combinations. In this way one can determine areas in the m, ℓ plane which are characteristic for certain types of traffic facilities.

The study should also be extended to the evaluation of multi-regime systems. Several authors have mentioned that these give a better description of actual data for the speed-density relationship.

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A Digital Simulation of Car Following and Overtaking

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A model representing the single-lane no-passing driving situation was formulated and run on a digital computer. Although the model involves the use of a car-following equation, the simulation also includes human factors, such as reaction time lag, driver sensitivity, and the threshold of detection of relative velocity. The model allows for variation of these characteristics both between drivers and over time for each individual driver.

The study was directed to the accident prevention problem with the aim of determining the critical parameters of the driving situation, and of ascertaining the ranges of values of these parameters which define a safe or stable driving situation.

•IN the past 10 years digital simulation of automobile traffic has been increasingly employed as a research technique. The bulk of the work has been done either on simulations of urban traffic with emphasis on the intersection problem, or on freeway simulations emphasizing problems of ramp design.

Some simulations of car following have been carried out, notably in the work of Helly (5) which was aimed at the study of shock-wave development in tunnels, and in the more recent work of Howat (6), and Todosiev (10).

The simulation of car following and overtaking discussed in this report was done especially to investigate stable and unstable behavior with regard to accident-producing situations. It seems clear that a large proportion of accidents on freeways are rear-end collisions caused either by tailgating or too rapid overtaking. In this study, single-lane driving is simulated by a model that includes individual human factors and which is flexible enough to represent a variety of situations. In this way the simulation should result in an identification of the principal factors and parameters influencing safe or stable driving behavior, and a determination of the ranges of parametric values which will exclude accident occurrence.

CAR-FOLLOWING MODELS

A car-following model is essentially some form of a stimulus-response equation where the response (acceleration or deceleration) of the driver is determined by a stimulus function involving the relative velocity between his car and the car ahead, their relative spacing, the absolute velocity level, the driver's sensitivity, and many other factors, human, mechanical, and environmental.

Early car-following models were based on a simple linear relation with driver response dependent only on relative velocity. Thus (Fig. 1), the acceleration or deceleration response of the (n + 1) st car was approximated by

$$\frac{dx^2_{n+1}}{dt^2} = \lambda\left(\frac{dx_n}{dt} - \frac{dx_{n+1}}{dt}\right) = \lambda(v_n - v_{n+1}) \qquad (1)$$

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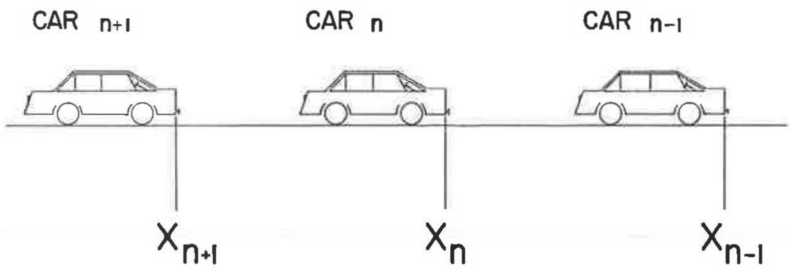

Figure 1. Car-following model.

where

λ = driver sensitivity factor (i.e., intensity of response);
x_n = position of car n; and
v_n = velocity of car n.

Later it became clear that response depended on relative spacing (inversely), on velocity level and, rather critically, on the time lag in driver response, say, T. Consequently, Herman (3), Edie (2), and others studied models of the form

$$\frac{d^2x_{n+1}(t)}{dt^2} = \alpha v_{n+1}(t) \frac{(v_n(t-T) - v_{n+1}(t-T))}{(x_n(t-T) - x_{n+1}(t-T))^p} \tag{2}$$

where

α = new sensitivity factor (λ in Eq. 1 is replaced by $\dfrac{\alpha v_{n+1}}{(x_n - x_{n+1})^p}$ in Eq. 2), and
p = usually 1 or 2.

Thus, in Eq. 2 the driver of the (n + 1)st car accelerates or decelerates an amount which depends on his current velocity, and on the velocity difference and separation between his car and the next at some previous instant. The lag, T, is assumed to include the times for perception, decision, and response; it has been used in the same way implicitly by other investigators.

In actual driving experience people do not, of course, drive continuously in strict adherence to such rules. As Michaels (7) has discussed, a driver cannot even detect relative velocity until the rate of change of angular motion of the image across the retina assumes some minimum threshold value. From Figure 2, the rate of change of angle, $\frac{d\theta}{dt}$, can be determined from the equation

$$L\theta \cong W \text{ or } \theta \cong \frac{W}{L}$$

so that

$$\frac{d\theta}{dt} \cong -\frac{W}{L^2}\frac{dL}{dt}$$

Then, since L is relative spacing,

$$\left|\frac{d\theta}{dt}\right| \cong W \left|\frac{(v_n - v_{n+1})}{(x_n - x_{n+1})^2}\right| \tag{3}$$

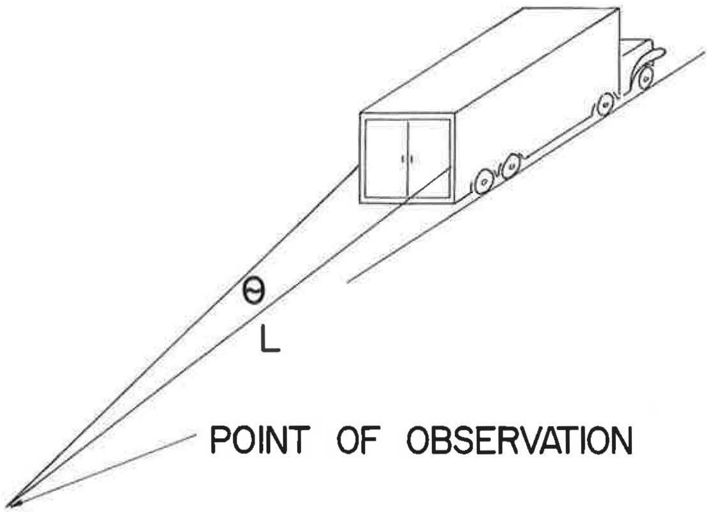

Figure 2. Angular velocity detection.

It has been shown (8) that $\left|\frac{d\theta}{dt}\right|$ is below the threshold of relative velocity detection until the quantity

$$W\left|\frac{v_n - v_{n+1}}{(x_n - x_{n+1})^2}\right| \tag{4}$$

exceeds approximately 6×10^{-4} (for x_n and W in feet and v_n in feet per sec).

Comparison of Eq. 3 with Eq. 2 reveals the extremely interesting fact that the two equations are in agreement for the case p = 2, the value recommended by Edie. It is certainly one of the most intriguing results of the current investigation to note that Eq. 3, based on human factor threshold studies, supports Eq. 2 which has been derived quite differently from empirical fits to experimental driving data. This confluence, which has not to our knowledge been noticed by previous investigators, sets the basic structure for our model.

SIMULATION MODEL

The car-following model set up for simulation is based on Eq. 2, with p = 2. This equation, however, governs the driver of each car only when the threshold test quantity for that driver, Eq. 4, exceeds a preset value for velocity threshold boundary. When the threshold boundary value is exceeded, the driver is considered in a velocity-detecting mode. Under threshold he is in a distance-detecting mode. The behavior of the simulated driver in the two modes is quite different, as outlined next.

Velocity-Detecting Mode

Over threshold, in the velocity-detecting mode, an individual driver in the simulation makes responses roughly in accordance with Eq. 2, although some overriding considerations arise. For example, the sensitivity factor, α, takes on different values for the accelerating and decelerating responses, since driver response differs in the two situations (9).

Furthermore, the values of acceleration are limited to the maximum values attainable at the current velocity (11), and deceleration is limited to given maximum values. In fact, except when the extremes of the driving situation warrant it, decelerations are limited to the more comfortable values of 8 to 11 ft per sec.

In addition, each simulated driver is assigned a certain desired velocity and attempts to keep his speed within ±15 percent or so of this value, provided there are no conflicting crises. He also has a preferred spacing (which depends on velocity levels) between his car and the car ahead.

Distance-Detecting Mode

Under threshold, a driver cannot detect relative velocity, but he can determine his actions on the basis of desired velocity, spacing, and so on. Furthermore, an actual driver does not monitor the situation continuously but samples various quantities at random intervals. Our simulated driver mirrors these responses, taking mild corrective action whenever he drifts away from the desired speed range.

Further Considerations

In our earlier models the brake light of the car ahead was not considered. When it was introduced into the model it had an excellent stabilizing influence, as it does in reality. In the model, a driver's brake light is considered to be on whenever his deceleration exceeds that caused by vehicle drag (11, p. 26), and a term representing the brake light of the car ahead is added to the threshold test quantity for the following car.

In the models discussed, the following car looks only one car ahead. In the current model the driver also considers the car two ahead and drives according to the equation

$$\frac{d^2x_{n+1}(t)}{dt^2} = \alpha v_n(t) \left[\frac{W_1 (v_n(t - T) - v_{n+1}(t - T))}{(x_n(t - T) - x_{n+1}(t - T))^2} + \frac{W^2 (v_{n-1}(t - T) - v_{n+1}(t - T))}{(x_{n-1}(t - T) - x_{n+1}(t - T))^2} \right] \qquad (5)$$

where

$$W_1 + W_2 = 1$$

so that he considers the relative spacing and velocity between his car and the car two ahead. A term representing the brake light of the car two ahead has also been added to the threshold test quantity, and these two additions have led to more realistic model behavior. In exceptional cases, mainly when the car two ahead is pulling away or when the driver of the $(n + 1)$st car is decelerating so rapidly that he would tend to look at most only one car ahead, the effect of the car two ahead is excluded.

The reaction time, T, not only varies from one driver to another, but it also varies over time for each individual driver, depending on his driving situation. When the driver goes over threshold his reaction time drops from its larger value in the distance-detecting mode to a smaller value, giving a quickened response time. Then after a while the driver's reaction time builds up, over time, until it reaches a maximum value which depends on whether or not the driver is over threshold. Whenever an unexpected event occurs, such as a rapid deceleration of the car ahead, reaction time drops down again. Figure 3 shows a typical reaction-time behavior.

IMPLEMENTATION OF THE MODEL

Although some consideration was given to constructing a special programming language to simulate the model, the idea was rejected as too time consuming and too far removed from the main aim of the study. The Fortran programming language was used, and on the whole it has proved very satisfactory. In Fortran the individual driver or vehicle characteristics can be stored as parameters with a single index so that T (Eq. 4), for example, is the response delay time of the driver of the fourth car in the platoon. The time-dependent quantities require two indices so that a quantity such as V (Eqs. 5, 2) represents the velocity of the fifth car two time steps ago.

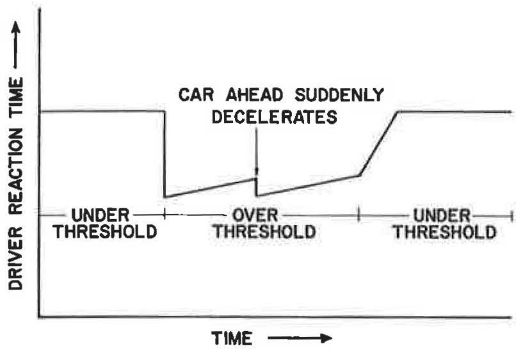

Figure 3. Driver reaction time.

The advantage of a common programming language such as Fortran is that a program can be moved with relative ease from one computer to another. The bulk of the computation has been done on an IBM 1620 computer, but the CDC 6600 computer at the Courant Institute of Mathematical Sciences of New York University is being used increasingly. Essentially the same Fortran program works on both.

The Computation

In the simulation, a platoon of cars is considered to be going along initially in a steady state at given spacings and velocities. The lead car then performs some maneuver, such as a rapid deceleration to a new velocity, and the response of the following cars is determined. At each computational time step, the response (acceleration or deceleration) of each following car in turn along the platoon is computed, and the equations are integrated numerically twice (using a simple parabolic rule) to determine the new velocities and positions. When the last car in the platoon, at that time step, has been computed, time is advanced by Δt (usually taken as 0.1 sec), and the procedure is repeated. In the course of the computation, care is taken to allow for time response lags, to test whether brake lights are on, and so on.

The considerable number of computations and decision tests that must be made require substantial computer time. On the IBM 1620-II computer, 5 sec of computer time is required for each car for each second of equivalent real time driving. Thus a half minute of real time driving for a platoon of six cars requires $5 \times 6 \times 30 = 900$ sec, or 15 min of 1620 time.

In the car-following situation, where things happen very quickly in real life, the length of run is fairly short and 1620 computation times are tolerable, but the longer overtaking studies require the faster speeds of the 6600 computer. The effective speed ratio of the two computers when running this simulation seems to be about 150 to 1.

Whenever the model is revised to any considerable extent, complete detailed data on the behavior of the model are printed out. The numerical values of the velocities, locations, and accelerations for each vehicle are printed at each time step, as well as the response time lags of the drivers, whether they are over or under threshold and whether the brake light is on. These data are used to check the correctness of the programming in detail, and to study the effect of various factors.

For general testing of the model, it has been found more convenient to put out data in graphical form. It is much easier to study the propagation of a disturbance down a platoon of cars by observing the plots of the velocities and the change in relative vehicle spacings than by reading numerical results. Figures 4 and 5 show a computer plot of the response of the platoon of cars to a rapid deceleration of the lead car. The sequential slowing down of each car in line is shown in Figure 4 (the curve marked by the numeral 2 represents the second car in line, etc.), and the change in relative spacing between cars is plotted in Figure 5 (where the curve marked by 2 now represents the relative spacing between the second car and the lead car, etc.). In this example, where T is 1.4 sec, the driving pattern settles down to a stable situation.

Figures 6 and 7 show the same lead car maneuver but carried out in a platoon where the response times of the following cars are so long that an accident occurs (T = 1.8 sec), a collision of cars 3 and 4. Whenever the results of a run show particular interest, as this one does, the numerical data are printed out as well.

Some phase-plane plots of response using velocity vs distance or acceleration vs velocity have been made for individual cars, but they are not included here.

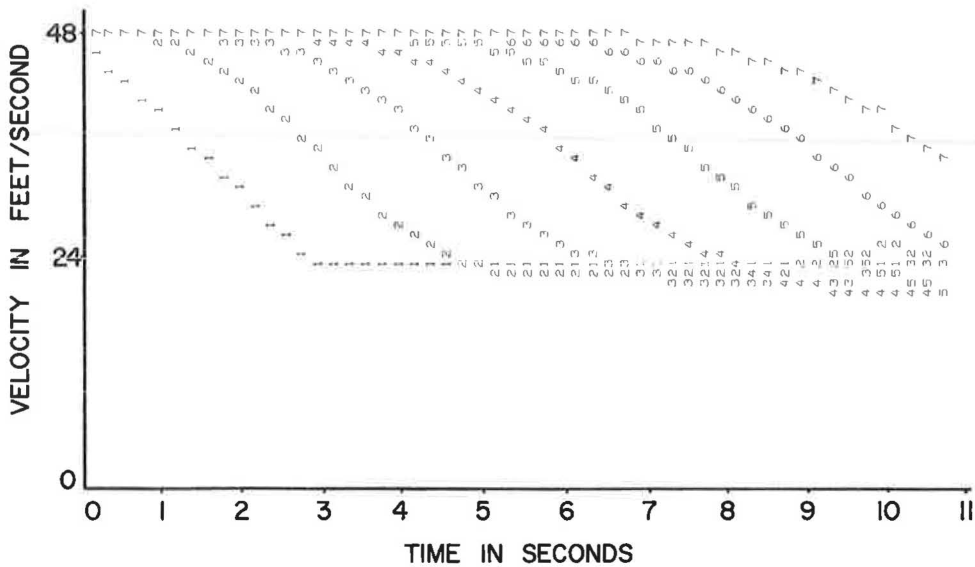

Figure 4. Individual car velocities.

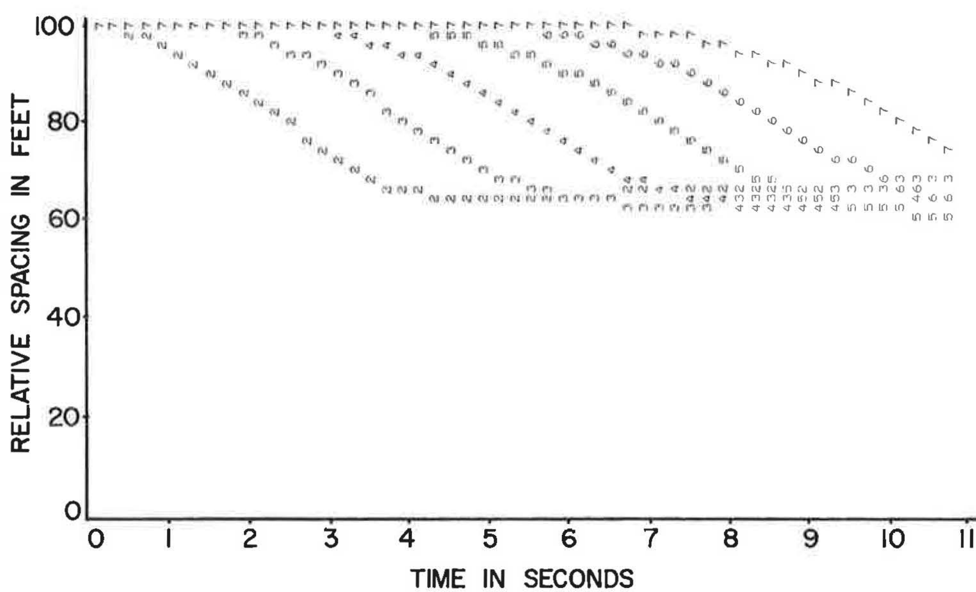

Figure 5. Relative spacing between cars.

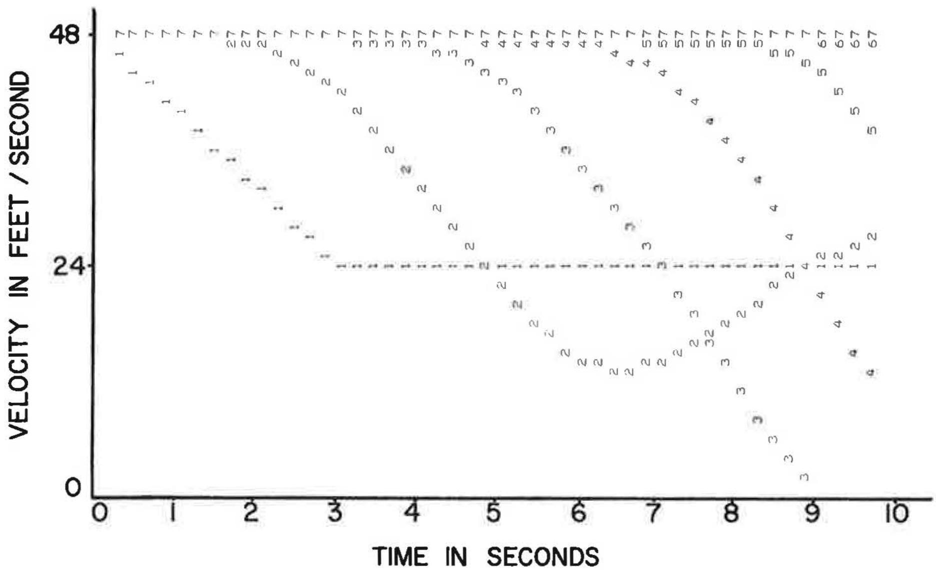

Figure 6. Individual car velocities—collision case.

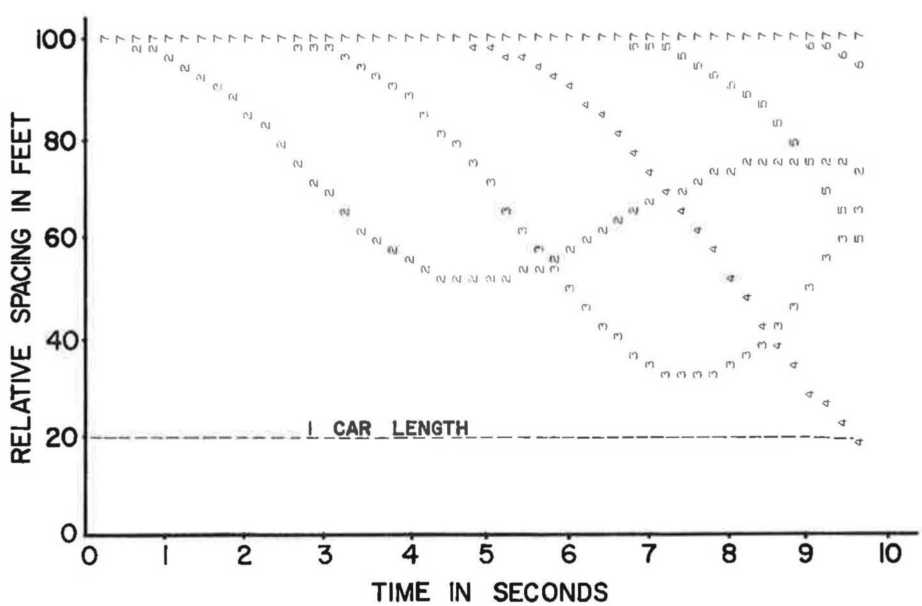

Figure 7. Relative spacing between cars—collision of cars 3 and 4.

RESULTS AND VALIDATION

In the past year, 400 to 500 cases have been run which study various models and various aspects of these models. The responses of the cars in a platoon to different lead car maneuvers have been explored, and the behavior of the model for various ranges of parametric values has been considered.

In the initial stages of the formulation, the aim was to develop a model whose behavior was acceptable on the average, i.e., when the behavior patterns were about average for each driver. It seemed reasonable to assume that only after the basic model was acceptable could the effect of individual driver anomalies on the driving situation be determined.

The investigation of parametric values suitable for representing typical car-following phenomena has supported the essential validity of the model. The approach in validating the model in this way has been an after-the-fact verification. Thus, for a given driving parameter the model is tested over a range of values to find the value interval giving a safe, stable and realistic simulation of driving behavior; the value interval is then checked against values determined by other investigators from actual experimental driving behavior.

This type of quantitative checking has yielded, for example, the following conclusions, supporting the essential correctness of the model: (a) if velocity detection thresholds are much larger than the 0.0005 to 0.0020 radians per sec given by the psychologists (7), the model exhibits instabilities; (b) if maximum decelerations are set too small, collisions occur; (c) if the range of tolerance about desired velocity is too large, oscillations arise; and (d) if the brake lights of the cars ahead are ignored, collisions occur.

A further quantitative check of this type is given by testing reaction times. If reaction times are set longer than the 1.4 sec found empirically by various investigators (3), the platoon behavior is unstable. Figure 8 shows a plot of the minimum spacing between pairs of cars achieved during platoon response to a moderately rapid lead car deceleration of 8 ft per sec. Note that for reaction times, T, of 1.4 sec or less the disturbance does not amplify as it propagates down the line of cars—no pair of cars gets closer than approximately 38 ft. For large reaction times, each pair of cars gets closer, leading to a collision of some pair of cars further along the platoon. At present the behavior of the model seems essentially correct. Both its behavior in a qualitative sense and its agreement with quantitative values are reassuring.

Some work has been done in attempting to validate the model with actual driving data from the Port of New York Authority. However, for each car both the aerial data and tunnel data are available only at 5- or 10-sec intervals, which is very coarse in terms of our model structure and in fact also in terms of the time scales of accident occurrence. It is necessary to have data at 0.5- or 1-sec intervals in order to measure the fine structure of individual driver behavior.

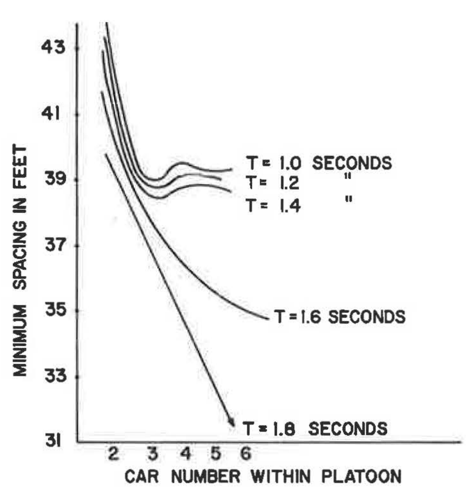

Figure 8. Effect of reaction time.

CONCLUSIONS

This research resulted in the construction of a model representing single-lane no-passing car-following driving situations. Simulation of the model on a computer shows that its behavior, on the average, corresponds very well with whatever data are known experimentally on the car-following problem. Therefore, usefulness of the simulation in projecting the situations and characteristics leading to instability and consequent accident occurrence seems assured.

Concentrated effort to validate the model will continue during the next phase of the study which will emphasize the study of the effect of individual driver differences on the driving situations leading to accident causation, especially in the overtaking case or in the inattentive car-following situation.

ACKNOWLEDGMENTS

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Georgia's Program for Automated Acquisition And Analysis of Traffic-Count Data

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•THE measurement of traffic flow on Georgia's highways provides basic data that are prerequisite to virtually all highway planning processes. The collection, processing and analysis of flow data constitute one of the major functions of the Division of Highway Planning, thus efficient and economical methods are essential. Because of the digital composition of traffic-flow data, the electronic computer has, of necessity, come to play an ever increasing role in the development of modern, large-scale traffic-counting and analysis programs. Georgia is currently experimenting with an expanded use of the computer to monitor and control a system of automatically collecting data accumulated at remote continuous-count station locations.

The automatic traffic data telemetry system uses a centrally located computer connected by telephone equipment to an electrical read-out counter at each remote location. The remote locations are automatically polled according to a predetermined schedule. Through the execution of a stored computer program, data are collected, edited and recorded. After a year of testing, the system is proving superior to previously used methods of collecting continuous-count data.

In conjunction with the installation of the telemetry system, the entire traffic-counting program was redesigned according to current statistical techniques. These revisions were necessary in order to extract the desired advantage from a traffic-counting program oriented toward maximum utilization of the computer.

INTRODUCTION

The scope of Georgia's pre-1964 program of traffic counting and analysis was largely dictated by economic limitations that could be directly attributed to the fact that the program was essentially oriented toward manual data-processing procedures. Any increase in the magnitude of the program would of necessity have resulted in a proportionate increase in the office staff.

The program included 28 continuous-count stations, 326 seasonal-control stations, embracing some 1493 count locations, and a program of coverage counting that included approximately 16,500 annual 24-hour weekday counts. The continuous-count segment of the program was already at least partially automated in that data produced by the stations were keypunched into data cards, thus allowing machine summary and analysis. Printed paper tapes containing hourly measurements of flow data were retrieved on routine weekly or biweekly service visitations. In spite of the rather high cost of such a procedure, some 13 percent of the count data was being lost due to equipment failures between service visits. Also, manual editing, coding and keypunch procedures proved to be slow and costly.

The 1493 locations in the seasonal control program were counted for one 24-hour weekday period during each calendar quarter. The results were manually summarized and posted directly on county maps. Whereas the quantity of counting locations assured widespread flow measurement on the state's highway network, the amount of data obtained provided no more than a marginal sketch of seasonal behavior patterns and gave virtually no consideration to weekend traffic.

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Traffic at coverage-count locations was counted for one 24-hour period annually. Although the count locations were generally repetitive from one year to the next, they were not in any way identified for continuing administrative purposes. The total 24-hour counts were posted on the same maps containing seasonal-control data. The only routine occasion for adjusting coverage counts to estimates of annual average daily traffic (AADT) was for the rather broad purpose of preparing the annual state traffic flow map. The individual county maps on which basic seasonal-control and coverage data were posted were not easily reproducible; thus, the distribution of data to persons requiring its use was severely limited.

Because of the voluminous amount of statistical data produced by a statewide traffic-counting program, it became apparent that an effective program could be developed only after a complete redesign of procedures, and by utilizing the most advanced computer applications and more sophisticated statistical techniques. Methods for using statistical techniques to objectively design traffic-counting programs had, by 1964, progressed to the point where the U. S. Bureau of Public Roads could draw from previous research to develop firm procedural outlines. At about the same time, the development of procedures that would permit the automatic acquisition of continuous-count data was being considered.

The basic technique employed by the State Highway Department of Georgia to obtain traffic-count data through a remote acquisition system was first subjected to field experiment in 1963. At that time a trial was conducted by Southern Bell Telephone and Telegraph Company for the Tennessee Department of Highways in which one remote vehicle detecting station was located in the Nashville area and dialed manually from the Department's headquarters office. The count data were received from the remote detector and punched into cards by an IBM keypunch machine. It was not until mid-1964, however, that Southern Bell's Data-Phone system of communication was interfaced with an IBM computer to permit a completely automated system for the acquisition of continuous-count data.

AUTOMATIC ACQUISITION AND EDITING PROCEDURES

The Highway Department embarked into the experimental field of remote acquisition of traffic data in June 1965. After having witnessed a demonstration held in Chicago, Illinois, in August 1964 by IBM in cooperation with Southern Bell and Streeter-Amet Company, a fully automated system for the collection of traffic data seemed feasible.

An IBM 1710 real-time system, necessary for interruptible computer programming, was installed in the Division of Highway Planning. The system consisted of a Model I 1620 central processor, a 1622 card reader, two 1311 disk drives, a 1443 printer, a 1711 data converter containing a real-time clock, and two 1712 terminal units (Fig. 1).

Southern Bell installed an 801-C automatic dialing unit and a 401-J Data-Phone data set in the home office. At each of the continuous traffic-counting locations selected for these tests, a 401-H Data-Phone data set was also installed. Connection between the data processing center and each of the remote locations was established by using wide area telephone service (WATS).

The traffic counters were manufactured and installed by the Streeter-Amet Company. These counters, powered by a trickle-charged battery with a reserve power adequate for a minimum of 8 hours and a maximum of 36 hours of operation in the event of loss of AC power, have nondestructive read-out capability. Up to 10 counters, each of which recycle every 9999 impulses, may be installed at any remote traffic-counting location.

Four test sites were selected randomly from 28 traffic-counting stations which were in operation at that time. Several prerequisites guided these selections. It was stipulated that each station be located at varying distances from the home office and in areas of diverse climatic conditions. It was preferred to have one of the 4 stations in an area where telephone service was supplied by a relatively small independent utility. In addition, one trial station was to be located at a site where volumes are accumulated directionally.

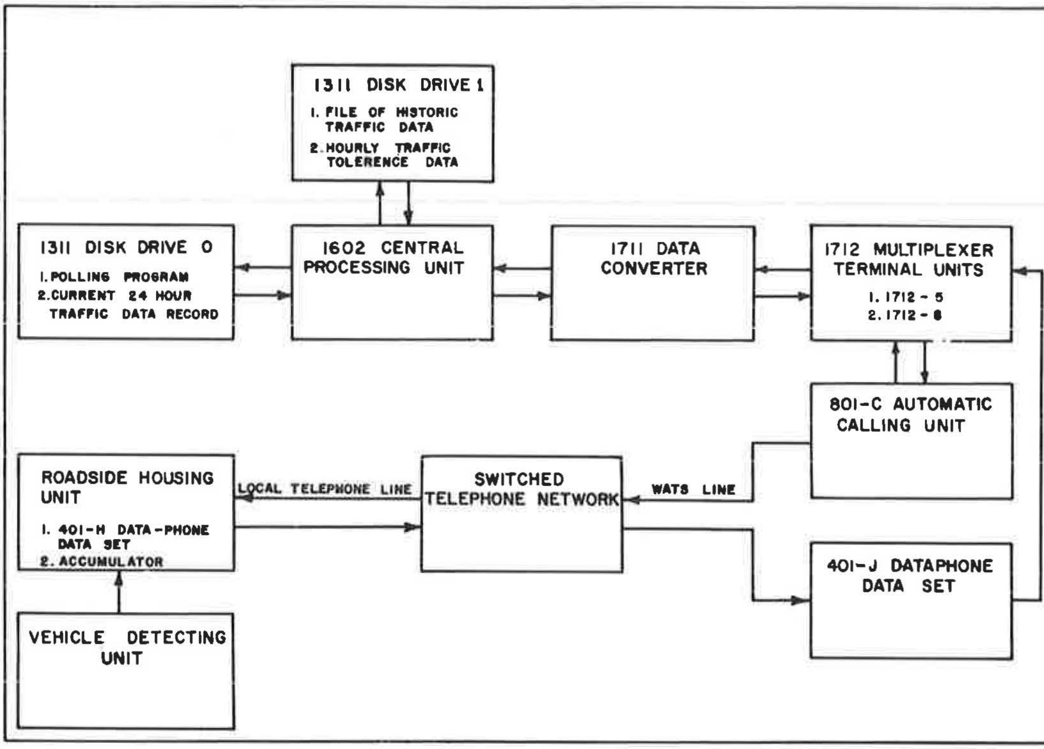

Figure 1. Automatic traffic data telemetry system hardware.

Various types of traffic detectors were used. Two stations employed pneumatic road tubes, one station employed an overhead ultrasonic detector, and the fourth utilized a magnetic detector. At the time of this writing, however, one of the pneumatic tube detectors had been replaced by an induction loop and the only magnetic detector used in these tests had been replaced by a pneumatic tube.

Each counting device was installed at the roadside in a weatherproof housing, which also contained each station's respective 401-H data set.

General Method of Operation

The computer program, which governs the telephone dialing of the traffic-counting stations and storage of incoming data, had been written and tested by IBM at an earlier date. However, due to desired program modification and additional debugging, this program was not operational until July 1965. Other program alterations made since that time by the Georgia Highway Department enable the polling program to monitor the entire telemetry system.

A confederate program, which is automatically called into execution immediately after the 12 p. m. poll of traffic-counting stations, edits all traffic data collected the previous day and supplies any volumes which could not be obtained during the scheduled polls. As soon as the data are edited, it is added to a historic file from which any desired report relating to traffic volumes may be prepared.

The performance of these two programs is dependent on external interruptions of routine processing. Therefore, the 1710 system must be in the automatic and interruptible mode prior to the execution of the program. Any founctional component of on-line hardware being in an interlocked status during the poll of counting stations may result in a program malfunction.

BITS	CORE STORAGE POSITION 1	2	3	4	5	6	7
F							
8							
4		2	5	8	11	14	17
2	0	3	6	9	12	15	18
1	1	4	7	10	13	16	19

Figure 2. Assignment of 1712-5 contact points to bits in core storage.

	B1	B2	B3
A1	1	2	3
A2	4	5	6
A3	7	8	9
A4		0	

Figure 3. Matrix codes assigned to parallel bits received by 401-J Data Phone.

TERMINAL POINTS	BIT ASSIGNMENT
00	8
01	4
02	2
03	1
04	F
05	8
06	4
07	2
08	1
09	F
10	8
11	4
12	2
13	1
14	F
15	8
16	4
17	2
18	1
19	F

Figure 4. Assignment of 1712-8 terminal points to bits in core storage.

Each hundredth of an hour, an electrical impulse emitted by the 1711 data converter interrupts the main-line program currently being executed by the computer. This impulse compels a process branch to a predetermined location in core storage where instructions are stored to test programming interrupt indicators. If it is determined that the interrupt was produced by only the 0.01 hour indicator, control is returned to the main-line program with a loss of only 400 microseconds of processing time. If the program deduces that a valid external interrupt signifying a poll has occurred, the interrupted program residing in the first 20,000 positions of core storage is transferred to the monitor disk pack, and the polling program is read into core storage and the polling process commences.

To execute the poll, the program reads into core storage, from the on-line disk drive, the telephone number of the first counting station. A call request must be entered before the 801-C automatic calling unit (ACU) can be instructed to dial a digit. This is done by executing programmed instructions which close two contacts in the 1712-5 terminal unit (Fig. 2). This completes a circuit to the ACU which responds with a dial tone. The ACU, when ready for a digit to be presented for dialing, will emit an electrical impulse which turns on another process-branch indicator. The program, by referring to an encoding table stored in core, then closes those contacts on the 1712-5 terminal unit necessary to command the ACU to dial the desired digit. These contacts and equivalent digits are shown in Figure 2. After each digit is presented to the 1712-5, another contact is closed to notify the ACU that the digit contacts have been set.

The program must then await an impulse generated by the ACU requesting the next digit. In this manner, the entire telephone number of the counting station is sequentially presented to the 801-C dialing unit. A 7-digit telephone number plus a 3-digit area code can be dialed in approximately one second.

As soon as all digits of the telephone number have been presented to the ACU through the 1712 terminal unit, the polling program is returned to the on-line disk pack, the interrupted program is read back into core storage and main-line processing is resumed. The 401-H data set at the roadside counting station, upon answering the call, transmits a 2,025-cps tone or lead signal to the calling unit. The lead signal generates a second interrupt of main-line processing by impulsing its respective process branch indicator within the 1711 which again causes the polling program to be interchanged with the main-line program. Simultaneously, the remote 401-H data set has signaled the traffic accumulator to read out the current volume. This volume is relayed over the telephone facilities by the 401-H data set in parallel by bit, but serially by digit in the form of multi-frequency tones. These tones are detected by the 401-J data set in the computer center and converted to contact closures in the 1712-8 terminal unit. The digit which these parallel bits represent depends on their conversion within a 4 by 3 matrix. To facilitate decoding, the contacts associated with the three channelled dimension (B channels) of the matrix are wired directly to processing indicators, whereas the contacts associated with the four channelled dimension (A channels) are multiplexed and transmitted to a Digital Input Adapter in the 1711. This unit is designed to transmit digits in groups of four directly into four positions of core storage in 8-4-2-1 format, even though only one digit is significant in this case. The value of the transmitted digit can then be ascertained by programmed examination of the A channel digit in relation to the status of the processing indicators set by the B channel impulse. Four digits must be sequentially decoded to constitute a valid accumulated volume. The matrix and bit assignment of terminal points on the 1712-8 are shown in Figures 3 and 4.

If any counting station does not respond with a valid 4-digit volume when polled, a coded message is immediately typed defining the nature of the failure. The types of possible irregularities which have been programmed for detection are shown in Figure 5. In addition to monitoring hardware failures, each volume is immediately checked to ascertain whether or not the volume accumulated since the last poll is within expected tolerances. These tolerances, which are extracted from historical data, represent the square root of the average of the squares of a set of deviations about an arithmetic mean. These standard deviations have been calculated for every hour, day of week and month, before being stored on an on-line satellite disk drive (disk drive 1). The tolerance data for the current day are always stored with the associated telephone number for each counting station on disk drive 0. The 4-digit high and low tolerances for the current hour are replaced by the accumulated volume and time of reading after the tolerance tests have been made. These tolerance records are automatically updated at midnight of each day as the daily record of each counter is added to a historic file. This file is built on the same satellite disk drive on which the current monthly record of hourly tolerances has been stored.

As each station is called, an internal indicator is set if a valid volume is not received. After all counting stations have been called, any station for which an indicator has been set is recalled. No station is called more than twice during the same poll. The total machine time required to poll each station is approximately 4.5 sec. However, since 15 to 20 sec are required to make the connection with the counting station after the telephone call is placed, 20 to 25 sec are consumed in polling each station.

After all stations have been polled, the latest accumulated volume for each station is compared with the accumulated volume received during the two previous polls. If no change is apparent, the "check box" message shown in Figure 5 is typed. This test aids in detecting damaged detecting units which have ceased to impulse their associated accumulator.

CODE	STATION	TIME	DEFINITION
$\bar{1}$	$\bar{0}10$	$\bar{1}301$	REPLACE CALL. LINE BUSY OR OUT OF ORDER.
$\bar{2}$	$\bar{0}22$	$\bar{1}301$	INVALID DIGIT RECEIVED FROM FIELD.
$\bar{3}$	$\bar{0}26$	$\bar{1}302$	NOT ENOUGH POSITIONS ON POLL RECORD TO STORE RESPONSE.
$\bar{4}$	$\bar{0}24$	$\bar{1}303$	NO POWER ON DATA SET.
$\bar{5}$	$\bar{0}10$	$\bar{1}401$	NO REPONSE WITHIN 32-64 SECONDS AFTER DIALING.
$\bar{6}$	$\bar{0}22$	$\bar{1}402$	VOLUME NOT WITHIN PREDICATED TOLERANCES.
$\bar{7}$	$\bar{0}26$	$\bar{1}403$	PRESENT-NEXT DIGIT INDICATOR TURNING OFF TO SLOWLY.
$\bar{8}$	$\bar{0}24$	$\bar{1}404$	FALSE INTERRUPT DURING POLL.
$\bar{9}$	$\bar{0}10$	$\bar{1}501$	PRESENT-NEXT DIGIT INDICATOR TURNING ON TOO SLOWLY.
CHECK	BOX 022		NO ACCUMULATION FOR A PERIOD OF TWO HOURS.

Figure 5. Description of telemetry messages.

Because the monitor disk on which the polling program and all current data are stored has limited storage capacity, each day's record of traffic volumes must be transferred to an auxiliary file. This is performed automatically after the completion of the 12 p. m. poll. At this time, the midnight program is brought into execution which edits the data collected the previous day. This edit entails the estimating of absent volumes, adjusting volumes for extra axles when necessary, and netting of each hour's accumulated total.

Two techniques are being employed to estimate the accumulated hourly volume at any station from which no reading was possible. If a successful poll was accomplished at the hour immediately prior to and after the hour, or hours, with missing volumes, the difference between the two accumulated readings is prorated in proportion to the midpoints of the tolerance ranges for the hours being estimated. For instance, had it been impossible to contact a counting station at 6, 7, and 8 p. m., but a reading had been successfully taken of 1500 at 5 p. m. and 1900 at 9 p. m., the actual difference of 400 counts would be prorated between 6, 7, 8, and 9 p. m. This would be done by accumulating the midpoints of the tolerance range of volumes for each hour and computing the percentage that each accumulated midpoint volume is of the total of the midpoint volumes for the 4 hours. This percentage is then applied to the actual accumulated volume of 400 to determine the estimated accumulated volumes to be recorded. This produces a realistic profile of hourly volumes for any given day and results in an unadjusted 24-hour total volume.

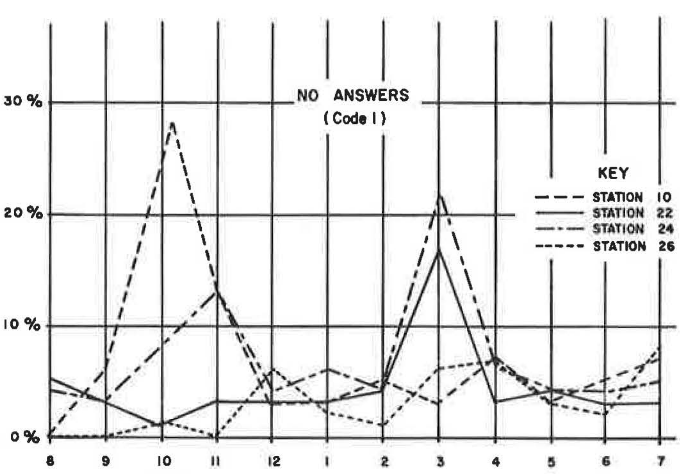

Figure 6. Monthly occurrence of no answers.

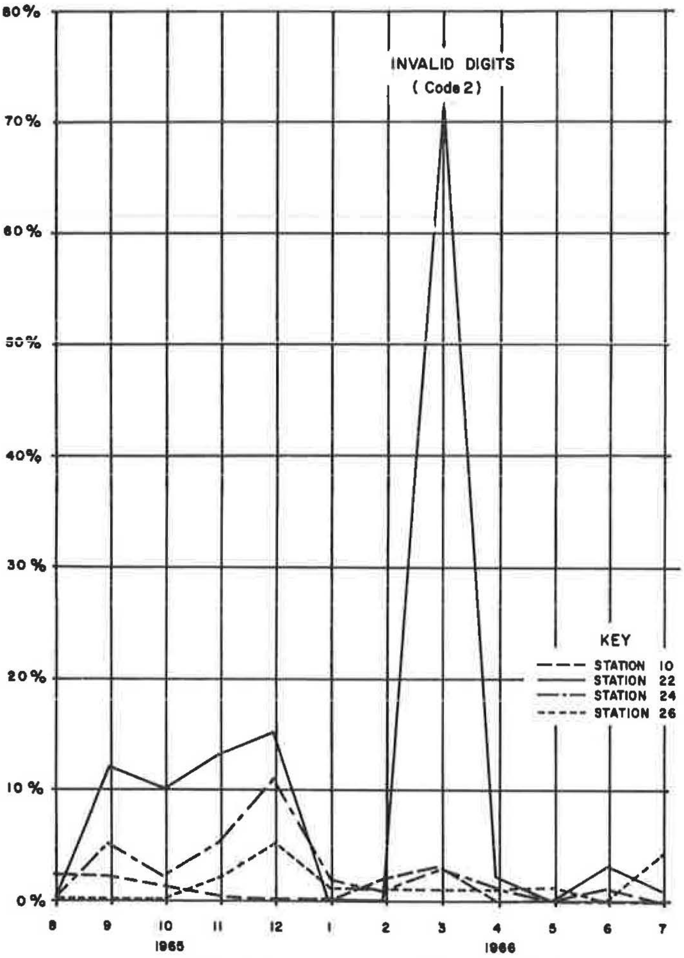

Figure 7. Monthly occurrence of invalid digits.

It is necessary, however, to take a slightly different approach if a valid reading of any given counter is not taken at midnight. In this case, the volume for this hour, plus any hours with missing volumes immediately preceding midnight, is estimated by using the associated tolerance midpoint itself for each hour within the open-end time interval. The entire operation, consisting of the polling of traffic-counting station and editing of collected data, is accomplished without any operator intervention.

Evaluation of the Telemetry System

After one year of exhaustive testing, it is apparent that the telemetry technique of traffic-data acquisition constitutes an improvement over any former method employed by the State Highway Department of Georgia. The gathering of traffic data by a central collector not only lends itself to a continually current appraisal of the status of all counting stations, but maintenance personnel can be dispatched to any counting station within 3 hours after the occurrence of a failure at any remotely located field installation.

While the telemetry system is monitored by the stored program for 10 different types of irregularities (Fig. 5), only 3 have given any cause for concern. These have been: (a) failure of the remote 401-H data set to answer the call; (b) reception of invalid digits by the 1712-8 terminal block; and (c) no response from the station after the call has been answered. By consulting Figures 6, 7 and 8, it can be seen that these troubles are steadily being eradicated.

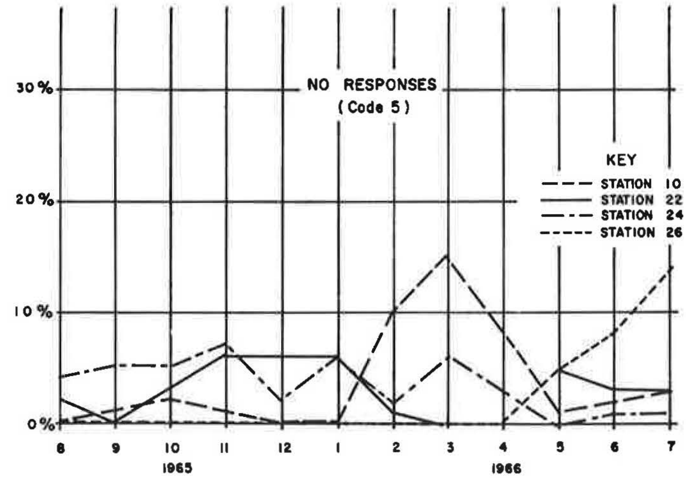

Figure 8. Monthly occurrence of no responses.

Figure 6 shows the continuing improvement in the curtailing of "no answers." Although it cannot be expected that a zero percentage of no answers will be achieved, it is within the realm of probability to confine these to less than 2 percent of all calls placed. The no answer situation results when an incorrect telephone connection is made at the switching station, when the circuits between the computer center and the remote data set become overloaded, or when the remote data set fails to answer within 32 to 64 sec after being dialed. The latter cause is the least frequent of the three. Incorrect connections due to switching malfunctions in excess of 2 percent of all calls placed can usually be decreased by changing the telephone number of the problem traffic-counting station.

Figure 7 shows the occurrence of invalid digit receptions from August 1965 through July 1966. The abundance of these digits was caused by the voltage that is relayed to the 1712-8 by the 401-J being above designed tolerances. Reduction of the voltage of this impulse has restricted the transmittal of 4-digit volumes with an invalid character to less than 1 percent. This remaining 1 percent is a result of electrical disturbances along the transmission facilities, or improper operation of computer hardware during the polling operation.

Figure 8 shows the percentage of time during each month that less than 4 digits were received after a call was placed, even though the 401-H data set answered and returned a lead signal. This was caused either by the accumulator's failure to properly read out the volume, or by the 401-H data set's failure to transmit the volume. Although the occurrence of "no response" has been reduced to acceptable limits, improvements in the design of the field hardware should further diminish the no response problem (Fig. 9).

The efficiency of the telemetry system, at this stage, compares very favorably with former methods of traffic-data collection. Although certain components of the system were experimental in nature, the hours having no volumes due to unsuccessful polls during August 1965 through July 1966 represent only 11 percent of the total (Fig. 10). On the other hand, prior operations suffered a loss of 13 percent due to equipment failures. It also should be noted that the 11 percent unobtained volumes were scattered, thus facilitating accurate and automatic estimations.

Redesigned accumulators have recently been installed at stations 10, 22, and 24. Polling of these counting stations is now being successfully performed approximately 93 percent of the time (Fig. 9). Due to the degree of success of current operations, 9 additional counting stations are being added to the telemetry system.

Accessory computer programs are being implemented to tabulate and update the hourly volumes collected from any counting station by the telemetry process (Fig. 11). The tabulation can be scanned by a traffic analyst to insure the acceptability of all data.

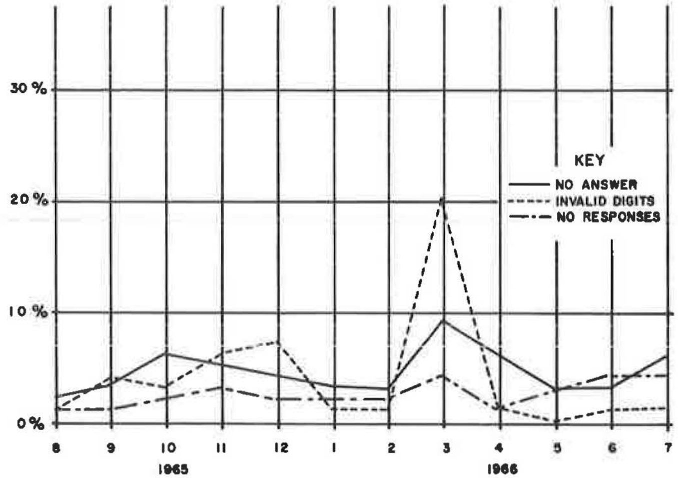

Figure 9. Monthly occurrence of no answers, invalid digits and no responses for all test stations.

Based on the educated discretion of the analyst, any unacceptable recorded hourly volume may subsequently be discounted and a substitute volume mechanically estimated by the computer, or as an alternative, a volume could be estimated by the analyst. This would require introducing an appropriately coded control card to the computer system.

Since a current file of edited traffic volumes is always maintained on a directly accessible disk pack, retrieval of these data for any purpose of utilization can be effortlessly and speedily accomplished.

GENERAL METHODOLOGY FOR REDESIGNING THE RURAL COUNTING PROGRAM

Georgia's program of traffic counting through the year 1963 was probably a fairly typical result of the expansion of methodology that began with the establishment of the state highway planning surveys during the latter 1930's. The prime motivation for redesigning the 1963 traffic-counting program must be attributed to the decision to adopt the previously described automatic traffic data telemetry system. However, any effort to obtain a program of maximum efficiency could not ignore the necessity for concurrently developing a total counting and analysis program that was as refined as

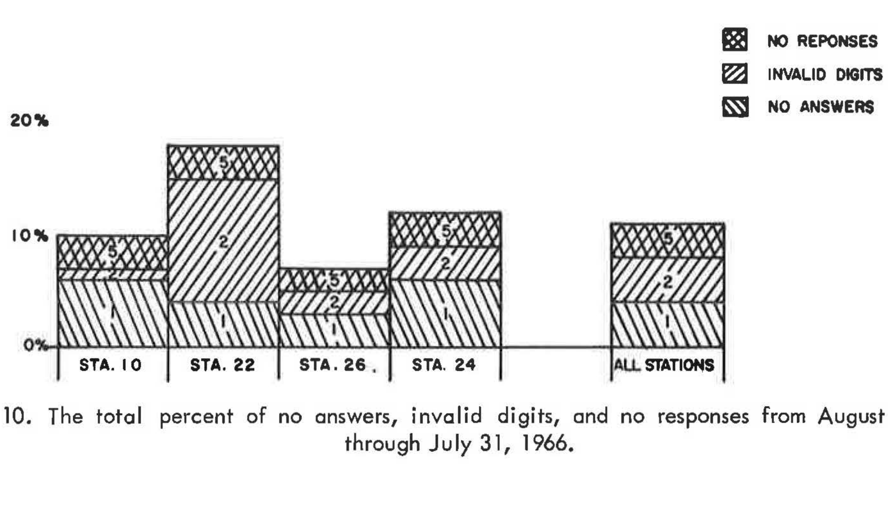

Figure 10. The total percent of no answers, invalid digits, and no responses from August 1, 1965 through July 31, 1966.

TELEMETRIC TRAFFIC VOLUMES

STATION 010

DATE	HOUR	VOLUME	RANGES	REMARKS
08/19	0100	0091	65- 129	
08/19	0200	0081	43- 107	
08/19	0300	0064	49- 89	
08/19	0400	0064	29- 125	
08/19	0500	0084	39- 103	
08/19	0600	0119	56- 164	
08/19	0700	0186	98- 258	
08/19	0800	0205	149- 293	
08/19	0900	0300	171- 407	
08/19	1000	0398	207- 475	
08/19	1100	0425	216- 524	
08/19	1200	0474	230- 534	
08/19	1300	0399	225- 505	
08/19	1400	0471	212- 560	
08/19	1500	0506	224- 600	
08/19	1600	0565	250- 618	
08/19	1700	0581	230- 614	
08/19	1805	0518	216- 604	
08/19	1904	0422	166- 522	
08/19	2004	0358	150- 458	THIS VOLUME IS ESTIMATED
08/19	2106	0287	121- 365	THIS VOLUME IS ESTIMATED
08/19	2205	0237	87- 315	THIS VOLUME IS ESTIMATED
08/19	2300	0210	92- 264	THIS VOLUME IS ESTIMATED
08/19	0000	0163	81- 217	

Figure 11. Sample printout of telemetric traffic volumes.

the existing state of the art permitted. To have effected an advanced system of data acquisition while retaining relatively archaic portions of the existing program would have been at least incongruous, if not a total negation of any advantage that a data telemetry system might have offered.

Two considerations have emerged in the past decade that have vastly altered the concept of what constitutes a desirable program for obtaining and analyzing traffic-count data. The first is the greatly expanded availability and utilization of the electronic computer for highway-planning activities. The second is the development of techniques for applying statistical methodology to the design and objective evaluation of large-scale traffic-counting programs.

Grouping of Road Sections by Pattern Similarity

Traffic-flow measurement is used in one or more of its various forms to satisfy requirements related to the planning, programming, traffic control, design, maintenance and general administration of the highway program. To provide this required information, the traffic-counting program should ideally provide the following:

1. Values representing AADT for all system road sections;
2. Data related to trends and characteristics of design hour volumes;
3. Volume growth trend data; and
4. Composition of traffic volume by vehicle type.

With the exception of composition data, this information could very well be obtained by operating a traffic-recording device continually, over a period of years, on each road section for which the data are required. Because of the obvious financial and physical impracticality of such a procedure, historical practice has been to obtain a short-term sample count that could be used as the basis for estimating AADT for each system road section. A limited number of strategically located points could then be operated as continuous-count stations capable of producing design hour and growth trend data as well as providing the factors necessary to adjust the short-term samples into estimates of AADT.

The problem of determining an objective method of identifying or associating the location of short-term sample-count stations with the various seasonal patterns measured at continuous-count station locations has been the object of a great deal of research and discussion over the past few years. In pre-1964 practice, Georgia's procedure for accomplishing this association must be described as primarily an intuitive one with the major criterion being geographic proximity. Because of the sheer size of the seasonal-control counting program, a station leg was almost never very far removed from any given coverage-count location. This meant that the majority of the coverage counts were adjusted to AADT by comparison to data produced by a control-station leg that was counted for a 24-hour weekday period 4 times per year. Earlier studies had shown that this procedure resulted in a standard error of approximately ± 15 percent, a value considered unacceptably high, particularly in terms of the cost of operating such an extensive seasonal-control counting program.

The redesign of Georgia's traffic-counting procedures has generally followed the outline provided by the U. S. Bureau of Public Roads "Guide for Traffic Volume Counting Manual." The "Guide" advances a procedural outline that is based on the concept that patterns of monthly variation tend to persist over a significant number of contig-

GROUPING OF 1963 CONTINUOUS COUNT STATIONS AND GROUP MEAN MONTHLY FACTORS

STATION NO.	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
GROUP I												
1	1.22	1.16	1.07	1.02	.97	.94	.90	.93	1.08	1.14	1.11	1.13
4	1.30	1.26	1.15	1.08	1.09	.99	.93	.93	.98	1.04	1.07	1.15
8	1.25	1.11	1.02	1.01	1.08	.98	.95	.92	1.01	.95	1/	1.00
12	1.16	1/	1/	1/	1.08	.87	.85	.83	1.05	1.07	.99	1.01
22	1.29	1.24	1.17	1.06	1.07	.98	.90	.86	.96	1.00	1.01	1.09
24	1.19	1.15	1.08	1.03	1.07	.99	.91	.97	1.06	.97	.98	.97
25	1.21	1.18	1.09	1.06	1.13	.93	.89	.91	1.10	1.14	1.12	1.08
26	1.13	1.11	1.02	1.00	1.09	.96	.93	.95	1.04	1.03	1.14	1.10
28	1/	1.09	1.04	1.09	1.03	.89	.88	.92	1/	1/	1/	1/
29	1.16	1.19	1.11	1.06	1.00	.95	1/	.99	1.04	1.03	1.05	1.05
32	1.20	1.16	1.11	1.01	1.04	.93	.93	.90	1.01	1.05	1.03	.99
35	1.11	1.10	1.05	1.01	1.01	1/	.93	.89	.97	.97	1.00	1.04
39	2/	2/	2/	2/	2/	.80	.83	.81	2/	2/	2/	1.00
GROUP MEAN	1.20	1.16	1.08	1.04	1.05	.93	.90	.91	1.03	1.04	1.05	1.05
GROUP II												
2	1.17	1.14	1.07	1.06	1.03	1.00	1.03	.93	.91	.98	1.00	.99
3	1.17	1.18	1.00	.97	1.04	1/	1.04	.93	.96	1.02	1.05	1.07
21	3/	3/	3/	3/	3/	3/	3/	3/	1.11	3/	3/	3/
23	1/	1.20	1.14	1.09	1.08	1.04	.94	.89	1.01	1.04	1.07	1.18
30	1.19	1.12	1.12	1.02	1.02	1/	1/	.78	1.08	1.18	1.20	1/
33	1.20	1.11	1.14	1.14	1.06	1.10	1.05	.98	1.00	1.07	1.04	1.08
GROUP MEAN	1.18	1.15	1.09	1.06	1.05	1.05	1.02	.90	1.01	1.06	1.07	1.08
GROUP III												
7	1.43	1.24	1.07	1.06	1.14	.89	.84	.86	1.09	1/	1/	1/
31	1.42	1.34	1.24	1.09	1.07	.96	.85	.83	1.00	1/	1/	1/
34	1.34	1.32	1.25	1.11	1.09	.90	.79	.78	.99	1/	1.23	1.37
37	1.31	1.25	1.14	1.01	1.05	.91	.82	.88	1.09	1.23	1.23	1.27
38	1/	1/	1.25	1.06	1.08	1.02	.89	.96	1.14	1.27	1.27	1.35
GROUP MEAN	1.38	1.29	1.19	1.07	1.09	.94	.84	.86	1.06	1.25	1.24	1.33
GROUP IV												
10	1.16	.99	.95	.92	1.12	.99	.89	.93	1.24	1.24	1.17	1.05
36	1.09	.92	.93	.86	1.14	.92	.88	.87	1.17	1.16	1.17	1.08
GROUP MEAN	1.13	.96	.94	.89	1.13	.96	.89	.90	1.21	1.20	1.17	1.07

1/ VALUES FOUND UNACCEPTABLE FOR VARIOUS REASONS.

2/ STATION 39 PLACED IN GROUP I AFTER EXAMINATION OF PREVIOUS RECORDS. DATA MADE UNUSABLE IN 1963 DUE TO CONSTRUCTION AND/OR OPENING OF PORTIONS OF I-20 IN ADJACENT STATE.

3/ STATION 21 PLACED IN GROUP II AFTER EXAMINATION OF PREVIOUS DATA. 1963 DATA UNUSABLE DUE TO CONSTRUCTION.

Figure 12. Continuous-count station groupings, 1963.

STANDARD DEVIATION OF MONTHLY FACTORS FROM THEIR RESPECTIVE MEANS STANDARD ERROR OF THE MEAN & "F" TEST RESULTS	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
GROUP I												
Mean Factor	1.20	1.16	1.08	1.04	1.05	.93	.90	.91	1.03	1.04	1.05	1.05
Standard Deviation	± .0612	± .0559	± .0497	± .0318	± .0549	± .0570	± .0358	± .0517	± .0455	± .0638	± 0577	± .0588
Standard Error of Mean	± .0184	± .0168	± .0150	± .0096	± .0133	± .0165	± .0103	± .0143	± .0137	± .0192	± .0163	± .0170
GROUP I SAMPLE 1												
Mean Factor	1.20	1.18	1.09	1.04	1.06	.94	.90	.92	1.05	1.08	1.08	1.09
Standard Deviation	± .0610	± .0543	± .0483	± .0265	± .0613	± .0400	± .0332	± .0533	± .0412	± .0527	± .0551	± .0518
Standard Error of Mean	± .0249	± .0242	± .0216	± .0118	± .0250	± .0163	± .0148	± .0218	± .0168	± .0215	± .0225	± .0211
GROUP I SAMPLE 2												
Mean Factor	1.21	1.14	1.08	1.04	1.05	.95	.92	.91	1.00	.99	.99	1.02
Standard Deviation	± .0680	± .0557	± .0550	± .0338	± .0276	± .0430	± .0253	± .0369	± .0397	± .0381	± .0308	± .0477
Standard Error of Mean	± .0304	± .0227	± .0224	± .0138	± .0113	± .0192	± .0103	± .0151	± .0178	± .0170	± .0138	± .0213
Value of "F"	1.245	1.05	1.296	1.63	4.95	1.156	1.72	2.088	1.076	1.917	3.20	1.175
Limit at .05 Level	5.19	6.26	6.26	6.26	5.05	6.26	5.19	5.05	6.26	6.26	6.26	6.26
GROUP II												
Mean Factor	1.18	1.15	1.09	1.06	1.05	1.05	1.02	.90	1.01	1.06	1.07	1.08
Std. Deviation	± .0152	± .0387	± .0600	± .0644	± .0245	± .0505	± .0510	± .0754	± .0742	± .0757	± .0803	± .0779
GROUP III												
Mean Factor	1.38	1.29	1.19	1.07	1.09	.94	.84	.86	1.06	1.25	1.24	1.33
Std. Deviation	± .0594	± .0500	± .0815	± .0381	± .0339	± .0543	± .0371	± .0666	± .0647	± .0283	± .0234	± .0529
GROUP IV												
Mean Factor	1.13	.96	.94	.89	1.13	.96	.89	.90	1.21	1.20	1.17	1.07
Std. Deviation	± .0500	± .0500	± .0141	± .0424	± .0141	± .0500	± .0100	± .0424	± .0500	± .0566	± .00	± .0224
ALL STATIONS COMBINED												
Mean Factor	1.22	1.16	1.10	1.04	1.07	.95	.91	.90	1.05	1.08	1.10	1.10
Std. Deviation	± .0934	± .0968	± .0855	± .0617	± .0437	± .0643	± .0672	± .0574	± .0753	± .0918	± .0902	± .1127

Figure 13. Standard deviation, standard error of the mean and F test results on grouped data.

uous road sections and are annually repetitive over relatively long periods of time. This concept is supported by studies of data produced by many continuous-count stations in various states which have revealed sufficient similarities of measured patterns to permit organizing the continuous-count stations, and consequently the road sections on which they are located, into groups such that the monthly variation pattern exhibited by individual stations comprising the group does not differ from the group mean variation pattern by more than approximately ± 0.10. The seasonal pattern or configuration thus measured is sometimes ascribed to the general concepts of Gestalt Psychology. The bases used for comparing and grouping patterns of monthly variation are ratios obtained by dividing AADT by the average weekday volume computed for each month.

Figure 12 shows the results of the grouping of 26 rural continuous-count stations operated in Georgia during 1963. Individual ratios or factors, as described previously, are shown for each station by month along with the arithmetic average or mean by group for each month.

To investigate the dispersion of individual factors about the mean, the standard deviation for each month, by group and for all groups, was computed. Since Group I was formed by 13 of the stations, or half the total number under study, the group was randomly divided into two samples for the purpose of determining the effect of reducing the total number of stations in this group. The standard deviation for each sample was computed along with the standard error of the mean. The statistical F test of significance was used to compare the two samples. This permitted the conclusion that the samples probably did come from the same population and that the total number of stations in Group I could be reduced without seriously affecting factor data produced for the group. Because of the smaller number of stations in Groups, II, III, and IV, no consideration was given to reduction of stations in these areas. The results of the described examination of grouped data are shown in Figure 13. It is interesting to note, relative to the concept of group stability, that only 4 of the 26 stations transferred from one group to another during the period from 1963 through 1965, and each of these transfers could be attributed to the fact that an Interstate or high type primary facility had been opened to traffic either parallel or contiguous to the station location. In each instance of the opening of a parallel route, a continuous-count station located on the new route produced data that identified with the previous behavior group.

The approach to the problem of allocating road sections to seasonal variance groups was based on the concept of similar patterns of variation persisting over significantly long sections of highway. Granted this, it became possible to associate a considerable number of intermediate road sections between continuous-count station locations by extending the measured pattern on the basis of continuity from and between the several known points. Data produced by the seasonal-control counting program offered the best available basis for group assignment of the more difficult sections. Additionally, these data provided substantiating evidence on those sections assigned by pattern extension from continuous-count stations.

In order to ascertain the identification of each control-station leg with the group means produced by the major behavior patterns, computer programs were developed to utilize a method of least squares to determine the group of best fit. Briefly, this involved comparing the factors produced by seasonal-control counts to each set of group mean monthly factors. The difference observed in the compared values were squared and summed by group. The resulting summation producing the lowest value was interpreted to be the group of best fit.

The actual mechanics of grouping road sections became relatively simple once all seasonal-control stations were analyzed and identified with the major behavior patterns. Continuous-count and seasonal-control station locations were noted on a map showing all rural state and Federal-aid system roads. The pattern group with which each location had been associated was symbolically noted and the pattern scheme extended to include the maximum number of contiguous road sections belonging to the same pattern group. With the vast amount of seasonal-control data available, this procedure permitted, with some degree of objectivity, the grouping of approximately 85 percent of all rural road sections having an AADT volume in excess of 500 vehicles per day.

The Continuous-Count Program

The approach to the overall redesign of Georgia's traffic-counting program examined continuous-count stations first because of their inherent importance as the producers of data around which other counting activities are designed, and because of the urgency introduced by the impending adoption of the traffic-data telemetry system. A very broad delineation of the needs for continuous-count data is as follows:

1. The production of factor data necessary for converting short-term count observations into reasonable estimates of AADT;
2. The determination of composite or statewide long-range travel trends;
3. The determination of the relation of design hour and other high-hour volumes to AADT; and
4. To facilitate detailed corridor analysis preceding the development of design traffic assignments.

With respect to item one, it is generally conceded that a minimum of 4 station locations are required on road sections for which an independent set of mean monthly factors is to be obtained. Thus, all rural road sections were stratified into three general classifications for the purpose of quantifying continuous-count needs in terms of AADT estimating requirements. These are termed: (a) Category I, rural road sections, AADT = 500 vpd (Interstate excepted); (b) Category II, rural road sections, AADT = 500 vpd; and (c) Category III, rural Interstate road sections.

The 1963 continuous-count program consisted entirely of station locations that could be ascribed to Category I. The grouping of data produced by these stations revealed four distinctly definable seasonal behavior patterns. In order to minimize bias that potentially could be injected into the AADT estimating procedure and to assure that points selected would be representative of the entire statistical population to be sampled, it was decided to adhere as closely as possible to the concept of randomness in choosing future continuous-count locations for Category I. Such a procedure was possible in this category because the majority of eligible road sections had been assigned to one of the four behavior patterns (populations). From the standpoint of statistical theory, the locations could be considered randomly selected if the choosing process allowed every road section within the population an equal chance of being selected. The actual process of selecting Category I stations required that the purely random concept be modified to a degree because of the telemetry system's requirement for the presence of electric and telephone service at each of the station locations. Another consideration was the desire to have as much assurance as possible that station locations finally selected would, in fact, produce the pattern of factor data that had been expected. This involved an examination of the history of each road section's grouping for a period of 4 years (1961-64). Sections showing a significant tendency to transfer from one group to another were eliminated from the base from which future locations were drawn. It is thought that this procedure provided the most satisfactory method of locating the 16 Category I stations in terms of minimizing bias as well as conceding necessary considerations to practicality.

Past studies have indicated that it is impossible, within practical limits, to design a program for low-volume rural roads (Category II) that will produce estimates of AADT that are as accurate as those for high-volume roads. However, the same level of accuracy is not ordinarily required on these road sections. In Georgia, it was decided to explore the area-control method of producing AADT estimates on Category II road sections. This method implies that because of similarities in economic activity, climatic conditions, population densities and other related factors the monthly distribution of traffic flow would be reasonably constant throughout the designated area.

Since historical continuous-count data were very limited on low-volume roads, the initial step has been to divide the state into three areas generally described as Mountains and Upper Piedmont, Lower Piedmont, and Coastal Plain. Utilizing a probability procedure similar to that used in Category I, one continuous-count station was located in each area. Additionally, three seasonal-control stations operated for a 7-day period in each month were established on low-volume road sections in each of the three areas. Such an arrangement will provide monthly data from four points in each area that can be grouped to produce a set of group mean monthly factors. This procedure, established January 1, 1966, will be subject to extensive analysis once sufficient lead data are accumulated.

The selection of Category III was not subjected to any probability procedure. A general administrative criterion was established that the scope of the program should be such as to provide at least one continuous-counting point on each major segment of the Interstate System in Georgia. This dictated approximately 9 Category III continuous-count stations. Where possible these locations are being established to allow correlation with historical data produced by locations on the former Interstate travel-way and to facilitate evaluation of Interstate design traffic assignment and forecasting techniques. For purposes of AADT estimating, it is expected that these road sections will identify with the patterns established in Category I. However, a separate categorizing of Interstate sections may ultimately permit a refinement of estimating procedures for this category.

Continuous-count data accumulated by the telemetry system are, as described earlier, edited and filed on disk packs. These data are immediately accessible, through programming, for any analysis that may be desired. There are, however, certain analyses that are performed routinely to satisfy data needs of the Department as well as to fill the U. S. Bureau of Public Roads' requirements. Figure 14 shows an example of the monthly summary of hourly data obtained at a continuous-count location. Volumes are summed for the entire month and for each day of the month individually. The average weekday, Saturday, Sunday and day of the month count is computed and listed. At directionally counted stations, a summary is made for each direction separately and for both directions combined. Using manual retrieval, coding, keypunch and card input procedures, approximately two weeks were required to produce these summaries for

STATE HIGHWAY DEPARTMENT OF GEORGIA
DIVISION OF HIGHWAY PLANNING

IN COOPERATION WITH
U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
MONTHLY CONTINUOUS TRAFFIC RECORDER DATA

JANUARY, 1965

STATION 8

LOCATION - US-80, 6.2 MI. NW. OF STATESBORO

NON-DIRECTIONAL

DAY OF MO	WK	1AM	2AM	3AM	4AM	5AM	6AM	7AM	8AM	9AM	10AM	11AM	12M	1PM	2PM	3PM	4PM	5PM	6PM	7PM	8PM	9PM	10PM	11PM	12PM	DAILY COUNT
1	6	65	57	38	28	27	31	46	83	112	155	150	148	150	157	167	193	196	192	195	149	109	80	62	58	2648
2	7	46	40	21	21	25	20	27	101	128	149	221	221	212	186	213	216	238	226	215	171	102	87	97	88	3071
3	1	56	42	28	26	22	25	25	30	96	153	206	203	216	249	342	435	413	371	293	260	156	114	91	82	3934
4	2	43	41	26	24	40	52	101	164	171	187	189	173	175	184	192	180	189	220	197	153	85	83	61	65	2995
5	3	39	39	30	42	39	49	79	154	127	171	183	157	172	155	171	173	210	234	220	139	86	82	68	39	2857
6	4	37	33	36	22	42	39	63	155	173	253	163	173	145	191	185	169	241	234	193	119	92	96	75	55	2984
7	5	52	42	26	26	35	37	86	180	144	176	188	188	152	178	200	184	209	220	172	112	78	65	64	39	2853
8	6	53	36	19	31	22	45	67	58	169	159	161	158	159	159	181	212	218	264	257	202	112	113	98	83	3036
9	7	82	46	40	25	37	13	47	98	151	188	192	185	225	226	206	214	210	205	207	152	94	83	83	79	3088
10	1	62	39	19	11	18	24	17	53	62	110	155	173	186	156	184	220	249	264	251	146	104	106	81	69	2759
11	2	40	28	27	34	41	46	75	159	157	160	183	159	169	138	160	197	212	234	185	151	75	62	101	55	2848
12	3	45	43	27	32	32	47	68	157	140	161	154	158	143	152	158	186	200	206	211	150	88	84	88	73	2803
13	4	57	47	18	28	47	38	53	144	145	183	161	189	187	187	189	192	216	219	185	137	80	79	65	48	2894
14	5	40	34	31	27	32	40	78	164	143	166	182	173	187	159	162	177	201	256	191	98	71	72	71	66	2821
15	6	37	49	26	26	28	43	66	155	151	175	186	169	182	185	224	282	270	314	245	171	95	95	94	77	3345
16	7	56	29	34	16	27	18	52	114	136	164	203	158	194	170	207	218	205	167	149	108	64	61	61	62	2673
17	1	50	27	19	23	17	11	11	28	36	79	134	156	139	133	225	243	293	257	237	167	113	113	63	49	2623
18	2	73	33	28	23	42	51	81	158	149	131	159	145	175	138	200	173	212	198	180	125	96	68	76	55	2769
19	3	50	48	40	42	29	48	79	140	157	169	149	154	161	142	171	168	207	234	168	125	103	85	65	55	2789
20	4	59	49	29	39	45	49	69	140	165	168	155	175	147	201	179	216	200	225	175	126	84	85	64	40	2884
21	5	42	30	45	44	52	42	89	152	153	178	192	186	157	166	156	194	194	233	193	116	94	74	61	51	2894
22	6	32	29	27	28	17	36	70	156	185	159	181	183	178	214	191	261	276	281	251	218	114	118	100	71	3376
23	7	75	37	42	20	40	30	69	121	137	190	226	194	198	220	195	200	232	213	198	160	101	61	103	81	3143
24	1	70	34	16	13	16	13	23	46	58	107	145	152	143	139	211	270	312	319	258	188	153	116	87	67	2956
25	2	52	27	22	32	37	39	79	183	167	211	157	169	187	141	160	192	205	223	226	142	89	68	95	60	2963
26	3	46	36	25	36	18	40	70	137	161	180	147	164	183	161	160	189	221	237	223	204	97	127	109	64	3035
27	4	51	33	33	32	40	49	85	136	170	157	178	213	159	154	169	171	217	258	203	156	116	72	90	56	2998
28	5	35	42	41	41	44	71	81	159	162	172	181	132	185	186	196	208	215	253	216	124	98	93	89	50	3124
29	6	56	40	31	39	37	37	55	169	170	189	171	179	161	198	201	242	271	284	259	228	139	153	88	91	3488
30	7	51	39	47	34	26	30	47	103	158	190	199	209	207	215	208	225	246	172	174	154	102	78	75	71	3060
31	1	54	46	31	17	20	14	17	34	45	92	170	179	152	153	249	299	357	322	269	182	145	133	68	68	3116

AVERAGE - WEEKDAY 2972
SATURDAY 3007
SUNDAY 3078
DAY OF MONTH 2994

NUMBER OF DAYS IN MONTH - 31

TOTAL COUNT FOR MONTH - 92827

Figure 14. Example of monthly computer summary of data obtained at a continuous-count station.

STATE HIGHWAY DEPARTMENT OF GEORGIA
DIVISION OF HIGHWAY PLANNING

IN COOPERATION WITH
U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
ANNUAL RECAPITULATION
OF
CONTINUOUS TRAFFIC RECORDER DATA
FOR
1965

STATION 008

LOCATION - US-80, 6.2 MI. NW. OF STATESBORO

NON-DIRECTIONAL

MONTH (COL.1)	MONTHLY TOTAL (COL.2)	AVERAGE WEEKDAY (COL.3)	AVERAGE SATURDAY (COL.4)	AVERAGE SUNDAY (COL.5)	AVERAGE DAY OF MONTH (COL.6)	AVG. DAY OF MO. TO AVG. WEEKDAY (COL.7) (COL.6/COL.3)	AVG. DAY OF MO. TO AVG. DAY OF YEAR (COL.8) (COL.6/A.D.T.)	AVG. DAY OF YEAR TO AVG. WEEKDAY (COL.9) (A.D.T./COL.3)
JANUARY	92827	2972	3007	3078	2992	1.007	.862	1.168
FEBRUARY	86375	3069	3140	3107	3085	1.005	.889	1.131
MARCH	99500	3204	3267	3188	3211	1.002	.925	1.084
APRIL	101716	3332	3433	3668	3394	1.019	.978	1.042
MAY	104597	3321	3407	3564	3368	1.014	.970	1.045
JUNE	112447	3632	3983	4155	3757	1.034	1.082	.956
JULY	127896	4010	4501	4293	4121	1.028	1.187	.866
AUGUST	125354	3929	4223	4406	4039	1.028	1.163	.884
SEPTEMBER	104582	3447	3808	3377	3489	1.012	1.005	1.007
OCTOBER	110121	3514	3651	3613	3548	1.010	1.022	.988
NOVEMBER	100100	3293	3259	3655	3340	1.014	.962	1.054
DECEMBER	103118	3396	3175	3077	3319	.977	.956	1.022
DAILY AVERAGE FOR THE YEAR		3427	3571	3598	3472	1.013		

Figure 15. Annual computer summary of continuous-count data.

1965 HIGH HOUR DATA
CONTINUOUS COUNT STATION NO. 008
LOCATION-US-80, 6.2 MI. NW. OF STATESBORO

A.D.T. = 3472

ORDINAL HIGH HOUR	DATE OF OCCURRENCE DAY OF WK	MO.	DAY	YR.	HR.	HIGH HOUR VOLUME	DIRECTIONAL DISTRIBUTION	K FACTOR
1ST	7	07	31	65	11	509	69	14.7
2ND	1	06	06	65	18	501	69	14.4
3RD	1	11	28	65	17	470	56	13.5
4TH	1	11	28	65	18	450	59	13.0
5TH	7	07	31	65	10	447	63	12.9
6TH	4	11	24	65	16	442	63	12.7
7TH	1	11	28	65	16	442	51	12.7
8TH	1	01	03	65	16	435	55	12.5
9TH	6	10	29	65	18	431	67	12.4
10TH	6	10	01	65	17	423	65	12.2
15TH	1	11	28	65	19	412	71	11.9
20TH	1	06	27	65	16	400	59	11.5
25TH	1	04	18	65	19	395	64	11.4
30TH	6	10	01	65	19	385	67	11.1
35TH	1	08	22	65	10	382	60	11.0
40TH	1	08	15	65	17	375	50	10.8
45TH	6	09	10	65	20	370	63	10.7
50TH	6	10	15	65	17	366	60	10.5
75TH	7	08	14	65	12	352	63	10.1
100TH	6	08	27	65	18	339	59	9.8
125TH	1	02	28	65	18	329	58	9.5
150TH	6	10	15	65	16	323	55	9.3
175TH	6	08	06	65	16	318	54	9.2
200TH	6	08	20	65	18	312	61	9.0

Figure 16. Annual computer summary of high-hour data.

STATE HIGHWAY DEPARTMENT OF GEORGIA
DIVISION OF HIGHWAY PLANNING

IN COOPERATION WITH
U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
SEASONAL CONTROL COUNTS

SAMPLE NO. 06,1966

STATION 037-0204-3

LOCATION I-75-1, N. OF INTERC. WITH FAS-1950 N. OF CFCIL

* * * * * * * C O U N T F O R H O U R P E R I O D E N D I N G * * * * * * * * *

DAY OF MO	WK	1AM	2AM	3AM	4AM	5AM	6AM	7AM	8AM	9AM	10AM	11AM	12M	1PM	2PM	3PM	4PM	5PM	6PM	7PM	8PM	9PM	10PM	11PM	12PM	DAILY COUNT
													NORTHBOUND													
13	2	70	67	54	54	51	77	130	167	280	379	426	457	416	439	499	400	375	303	245	180	167	148	118	96	5598
14	3	89	94	81	72	99	113	136	202	276	352	400	433	415	482	420	418	387	499	260	189	166	140	110	126	5959
15	4	112	87	76	68	81	121	137	225	263	378	415	478	461	458	469	487	465	355	307	217	163	141	145	125	6234
16	5	86	104	85	75	68	109	149	239	373	406	488	526	474	505	534	427	486	423	305	241	172	173	153	119	6720
17	6	127	103	109	87	96	106	178	221	378	471	531	427	448	502	616	594	577	509	392	391	309	277	265	216	7930
18	7	213	181	168	160	115	198	224	322	445	541	497	663	575	604	508	470	369	384	272	263	204	220	167	145	7908
19	1	98	106	88	80	94	78	138	184	248	416	531	524	472	511	541	541	517	459	334	274	204	174	152	117	6881
		AVERAGE WEEKDAY			6488								AVERAGE DAY		6747							SAMPLE TOTAL -			47230	
													SOUTHBOUND													
13	2	105	131	93	102	104	115	173	271	421	503	629	609	418	473	462	506	419	405	317	223	173	131	130	102	7015
14	3	84	97	78	65	64	63	122	209	302	412	432	449	378	399	425	386	359	353	327	214	175	119	91	113	5716
15	4	87	89	74	77	72	88	94	207	259	337	443	316	372	331	337	387	330	298	258	174	141	107	131	105	5114
16	5	103	117	84	66	87	85	141	202	300	342	402	351	303	343	360	366	344	327	289	208	173	149	140	93	5375
17	6	82	74	86	86	59	82	123	201	234	427	433	421	391	388	452	495	457	406	323	343	258	253	207	178	6459
18	7	180	150	144	145	156	164	217	282	404	583	648	606	506	530	575	568	562	457	362	369	234	188	160	143	8333
19	1	122	80	107	121	96	145	195	334	459	603	766	724	663	660	698	725	619	599	529	375	281	245	182	136	9464
		AVERAGE WEEKDAY			5936								AVERAGE DAY		6782							SAMPLE TOTAL -			47476	
													NON-DIRECTIONAL													
13	2	175	196	147	156	155	192	303	438	701	882	1055	1066	834	912	961	906	794	708	562	403	340	279	248	198	12613
14	3	173	191	159	137	163	176	258	411	578	764	832	882	793	881	845	804	746	852	587	403	341	259	201	239	11675
15	4	199	176	150	145	153	209	231	432	522	715	858	794	833	789	806	874	795	653	565	391	304	248	276	230	11348
16	5	189	221	169	141	155	194	290	441	673	748	890	877	777	848	894	793	830	750	594	449	345	322	293	212	12095
17	6	209	177	195	173	155	188	301	422	612	898	964	848	839	890	1068	1089	1034	915	715	734	567	530	472	394	14389
18	7	393	331	312	305	271	362	441	604	849	1124	1145	1269	1081	1134	1083	1038	931	841	634	632	438	408	327	288	16241
19	1	220	186	195	201	190	223	333	518	707	1019	1297	1248	1135	1171	1239	1266	1136	1058	863	649	485	419	334	253	16345
		AVERAGE WEEKDAY			12424								AVERAGE DAY		13529							SAMPLE TOTAL -			94706	

Figure 17. Computer summary of a 7-day seasonal-control station sample.

all stations. It is expected that approximately 1.5 hours will be required using the telemetry-disk input system.

Each month, as monthly summary reports are prepared, a summary card is automatically punched for each station. These summary cards are then used at the end of the year to produce an annual recapitulation of the station's data. Figure 15 shows this annual summary.

Programs have also been developed to extract selected high-hour data from the year's continuous-count records. The volumes, along with data related to time of occurrence, directional distribution (where available), and ratio of high-hour volume to AADT (K), are shown by Figure 16.

The Seasonal-Control Counting Program

The reasons for operating seasonal-control counting stations within the framework of the overall program may be stated as follows: (a) as a substitute for continuous-count stations to produce factor and trend data (Category II is an example); (b) to establish patterns of seasonal variation on those road sections not yet assigned to pattern groups; and (c) to study seasonal patterns in areas thought to be subject to change due to the opening of new facilities or other reasons.

The method of operating rural seasonal-control stations is to obtain a 7-day sample during each month with an hourly recording device. The initial scope of the program was dictated by equipment inventories and existing organizational capability which permitted the operation of 146 stations, 5 of which are counted directionally. During 1966, the initial year of operation, 9 locations were assigned to reason (a), 127 to reason (b), and 10 to reason (c). It is expected that all rural road sections will be grouped within a period of 2 to 3 years, after which a very great reduction in the effort devoted to seasonal-control counting activities can be effected.

Routine analysis of seasonal-control data will be very similar to that for continuous-count data. Programs have been developed to prepare a hard copy of each month's sample data (Fig. 17), as well as a summary data card that can be used for routine annual summarizing similar to that for continuous-count data.

There are several considerations under study that will permit further automation of the seasonal-control counting program, including the installation of induction loops in lieu of pneumatic road tubes for vehicle detection. These detecting devices would be used in conjunction with portable, punch tape hourly recorders that could be moved between locations as scheduling dictated. It is believed that available on-line equipment, with certain design modifications, would allow data to be introduced directly into computer core storage from the punched paper tape produced by the recorder thus eliminating existing coding, keypunch and manual editing procedures.

The Coverage-Count Program

The coverage-count program involves obtaining necessary short-term counts to produce annual estimates of AADT on all road sections for which these data are required. The design goal for the program was that it should produce such estimates, along with vehicle-mile summations, for all rural Federal-aid Interstate, Primary and Secondary, as well as state and major county road system sections. Special traffic survey maps were prepared that divided each county's defined road system into identified road sections (Fig. 18). A representative counting point was established for each section and the section's length determined.

Each of the coverage-count stations is operated annually for a weekday period of 24 hours either with an accumulative or hourly recorder. The sample count obtained, after having been compared to data from previous years to determine acceptability, is coded along with location identification, date obtained, peak-hour volume when available, section length and highway system. Also coded is the category and group designation for each road section. The coded data are keypunched and processed by the coverage-count computer program. An example of the program's printer listing is shown by Figure 19.

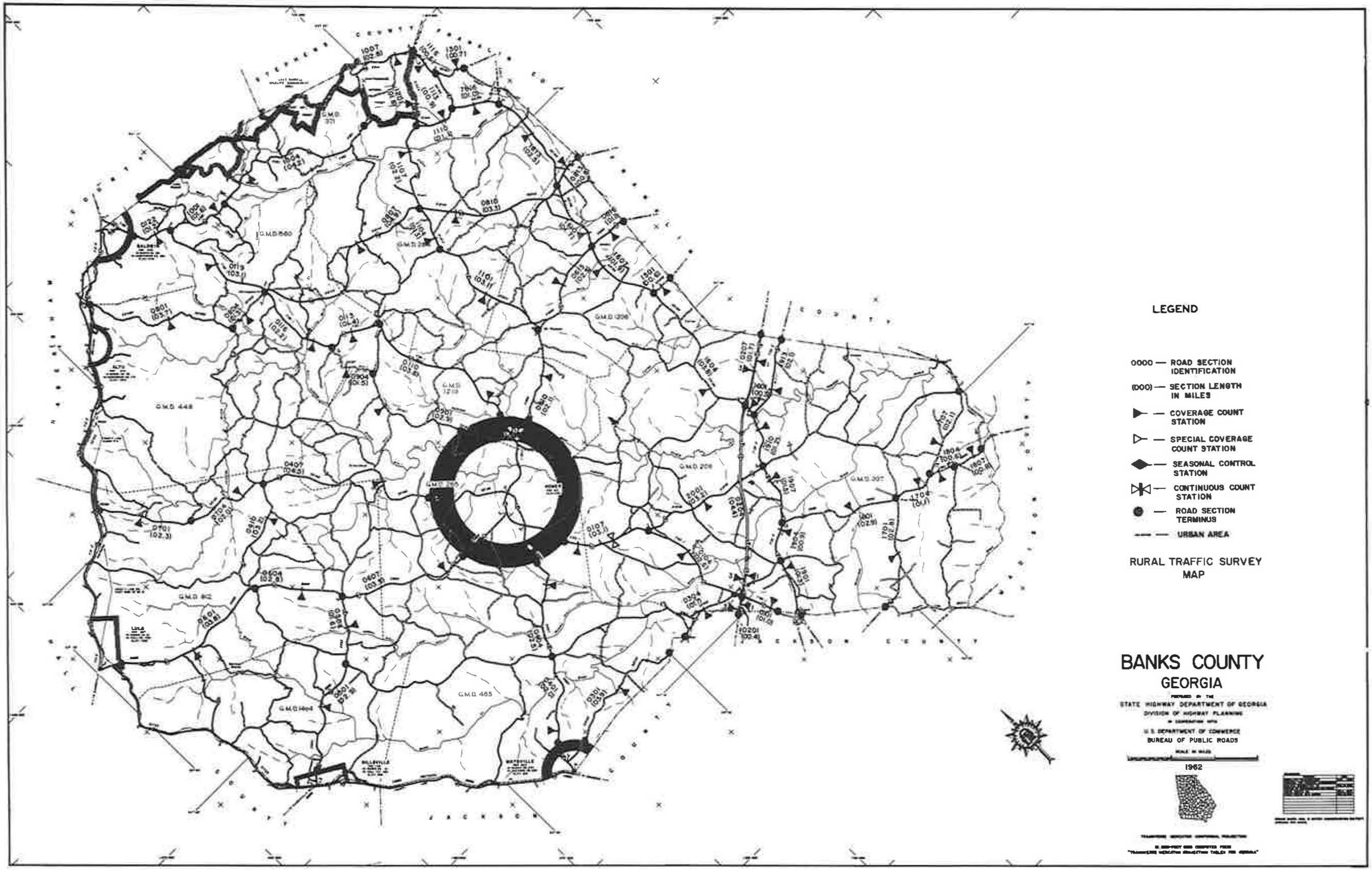

Figure 18. Rural traffic survey map, Banks County, Georgia.

Form HPS-7-41

STATE HIGHWAY DEPARTMENT OF GEORGIA
DIVISION OF HIGHWAY PLANNING
In cooperation with
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

BANKS COUNTY

1966 COUNTY COVERAGE
AND
ESTIMATES OF A.D.T.

1	2	3	4	5		6	7	8	9	10	11		12	13	14	15	16	17	18	19	20
				Pattern												Vehicle Classification Data					
Section Ident. Number	Serial No.	Direction	Highway System	Category	Group	Date Count Obtained (Mo. Day Yr.)	Day of Week	Sample Count	Peak Hour Percent	D.	Group Mean Adj. Factors: Weekday	Month	Estimated A. D. T.	Section Length (miles)	Estimated Daily Vehicle Miles	Last Date Observed (Mo. Day Yr.)	Percent Trucks 24-Hrs.	Percent Trucks in Peak Hour	D.	Percent Local P. C.	Percent Out-of-State P.C.
0101		0	03	1	3	1/13/66	5	3,268			1.04	1.41	4,792	1.0	4,792						
0104		0	03	1	3	1/11/66	3	1,822			1.06	1.41	2,723	2.6	7,080						
0107		0	03	1	3	1/11/66	3	1,614			1.06	1.41	2,413	3.1	7,480	1/12/66	12.3	6.6	51	65.1	7.4
0110		0	03	1	3	1/11/66	3	1,217			1.06	1.41	1,819	3.8	6,912						
0113		0	03	1	3	1/11/66	3	1,180			1.06	1.41	1,764	1.4	2,470						
0116		0	03	1	3	1/11/66	3	1,120			1.06	1.41	1,674	2.2	3,683						
0119		0	03	1	3	1/06/66	5	2,203			1.04	1.41	3,230	3.1	10,013						
0122		0	03	1	3	1/06/66	5	2,045			1.04	1.41	2,998	1.2	3,598						
0201		1	01	3	1	1/13/66	5	4,340			1.03	1.16	5,185								
0201		3	01	3	1	1/13/66	5	4,001			1.03	1.16	4,780								
0201		0	01	3	1	1/13/66	5	8,341	5.3	58			9,965	.4	3,986						
0204		1	01	3	1	1/13/66	5	4,421			1.03	1.16	5,282								
0204		3	01	3	1	1/13/66	5	4,075			1.03	1.16	4,869								
0204		0	01	3	1	1/13/66	5	8,496					10,151	4.4	44,665						
0207		1	01	3	1	1/12/66	4	3,880			1.03	1.16	4,636								
0207		3	01	3	1	1/12/66	4	3,808			1.03	1.16	4,550								
0207		0	01	3	1	1/12/66	4	7,688	5.6	52			9,186	1.7	15,616						
0301		0	07	2	2	1/13/66	5	116			1.07	1.15	142	3.9	554						
0304		0	07	2	2	1/13/66	5	76			1.07	1.15	93	1.7	158						
0401		0	05	2	2	1/14/66	6	436			.92	1.15	461	2.0	922						
1907		0	03	2	2	1/11/66	4	373			.98	1.15	420	1.5	630						
1910		0	03	1	1	1/12/66	4	658			1.03	1.16	786	1.2	943						
1913		0	03	2	2	1/12/66	4	250 *			.98	1.15	282	2.1	592					PAGE 3	
2001		0	05	2	2	1/11/66	3	198			1.07	1.15	244	3.2	781						
												SYSTEM	01 TOTAL	6.5	64,267						
												SYSTEM	03 TOTAL	25.4	49,785						
												SYSTEM	05 TOTAL	60.4	20,067						
												SYSTEM	07 TOTAL	37.9	5,874						
												SYSTEM	09 TOTAL	8.1	2,049						
												COUNTY	TOTAL	138.3	142,042						

*INDICATES ESTIMATED SAMPLE COUNT

Figure 19. Computer listing of rural coverage counts and estimates of ADT.

To accomplish this listing, the following operational steps are noted:

1. County names are stored for on-line disk reference. The appropriate name is selected according to a numeric code in the data card and printed at the top of each page.
2. Data fields 1 through 8 are listed directly from the data card. Data for directional coverage stations are listed for each direction individually and a total or "0" line is produced by the program.
3. A value will appear in data field 9 when the sample count is obtained with an hourly recording device. This value is the ratio, expressed as a percent, computed by dividing the peak hour by the 24-hour total.
4. A value will appear in data field 10 (directional distribution) when directional hourly data are obtained, and a ratio is computed by dividing the total peak-hour volume into the volume moving in the direction of dominant flow.
5. Data field 11 lists the group mean adjustment factors used to convert that particular sample count into an estimate of AADT. All factor data for a year are filed on disk storage. The appropriate factors are referenced by noting the month and day of the week on which the sample is obtained, along with the category and group with which the subject road section is associated.
6. Data field 12 shows the computed estimate of AADT (field 8 × field 11).
7. Data field 13 is the road section length in miles and is listed directly from the data card.
8. Data field 14 shows the computed estimate of average daily vehicle-miles for the subject road section (field 12 × field 13). Data fields 13 and 14 are summed by system by county.
9. Data fields 15 through 20 list directly from the data cards a summary of significant factors related to the most current vehicle classification observation.

In Figure 19, 1966 sample data are used to produce 1966 estimates of AADT. The factor data used, however, are extracted from summaries of 1965 continuous-count data. Therefore, the estimates of AADT produced during the same year that sample data are obtained must be considered preliminary. Final 1966 estimates can be obtained after all 1966 continuous-count and seasonal-control data records are summarized to produce current road-section groupings and an updated disk file of adjustment factors. There will probably be a small number of road sections that will change groups from one year to the next. Adjustments for this can easily be made by modifying one digit in the coverage count data card for the affected section. Once disk and card files are updated, approximately 45 sec per average 75-station county is required for processing final estimates.

Because previous studies have shown pattern groupings and resulting group mean factors to be reasonably stable from year to year, the immediate availability of preliminary estimates of AADT should compensate for any error in the estimates that might result from using factor data and road-section groupings from the previous year. In any event, estimates can be easily finalized immediately after the end of the calendar year. The same disk file of factors used to finalize estimates of AADT for the previous year can then be used to produce preliminary estimates of AADT in the current year.

The weekday factors that appear in data field 11 are the group mean ratios of the average weekday for a given month to the average specific weekday, Monday through Friday, for the same month. The inclusion of these factors in the estimating procedure, in effect, first adjusts the sample count to an estimate of average monthly weekday traffic which, in turn, is adjusted by the group mean monthly factor to an estimate of AADT. A number of statistical tests have indicated that the use of group mean weekday factors provides a logical refinement to the estimating procedure. This is illustrated by the fact that the standard deviation of estimating errors was reduced from ± 11.7 percent without weekday factors to ± 8.2 percent with weekday factors.

CONCLUSIONS

The employment of documented computer programs in the collection and editing of continuous traffic-count data permits full control of standardized statistical procedures with a minimum of supervision. Since these data are edited and added to a historic file as they are collected, traffic reports necessary for continuing highway-planning activities can be readily compiled as the necessity arises.

Any estimation of hourly volumes that may be required due to an inability to poll a particular continuous-count station is more accurate than an estimation customarily supplied by traffic coders, since programmed computer estimates can be based on vast amounts of stored historical traffic data. In addition, these estimates of hourly volumes will be less in total number and concentration due to the ability to promptly detect any malfunctioning counting station. This scattering of estimates has resulted in unaffected 24-hour total volumes 98.3 percent of the time. Also, the 1.7 percent of affected 24-hour volume totals are more accurate than 24-hour totals containing manually estimated hourly volumes.

The continuous-count program, integrated with remote capabilities of data acquisition, can be expanded, when necessary, with nominal increases in cost and essentially no increase in the labor force. By taking advantage of the flexibility of a stored computer program, any given counting station can be polled as often as the location may dictate to assimilate varying time intervals of traffic accumulations. Routine reports compiled from these data can be produced in a greatly reduced number of man-hours, because manual calculations of volume estimates, coding of printed traffic recorder tapes, and keypunching of coded data are eliminated.

The revised seasonal-control count program is initially being conducted on an extensive scale until such time that all road sections within the state are classified into groups of similar traffic variances. After this classification has been established, the extent of the program can be reduced without sacrificing any of its benefits.

Since seasonal counts are compiled at each location for 7 days during each month of the year, it is impractical to attempt to conduct this type of traffic count at all locations where traffic is not being measured by the continuous-count program. Therefore, after necessary factors essential for estimating AADT have been extracted from continuous and seasonal counts, the seasonal-counting program can, for the most part, be superseded by annual 24-hour counts obtained in the coverage-count program. These counts, which are systematically collected throughout the state each year, are sufficient for estimating statewide AADT volumes by mechanical application of factors calculated from the continuous-count program. The coverage-count locations and their resulting vital traffic statistics can, through the implementation of computer programming, be computed promptly and tabulated in a form desirable for reproduction and distribution.

The telemetry system, in conjunction with revisions in the seasonal-control counting and coverage-counting programs, has permitted a considerable expansion of the traffic-counting and analysis program within the framework of the existing organization. Without major revisions in the total traffic-counting program, maximum effectiveness of an advanced system of traffic data acquisition could not have been achieved.

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Discussion

PATRICK J. ATHOL, Illinois Expressway Surveillance Project—This paper is an excellent report on the traffic-counting program undertaken by the Georgia Highway Department. Their approach toward automation has proved its success in improved accuracy and greater economy, and it is a credit to the Highway Department and the cooperating manufacturers who undertook the work.

The question for discussion is the applicability of this system to other states. From a technological viewpoint, the demands of a traffic-count program in terms of data transmission and computer control are relatively straightforward. In the Georgia scheme, there were two important decisions, the computer configuration and the interconnect mode, which were dictated by local conditions. The computer development appeared governed by the fact that the planning department already had its own machine and the development of the interconnect system was influenced by the regular WATS line service already used by the Highway Department.

The computer system developed from the original IBM 1401 to an IBM 1620, and then to the control and data-collection system designated as the IBM 1710. The 1710 system used depended heavily on disk storage for both programs and data; this greatly expanded the utility of the system over the basic 1620 system. These additional items were necessary to expand the system capability for real-time control and data collection.

The WATS line is a rate schedule rather than a specific data-transmission specification. The Georgia system uses a conventional telephone system to dial individual counting stations. Each station has a number which is effectively the same as any house phone number. The count data transmission has no priority in this system and if sufficient highway personnel keep telephone lines busy, then they prevent the count data system from working. If an automatic call is placed for traffic data at a time when the telephone company's exchanges are busy, then the phone call is switched within the telephone system to various links. The circuitous routing through various exchanges may result in electrical noise problems which impair the accuracy of the data transmitted. Built-in automatic error checking ensures the rejection of erroneous data, but this does not prevent the loss of data in its then present form.

Developments in the small computer field have been so rapid and numerous that existing systems are quickly superseded by new equipment with better performance at reduced cost. Some of the greatest advances have been in the area of small real-time control computers. The agency considering an economical traffic acquisition system should consider a specialized computer for this use. A separate computer in a real-time system assures guaranteed priorities to the task undertaken and cannot be pushed off-line for administrative emergencies which so often preempt data-processing computer time. At a rental rate of from 1 to 2 times a senior engineer's monthly salary, a very complete and adequate range of computer systems is available. Rental pricing on control computers is based on full-time operations and does not involve additional shift times to provide continuous year-round operations.

In comparing the speed of performance of the computer and the telephone systems, it is quite noticeable how one deals with microseconds with a computer while the telephone system is functioning in terms of seconds. The reason for this apparent system discrepancy is one of economics. The polling of various count stations using a WATS line is the most economical method of gaining data; but one could operate at much faster rates, and with less interface equipment, if there were direct connections between the field detectors and the digital computer. With a direct hookup with each detector, one could gather the traffic detector signals at very high speeds and keep tally of the detector signals within the computer itself; this technique eliminates the needed equipment for dialing and storing data at the count station and in the central office. Within the framework of present highway operations and concepts, the cost of earmarked communication links initially appears prohibitive. If, however, in the design of the count system, adequate coordination with a total statewide communication system is provided, then the direct communications system to the computer may be economically feasible.

The challenge to any group undertaking the future design of a system for traffic counting will be greatly enhanced if the group can look forward to the total developing electronic needs of the highway department. Planning for a comprehensive approach to many of these communication and automation needs will enhance the long-term utility of most individual systems undertaken.

W. C. TAYLOR, Traffic Research Engineer, Ohio Department of Highways—The authors have done a fine job of describing the techniques used for automatic acquisition and analysis of traffic data in Georgia. Their conclusion that this technique represents a major step forward in providing highway planning data is incontestable. However, in reviewing this paper several questions regarding the implementation of the technique arise. Answers to these questions may be available from the information which the authors possess, but which did not find its way into the report. The purpose in raising these questions is to elicit answers so that the technique might be more easily adopted in other locations.

Specifically, two points are presented in the introduction as justification for embarking on a program of automatic data acquisition and analysis. The first point was the economic limitation of expanding the manual-count method. This is probably a valid point, but no cost data for either the present system or the proposed system are presented. It would be helpful if estimates of the annual cost of both systems for several different information levels were presented.

Secondly, it was noted that some 13 percent of the count data was being lost due to equipment failures in the manual data-processing method. Yet, after one year, the data loss on the four automatic counters used in the test was still 11 percent (Fig. 11 of the paper). In fact, during March 1966, this figure reached as high as 30 percent (Fig. 9 of the paper). In reading the paper, I suspect that the reasons for this high data loss were determined and corrected.

Prerequisites for locating the test sites included varying distances from the office, diverse climate conditions, and different telephone systems. A valuable addition to the paper would be a discussion of the effect of these variables, and the reasons for the detector changes indicated. We are told that the only magnetic detector used was replaced, and that one of the pneumatic tubes was removed in favor of an induction loop, but the reasons for these changes are not presented.

The point which disturbs me the most is the use of the expected tolerances as a check of data validity. The text reads that the tolerance presents the square root of the average of the square of a set of deviations about an arithmetical mean. If I understand the wording of the text properly, the tolerance limits are set at $\pm 1\ \sigma$. If you assume a standardized normal distribution, these limits include only about 70 percent of the data points. On the other hand, an equipment malfunction, if not complete, might not be detected. The tolerance limits are wide enough to permit a 16 percent deviation from the mean value without detecting a malfunction. The use of only one set of limits leads to this dilemma.

The point I am raising here refers to the conclusion that this technique provides increased accuracy over the manual-processing method. In the manual-processing method the authors indicated a known loss of 13 percent of the data, while the automated system checks found only an 11 percent loss of data. I would contend that these figures refer only to total losses, not to erroneous inputs. I have a suspicion, admittedly unconfirmed, that manual data-processing techniques would identify more erroneous inputs than can be found by the tolerance limit method of analysis.

JACK C. MARCELLIS, Assistant Traffic Engineer, City of Chattanooga, Tennessee—If a traffic-counting program is going to provide the required information at the appropriate time to the many highway department agencies, the data for the continuous-count, seasonal-control and coverage-count programs must be collected, processed and analyzed in an efficient and economical manner. The question is then asked: Does the current State Highway Department of Georgia traffic-counting program accomplish these two criteria in a better manner than did the previous program?

In the area of data collection for the former continuous-count program, it was observed that 13 percent of the data was being lost due to equipment failure between weekly or biweekly maintenance visits. During the 12-month study period of the telemetry system, volumes due to unsuccessful polls represented only 11 percent of the total, and after recent modifications at the counting station, successful polling of traffic volumes had increased to 93 percent. These unobtained volumes were randomly scattered throughout the counting duration instead of being grouped for large periods of time as in the manually collected system. These occurrences led to easier and more accurate estimation of the missing traffic volumes.

Two weeks were required to produce monthly summaries for all continuous-count stations using manual coding, keypunch and card input procedures. It was estimated by the authors that only 1.5 hours will be needed to perform the same task using the telemetry disk input system.

It is obvious that the telemetry system collects, processes and analyzes continuous-count traffic data in a more efficient manner than did the old manual methods. Still unanswered are the following questions: Is the telemetry system more economical than the previous method? Does the saving in data collection and processing personnel offset the capital and operation costs of a computer and telephone equipment?

Both the seasonal control and coverage-counting programs used pneumatic road tubes and hourly or accumulative recording devices for collecting traffic data, manual coding, keypunch and editing, and for processing traffic data and computer programs for obtaining summaries and AADT estimates. These two phases of the traffic-counting program require more traffic data on a station-day counting basis than does the continuous-count portion. Because of this, can one or more of the manual steps be eliminated and in turn improve greatly the efficiency of the total program?

Further automation of the seasonal-control counting program is currently under consideration. The authors have indicated that existing on-line equipment might be modified to allow traffic-count data to be introduced directly into the computer core storage from the punch paper tape produced by the hourly traffic counter, thus eliminating existing coding, keypunch and manual-editing procedures. Could something similar to this be used with the coverage-count data?

This author does not pretend to know the answers to these questions, but only asks them to stir the intellect of the highway engineering profession. If more efficient and economical methods are developed to collect, process and analyze the continuous-count, seasonal-control and coverage-count programs, the more complete and accurate the traffic data will be and the quicker then data will be ready for use by the various highway department agencies.

The authors are to be complimented on their substantial contribution in improving the traffic-counting program in Georgia. It is hoped that the authors and others like them will be motivated to continue the work of automating the various traffic-counting procedures.

EMORY C. PARRISH, Closure—Both Marcellis and Taylor have made reference to the cost for installing and operating the telemetry system. Prior to the time that the decision was made to go ahead with the proposed telemetry system, a rather comprehensive summary of operating cost was prepared for the procedures then being used. These costs were compared to the anticipated costs for operating the automated system that we have described. These anticipated costs were of necessity only estimates, since it is impossible to determine costs for a system that has never been operated. Any of you who are familiar with the Highway Planning Survey's relationship to the U. S. Bureau of Public Roads and to the overall Highway Department Administration are certainly aware that we could not have begun such a project without a reasonably comprehensive cost analysis.

While I do not have these exact figures before me, I can tell you that they did indicate the telemetry system would operate for about the same as existing procedures when 28 stations were "on line." We expect to ultimately expand the continuous-count program to about 80 stations at which time we anticipate a savings of some \$14,000 annually.

Because we have been—and are—operating under a mixed system, no attempt has thus far been made to evaluate exactly the validity of this earlier cost comparison, but our experience to this point has uncovered no major unexpected costs. I might add that we have not placed a dollar figure on the value of continuous-count data that were lost due to equipment malfunction for days or weeks under the old system.

For information, the average costs for equipment and installation in the field for a nondirectional station are approximately \$600 and for a directional station, approximately \$1100. At each station the leased telephone equipment costs approximately \$21 per month. The rental on telephone equipment in the Atlanta office is \$570 per month which includes \$500 for a WATS line that is used only 1/20th of the time for telemetry. The IBM equipment that adapts our computer to a telemetry system rents for about \$1000 per month.

Mr. Taylor referred to the comparison of lost counts by the two systems as contained in the written report. The counts lost under the manual system were reported as 13 percent of the total and 11 percent using the telemetry system. Although the percentage of unsuccessful polls by the telemetry system was approximately 11 percent of all polling attempts, it should be noted that during this time the experimental Telac device and telephone data sets were constantly undergoing minor design changes. These modifications were not possible until the hardware had been field tested and the unsatisfactory components isolated.

At the present time, operating with very limited replacement parts, the telemetry system as a whole is suffering a loss of only 7 percent of hourly volumes. Using as a guide two of our newest stations which utilize the latest versions of field equipment, we are confident that it will be possible to retrieve hourly volumes a minimum of 95 percent of the time. This results in the necessity for supplying electronically only 5 percent of the volumes. Four percent of this can be accurately recovered by prorating the traffic volume accumulated during the unsuccessful polling period using the midpoints of the traffic ranges for each respective hour during this period. This leaves only approximately 1 percent of hourly volumes having to be supplied by using the midpoints of the ranges.

Mr. Taylor voices a great deal of concern about the use of tolerance ranges to examine the validity of count data. To us, the use of standard deviation seems a logical tool to determine the probable variability of hourly volumes. We have retained a method of introducing human judgment into the final acceptance or rejection of a given count volume. He perhaps has a valid point about the tolerances failing to detect a partial failure of equipment—for example, a detector or counter that was very slightly under- or over-counting. However, our experience with the system, so far, has been that the failures experienced were total; i.e., they just quit working. Numerous manual counts have revealed no tendency toward these "not complete" equipment malfunctions that concern Mr. Taylor.

Mr. Marcellis has challenged us in Georgia and, I think, many of you to devise procedures that would perhaps eliminate some of the manual procedures that are now a part of collecting and processing seasonal-control and average-count data. As I mentioned previously, we have some tentative ideas concerning the seasonal-control program. However, their implementation will be delayed pending replacement of existing equipment. Our budget prevents this before existing inventories are expended.

The Georgia Highway Department has on order an IBM 360 System Computer that will have an optional scanner feature. This may permit the automated reading and processing of coverage-count field notes without manual intervention. Thought in this area is pretty much speculative at this point.

I would also like to mention, in closing, that we have extended many of the concepts advanced in this paper to a traffic-volume counting program for our 53 urban areas in Georgia. This, we think, is going to provide data in an area where we in Georgia have been significantly weak.

Evaluation of Rural Coverage Count Duration For Estimating Annual Average Daily Traffic

R. R. BODLE, Highway Research Engineer, U. S. Bureau of Public Roads

•ESTIMATION of annual average daily traffic (ADT) on sections of a state highway network has long been an important phase of the highway-planning process. ADT estimates have been used as a fundamental element in determining vehicle-miles of travel on the various categories of rural and urban highway systems. ADT estimates, together with other important characteristics of traffic, provide the highway engineer, planner, and administrator with information necessary for establishing a systematic classification of highway systems, determining design standards, evaluating safety programs, estimating change in annual traffic volumes, calibrating traffic assignment and distribution models, and developing programs for highway improvement and maintenance. In addition, many commercial activities, such as motels and hotels, restaurants, automobile service and repair industries, and recreational and amusement centers use traffic estimates as a basis for planning.

Three basic types of mechanical traffic-counting operations are commonly employed by state highway departments to obtain ADT estimates. Hourly recorders are operated continuously at a limited number of locations. Intermittent counts or seasonal-control counts are taken 4, 6 or 12 times a year for durations varying from 48 hours to 2 weeks. The greatest amount of traffic data results from short coverage counts taken for 24 or 48 hours, but may be as long as 5 or 7 days. In a very few states coverage counts are taken 2 or 4 times a year. It is necessary to utilize these coverage counts in arriving at ADT estimates for the many locations on the highway network where continuous recorders and seasonal-control stations are not operated.

Over the years, highway departments have used some factoring procedure for adjusting coverage counts to estimates of ADT. Generally this involved associating each short-count station with a single permanent recorder believed to have a similar pattern of monthly variation. An adjustment factor from the permanent recorder was then applied to the coverage count to obtain the ADT estimate.

In May 1963, the U. S. Bureau of Public Roads published the "Guide for Traffic Volume Counting Manual." This manual was the result of research by several highway departments in cooperation with the BPR, and it presents an efficient procedure for adjusting coverage counts to ADT estimates. The procedure involves grouping together the permanent recorders and seasonal-control stations having similar annual traffic patterns. In most states, three to five groups are defined. After assigning all of the state highway network to one of the several groups that may be identified, coverage counts are adjusted to estimates of ADT by applying the appropriate group mean monthly factor.

The monthly factor used is defined as: F = average annual daily traffic divided by average weekday traffic for the month. The group mean monthly factor is then the average of the individual monthly factors for the ATR's and control stations in the group. There are three sources of error in this method of estimating ADT at a point:

1. The monthly factor at a coverage-count station generally will not be exactly equal to the group mean;
2. The coverage count (24 or 48 hours, 5 days or 7 days) will differ from the average weekday (average day in the case of a 7-day count) of the month; and

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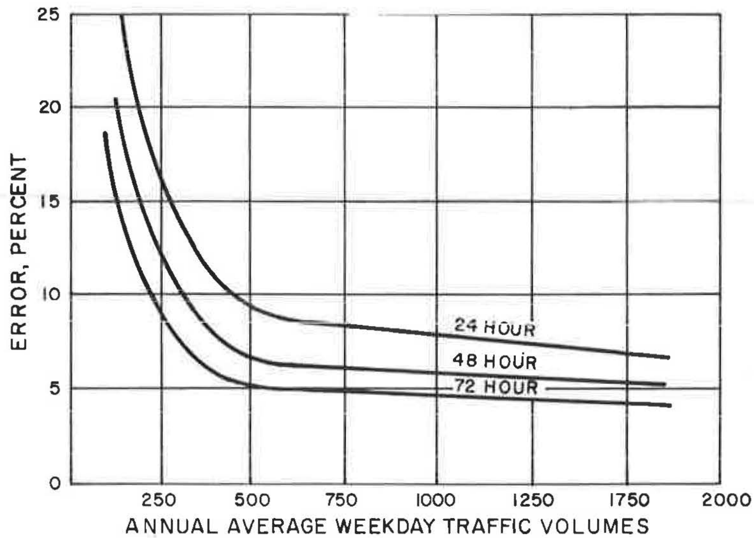

Figure 1. Variation of weekday traffic volumes.

3. The road section on which a coverage count is taken may have been assigned to the wrong group. This error is assumed to be negligible.

The percent error or relative error for any estimate of ADT for a link on which a coverage count is taken may be expressed as:

$$E = \frac{(XF - ADT)}{ADT} 100 \tag{1}$$

where

E = estimated percent error,
X = 24-hour count or the average day for the period counted, and
F = the appropriate group mean monthly factor.

The coefficient of variation of the relative error of the estimate may be expressed as:

$$CV = (CV_x^2 + CV_f^2)^{1/2} \tag{2}$$

where

CV_x = coefficient of variation of coverage counts, and
CV_f = coefficient of variation of the monthly adjustment factors.

Eq. 2 is based on the assumption that both x and f are random variables and the two variables are uncorrelated.

If the continuous traffic recorders were placed on every section of the road network assigned to the group, it may be assumed that the population of resulting monthly factors would be uniformly distributed about the group monthly means. The values for any month would have a range of 0. 20 from low factor to high factor. The coefficient of variation of the factors for a month will generally be 4 or 5 percent. Therefore, to attain a relative error of 10 percent in the ADT estimate (i.e., a coefficient of variation for the estimate of 10 percent), the coefficient of variation of the coverage counts for any month must not exceed 8 or 9 percent.

Many highway departments take coverage counts of 24- or 48-hour duration with cumulative type traffic counters. A few states take coverage counts of 72 hours, or 5 or 7 days. Generally, these longer counts are taken with portable hourly recorders.

TABLE 1

EVALUATION OF 48-HOUR COVERAGE COUNTS
(Georgia)

State	Station	Month	Observations	Avg. Weekday	Std. Deviation	Coef. of Variation
36	1001	1	19.0	9444.0	635.4	6.7
36	1001	2	19.0	9664.5	907.7	9.3
36	1001	3	21.0	9615.4	1394.1	14.4
36	1001	4	21.0	10535.1	658.6	6.2
36	1001	5	20.0	10860.5	658.5	6.0
36	1001	6	21.0	11499.8	1099.0	9.5
36	1001	7	22.0	11871.3	657.0	5.5
36	1001	8	20.0	11893.9	573.3	4.8
36	1001	9	21.0	11035.5	1073.7	9.7
36	1001	10	21.0	9544.3	672.1	7.0
36	1001	11	20.0	9615.9	737.9	7.6
36	1001	12	22.0	10032.3	1455.6	14.5
36	1002	1	19.0	8703.3	636.6	7.3
36	1002	2	19.0	9137.9	434.2	4.7
36	1002	3	21.0	9380.9	440.6	4.6
36	1002	4	21.0	9873.9	543.8	5.5
36	1002	5	20.0	10469.3	590.1	5.6
36	1002	6	21.0	10864.7	462.5	4.2
36	1002	7	21.0	10934.7	514.5	4.7
36	1002	8	20.0	11360.6	637.8	5.6
36	1002	9	21.0	11295.3	674.3	5.9
36	1002	10	21.0	9552.0	1006.8	10.5
36	1002	11	20.0	9405.7	869.4	9.2
36	1002	12	22.0	9535.0	1110.6	11.6
36	1004	1	19.0	3147.2	182.4	5.7
36	1004	2	19.0	3224.6	296.1	9.1
36	1004	3	21.0	3411.6	389.1	11.4
36	1004	4	21.0	3674.0	343.9	9.3
36	1004	5	20.0	3923.7	348.1	8.8
36	1004	6	21.0	4396.9	272.0	6.1
36	1004	7	22.0	4436.3	511.0	11.5
36	1004	8	20.0	4202.3	396.4	9.4
36	1004	9	21.0	3921.0	497.0	12.6
36	1004	10	21.0	3493.3	416.0	11.9
36	1004	11	20.0	3262.5	444.5	13.6
36	1004	12	22.0	3564.0	383.8	10.7
36	1007	1	19.0	5228.4	433.3	8.2
36	1007	2	19.0	5689.0	574.2	10.0
36	1007	3	21.0	6465.5	1180.5	18.2
36	1007	4	21.0	6417.0	613.5	9.5
36	1007	5	20.0	5944.6	1410.3	23.7
36	1007	6	21.0	1533.0	171.1	11.1
36	1007	7	22.0	1344.9	108.9	8.1
36	1007	8	20.0	1223.4	62.2	5.0
36	1007	9	21.0	1061.6	45.7	4.3
36	1007	10	21.0	1075.6	68.4	6.3
36	1007	11	20.0	1050.2	48.6	4.6
36	1007	12	22.0	1048.9	90.5	8.6
36	1008	1	19.0	2969.4	134.0	4.5
36	1008	2	19.0	3099.5	214.8	6.9
36	1008	3	21.0	3125.9	109.2	3.4
36	1008	4	21.0	3300.0	183.6	5.5
36	1008	5	20.0	3323.5	203.1	6.1
36	1008	6	21.0	3525.6	215.7	6.1
36	1008	7	22.0	3790.4	303.1	7.9
36	1008	8	20.0	3761.4	175.0	4.6

In the past, there have been studies to determine the most appropriate duration for coverage counts. The studies generally utilize continuous recorder data and randomly selected samples of varying duration. The samples are then compared with either the average weekday for the month or average day of the month, whichever is appropriate. A great deal of this work has been unreported.

The results of one such study were published in 1954 by Petroff and Blensly (1). Commenting on Figure 1 which is taken from that report, the authors say:

> The observation of the data presented in Figure 1 which is of utmost practical significance is that traffic counts of 24-hour duration on weekdays have a coefficient of variation of 10 percent or less when compared with the mean volume for a weekday in a given month at stations having the mean volume of about 500 vehicles per day or more. This applies usually to all months except the winter months in some states....Counts of 48-hours duration improve the accuracy by 20 to 25 percent, thus raising the confidence limit from 68 percent to about 75 percent for one standard deviation of 10 percent, also extending the range of volumes down to about 300 vpd.

This translated into everyday language means that two-thirds to three-fourths, depending on the length of the count, of all coverage or blanket

TABLE 2

MONTHLY COEFFICIENTS OF VARIATION OF RURAL TRAFFIC VOLUMES

State	Duration	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Avg.	Avg. (Mar. -Nov.)
(a) Summary															
Michigan	24 hr	13.2	11.8	15.5	13.3	17.9	14.0	14.1	11.5	22.8	15.6	21.6	16.1	15.6	16.2
	24 hr[a]	10.6	7.0	13.2	7.7	9.1	10.6	9.9	8.7	21.2	9.5	19.8	14.8	11.8	12.2
	48 hr	8.8	7.7	12.5	10.0	12.3	11.1	10.8	8.8	19.4	12.0	17.8	12.7	11.9	12.7
	5 day	4.4	2.9	7.0	5.7	5.4	5.9	5.2	4.8	14.0	6.1	12.1	9.2	6.9	7.4
	7 day	5.5	3.2	6.1	6.6	6.1	6.2	4.4	5.5	12.3	6.2	12.7	9.2	7.0	7.3
Georgia	24 hr	10.0	10.1	12.2	11.8	11.6	9.9	11.5	8.9	13.3	10.8	11.6	13.8	11.3	11.3
	24 hr[a]	8.4	6.9	8.6	8.1	6.7	7.4	7.1	6.1	11.2	6.5	10.4	14.1	8.4	8.0
	48 hr	7.1	7.6	9.7	8.8	8.1	7.8	8.9	6.7	10.5	7.4	8.0	11.8	8.5	8.4
	5 day	4.6	3.6	5.1	4.3	4.0	4.3	4.9	3.8	6.3	3.4	3.7	9.1	4.8	4.4
	7 day	4.7	3.6	5.8	4.7	4.2	4.5	4.5	4.2	6.3	4.0	3.8	9.2	5.0	4.7
Oregon	24 hr	13.2	10.7	13.7	10.6	14.8	11.4	11.3	8.9	16.5	15.8	15.3	—	12.9	13.1
	24 hr[a]	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	48 hr	10.6	7.7	11.6	7.6	10.1	9.2	8.6	6.7	13.0	11.0	11.4	—	10.2	10.1
	5 day	7.1	4.5	8.4	3.9	5.4	5.8	4.6	3.6	9.5	5.7	6.8	—	5.9	6.0
	7 day	8.2	6.0	8.3	4.7	6.7	6.1	4.6	4.2	8.7	5.3	6.6	—	6.3	6.1
Arkansas	24 hr	12.2	10.4	12.7	9.5	9.9	8.3	10.2	9.3	12.3	10.7	13.1	14.9	11.1	10.7
	24 hr[a]	11.8	8.8	10.9	8.0	8.2	7.2	7.9	7.6	11.2	8.6	13.0	14.2	9.7	9.2
	48 hr	8.8	7.7	9.8	7.1	7.2	6.4	7.8	6.3	9.2	8.2	10.2	12.0	8.3	8.0
	5 day	6.0	4.6	5.9	3.8	4.1	3.8	4.6	3.6	5.5	4.8	9.9	7.5	5.2	5.1
	7 day	6.9	3.9	5.7	3.9	4.0	3.6	4.2	3.6	6.1	4.9	6.3	8.2	5.0	4.7
Florida	24 hr	11.5	10.3	10.3	10.9	9.9	8.3	10.5	7.8	15.4	10.3	11.5	15.6	11.1	10.5
	24 hr[a]	10.3	9.1	7.8	8.5	6.0	6.4	7.5	5.7	14.4	7.2	10.7	14.6	9.0	8.2
	48 hr	8.9	8.5	8.6	8.7	7.1	6.5	8.6	5.7	12.6	7.7	8.7	13.9	8.8	8.2
	5 day	5.6	5.1	5.0	4.6	3.7	3.4	5.5	2.8	8.6	4.1	7.1	10.0	5.4	5.0
	7 day	5.7	5.2	5.3	4.0	3.9	3.3	5.0	3.0	8.3	4.5	3.8	10.6	5.2	4.6
(b) Average Mean Monthly Coefficients of Variation															
All	24 hr	12.0	10.7	12.9	11.2	12.8	10.4	11.5	9.3	16.1	12.6	14.6	15.1	12.4	12.4
	24 hr[a]	10.3	8.0	10.1	8.1	7.5	7.9	8.1	7.0	14.5	8.0	13.5	14.4	9.8	9.4
	48 hr	8.8	7.8	10.4	8.4	9.0	8.2	8.9	6.8	13.1	9.4	11.2	12.6	9.5	9.5
	5 day	5.5	4.1	6.3	4.5	4.5	4.6	5.0	3.7	8.8	4.8	7.9	9.0	5.7	5.6
	7 day	6.2	4.4	6.2	4.8	5.0	4.7	4.5	4.1	8.3	5.0	6.6	9.3	5.9	5.5

[a] 24-hr weekday counts taken Monday through Thursday.

> counts may be expected to have an error of about 10 percent or less when compared with the true mean weekday volume of the month during which they were taken when volumes are 300 to 500 vehicles per day or more.

These observations are for coverage counts in rural areas. Results of a similar study for urban areas were reported by Petroff and Kancler in 1958 (2). This paper is concerned with rural coverage counts only.

The present study examines on a population rather than a sample basis the relative accuracy of coverage counts of 24 and 48 hours on weekdays, 5 weekdays and 7 days. Counts of 72 hours were not tested because of their infrequent use. The study's objective was to determine if the observations of Petroff and Blensly are still applicable and to obtain information on the increase in accuracy of ADT estimates that may be expected by increasing the duration of coverage counts. In some states, Friday volumes are more like those on weekends than on weekdays. For this reason, 24-hour counts taken Monday through Thursday were also tested.

Rather than testing randomly selected sample counts of the desired duration from continuous recorder data, all combinations of data for the five selected count durations were analyzed. This was possible through use of an IBM 7010 computer. In order to draw definite conclusions for all states, it would be necessary to test data from all states. This would be impractical and prohibitively expensive. It was decided to utilize data from a limited number of states geographically distributed around the country. Hourly volume data were obtained for 386 continuous recorders in five states for 1964 as follows: (a) Arkansas, 76 recorders; (b) Florida, 80 recorders; (c) Georgia, 25 recorders; (d) Michigan, 116 recorders, and (e) Oregon, 89 recorders. The states were selected primarily on the basis of ready availability of data and convenient format.

Each state's data were subjected to the same testing procedure. All available volumes at each station for 24 hours and 48 hours on weekdays and five consecutive weekdays were compared with the appropriate average weekday of the month. The standard deviation and coefficient of variation were computed for each month at a station for the five selected count durations. Table 1 is a sample of the computer output.

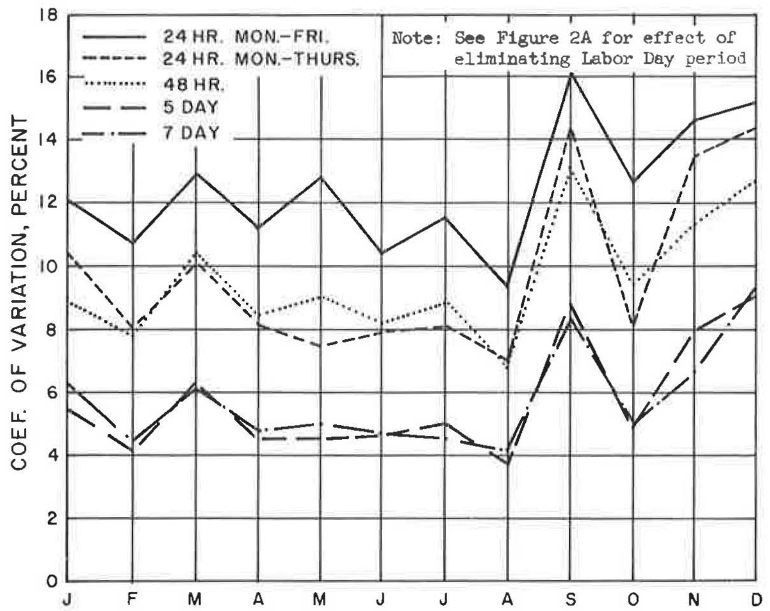

Figure 2. Monthly fluctuation of coefficients of variation for rural traffic volumes.

In computing the standard deviation for the 24-hour counts, the following formula was used:

$$\sigma = \left[\frac{\sum\limits_{1}^{N} X_i^2}{N} - \overline{X}^2 \right]^{1/2}$$

The following formula was used to compute the root mean error about the average weekday of the month for the three additional coverage count durations:

$$\text{RME} = \left[\frac{\sum\limits_{1}^{N} X_i^2}{N} - 2\,\overline{X}\,\overline{Y} + \overline{Y}^2 \right]^{1/2}$$

The formula of the root mean square error yields a slight overestimate of the standard deviation. It was adopted in order to compare the 48-hour, 5-day and 7-day counts with the same average weekday of the month as the 24-hour counts. Use of the first formula with the multi-day counts would have resulted in comparing each multi-day count with the average of the multi-day counts. Derivations of the two formulas are in the Appendix.

All computations, including summarizing hourly volumes into daily volumes, were done on an IBM 7010 computer. Fortran IV programs were written for each phase of the study.

TABLE 3

ANALYSIS OF RURAL COVERAGE-COUNT DURATION COEFFICIENTS OF VARIATION
(Summary Table)

State	24 Hr	24 Hr[a]	48 Hr	5 Day	7 Day	No. of Stations
Georgia	11.3	8.4	8.5	4.8	5.0	25
Florida	11.1	9.0	8.8	5.4	5.2	80
Oregon[b]	12.9	–	10.2	5.9	6.3	89
Michigan	15.6	11.8	11.9	6.9	7.0	116
Arkansas	11.1	9.7	8.3	5.2	5.0	76
Total						386
Average	12.4	9.7	9.5	5.6	5.7	

[a] Weekday counts taken Monday through Thursday.
[b] Due to a bad flood in December 1964, counts for this month are not included in the averages.

RESULTS

Table 2 gives a tabulation of the monthly and overall annual coefficients of variation of the coverage counts for each state and for the five coverage-count periods. These are monthly average coefficients of all stations in the state. In these computations,

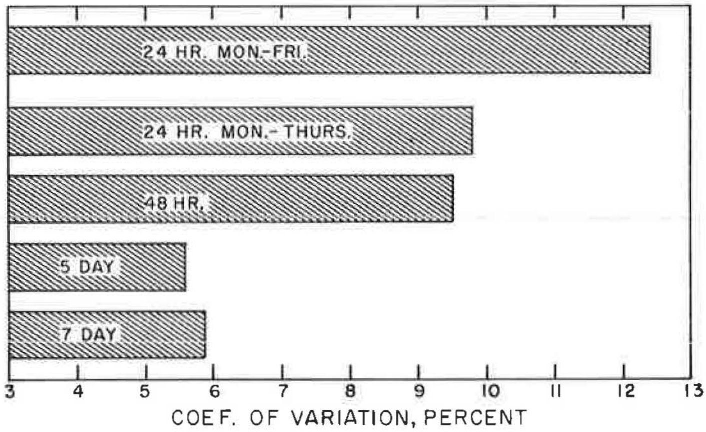

Figure 3. Annual mean coefficients of variation for rural traffic volumes.

the coverage counts for periods exceeding a 24-hour duration were reduced to 24-hour averages. Table 2 also gives averages of the monthly coefficients for the five states. These same coefficients are shown in graphical form in Figure 2. Please note that the December results in Oregon are not included in any summaries due to a flood which disrupted traffic in some parts of the state. Also in Oregon, the analysis of 24-hour coverage counts taken Monday through Thursday is omitted, since individual weekdays were not easily identified on the input tape. Table 3 is an overall summary of the coefficients in the last column of Table 2. Annual mean coefficients of variation for the five states are also shown in Figure 3.

One striking observation is that the coefficient of variation for 24-hour counts taken Monday through Friday is nearly always greater than ±10 percent. This is true for both the overall monthly averages and annual average (Table 3). Arkansas and Florida were the only two states where the monthly coefficients were under ±10 percent in more than one or two months. Consideration of only the months March through November, when some states do all coverage counting, does not alter the picture described. These are rather significant observations, since quite a number of states now use 24-hour coverage counts for ADT estimating purposes. Perhaps an evaluation of the 24-hour counts in these states would be in order to determine if this study's results also apply to the particular states concerned.

By taking 48-hour weekday counts, it appears that the coefficient of variation can be reduced below the ±10 percent level to a point where ADT estimates will more closely approach the desired accuracy. Exclusive of Michigan, where the volumes are more variable than in the other four states, the overall annual coefficient of variation for 48-hour weekday counts is ±9.0 percent. This is approximately the same relative improvement observed by Petroff and Blensly. Scheduling of 48-hour counts does not generally present any greater problems than those for 24-hour counts. There is the possibility that field men may have all their recorders picked up by noon Friday. Many states utilize Friday afternoons for equipment maintenance, special purpose counts, or travel.

TABLE 4

EFFECT ON VARIATION OF 24-HOUR WEEKDAY VOLUMES OF ELIMINATING LABOR DAY HOLIDAY PERIOD, 1964

State	Mean September Coefficients		Difference
	Original	After Eliminating Sept. 4 and 7	
Michigan	22.8	13.7	9.1
Georgia	13.3	10.7	2.6
Oregon	16.5	11.4	5.1
Arkansas	12.3	9.8	2.5
Florida	15.4	11.2	4.2
Average	16.1	11.4	4.7

In recent years, many states have observed that Friday traffic, especially Friday afternoon, more closely resembles weekend rather than weekday traffic. Total daily volume observations are substantiated in this study by the reduction in the coefficient of variation resulting from considering only Monday through Thursday for 24-hour

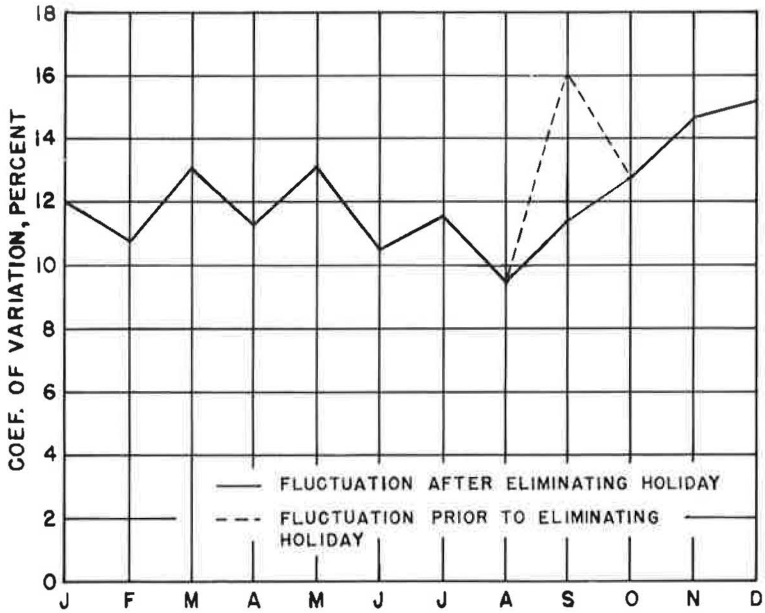

Figure 4. Effect on 24-hr weekday counts of eliminating Labor Day period.

counts. Table 2 and Figure 2 show a very close similarity between 48-hour counts and 24-hour counts which exclude Fridays.

The decrease in annual mean coefficients of variation between the 48-hour counts and the 5-day count was 3.9 percent as compared to 2.9 percent between 24-hour and 48-hour counts (Table 3). The results for the 7-day counts are almost the same as for 5-day counts. Either count duration cuts the 24-hour relative variation in half. The ADT estimates resulting from the expansion of 5- or 7-day counts should have a total relative error of about ±6 to 8 percent. This figure is obtained by using Eq. 2, which combines the effect of variation of daily volumes with the effect of using a group mean

TABLE 5

MEAN MONTHLY COEFFICIENTS OF VARIATION FOR RURAL TRAFFIC VOLUMES

Duration	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Avg.
24 hr	12.9	11.2	12.8	10.4	11.5	9.3	11.4	12.6	14.6	11.9
24 hr[a]	10.1	8.1	7.5	7.9	8.1	7.0	–	8.0	13.5	8.8
48 hr	10.4	8.4	9.0	8.2	8.9	6.8	–	9.4	11.2	9.0
5 day	6.3	4.5	4.5	4.6	5.0	3.7	–	4.8	7.9	5.2
7 day	6.2	4.8	5.0	4.7	4.5	4.1	–	5.0	6.6	5.1

[a]Twenty-four hour weekday counts taken Monday through Thursday.

Note: September coefficients excluding Labor Day period calculated only for 24-hour counts Monday through Friday.

TABLE 6

SUMMARY OF MEAN ANNUAL COEFFICIENTS OF VARIATION
(Effect of Eliminating Station With ADT Less Than 500)

States	24 Hr	24 Hr[a]	48 Hr	5 Day	7 Day	No. of Stations
Arkansas						
All stations	11.1	9.7	8.3	5.2	5.0	76
High-volume stations	9.2	7.5	6.8	4.1	4.1	55
Difference	1.9	2.2	1.5	1.1	0.9	11
Oregon						
All stations	12.9	–	10.2	5.9	6.3	89
High-volume stations	12.5	–	9.8	5.6	6.0	74
Difference	0.4	–	0.4	0.3	0.3	15

[a]Weekday counts taken Monday through Thursday.

TABLE 7

AVERAGE COEFFICIENTS OF VARIATION
(Low-Volume Rural Station)

Station	Coverage Count	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Sept. Adjusted[b]	Oct.	Nov.	Dec.	Avg.
Arkansas	24 hr	17.5	17.0	19.4	14.4	13.0	12.1	14.2	12.1	16.7	16.4	16.0	18.0	20.5	15.9
	24 hr	17.8	16.9	19.6	14.2	12.6	11.9	12.6	12.5	16.5	—	15.2	17.6	21.0	15.7
(20 stations)	48 hr	13.3	13.2	14.7	10.7	9.7	9.9	10.7	8.8	13.1	—	12.7	15.0	17.1	12.4
	5 day	9.2	8.3	9.3	6.4	5.4	6.3	6.4	5.4	8.1	—	8.6	—[a]	10.8	7.6
	7 day	9.3	7.4	8.1	6.5	5.1	6.0	5.5	5.0	7.8	—	8.3	10.0	10.9	7.5
Oregon	24 hr	18.3	11.6	16.3	13.7	15.4	15.2	14.4	11.3	18.0	12.2	19.4	17.0	—[a]	15.3
	24 hr	—	—	—	—	—	—	—	—	—	—	—	—	—	—
(15 stations)	48 hr	11.5	10.0	13.9	10.3	11.2	12.2	11.6	8.6	14.7	—	15.1	12.8	—[a]	12.1
	5 day	8.0	7.0	10.2	6.7	6.8	8.2	7.0	5.6	10.2	—	9.2	8.3	—[a]	7.8
	7 day	10.8	7.7	10.0	6.5	7.6	8.3	6.2	6.0	10.6	—	9.2	8.5	—[a]	8.1
No. 4,039	24 hr	18.1	10.2	15.5	17.4	19.9	14.3	12.2	10.2	36.1	19.8	6.5	23.5	17.7	17.0
Michigan	24 hr	15.9	9.5	17.0	16.2	17.8	14.9	12.9	10.7	39.1	—	7.6	25.6	17.0	17.0
	48 hr	16.1	6.3	10.7	15.8	16.4	13.2	10.6	8.8	32.2	—	6.8	20.5	13.6	14.2
	5 day	14.1	4.1	5.0	13.6	13.3	10.0	8.0	7.5	27.2	—	3.6	16.3	8.1	10.9
	7 day	16.3	4.1	5.9	22.0	15.4	8.9	6.3	9.7	28.9	—	5.7	18.4	10.7	12.7
No. 4,099	24 hr	33.7	13.0	26.8	22.9	34.6	14.3	18.8	17.8	78.5	15.5	25.9	53.9	24.5	30.4
Michigan	24 hr	31.0	9.7	29.6	19.3	10.1	10.2	10.9	14.8	86.7	—	17.8	59.0	23.1	26.8
	48 hr	28.9	8.9	20.6	18.3	20.9	11.2	13.6	15.3	63.6	—	21.9	50.3	19.3	24.4
	5 day	18.9	6.0	15.1	12.2	10.7	6.9	5.3	12.8	44.9	—	13.2	38.5	14.1	16.6
	7 day	38.4	9.1	11.9	12.5	16.2	6.4	11.2	14.0	41.5	—	18.1	42.4	15.9	19.8
No. 1,033	24 hr	9.8	8.8	11.9	11.8	9.1	25.8	12.0	12.1	11.2	10.9	10.6	11.2	14.6	12.4
Georgia	24 hr	10.7	8.5	8.1	12.6	9.0	23.3	8.4	12.6	9.2	—	8.4	10.1	15.4	11.4
ADT = 442	48 hr	6.0	6.6	8.7	9.3	7.3	24.0	8.2	5.8	7.6	—	6.1	8.4	12.3	9.2
	5 day	4.3	4.1	4.8	5.6	4.2	21.8	4.7	2.8	3.2	—	4.2	4.0	8.0	6.0
	7 day	5.5	3.1	4.6	4.5	3.5	18.9	4.5	4.3	4.6	—	5.2	4.3	6.1	5.7

[a]Coefficients not calculated.
[b]These coefficients show effect of eliminating Labor Day period and are not included in last column.

factor. These longer coverage-count durations are popular in northern climates and where there is extensive mileage of low-volume highways. From a scheduling standpoint, the 7-day counts may be more practical.

Many highway departments exclude the winter months from their coverage counting schedules. In order to determine the effect of winter months, the overall mean coefficients of variation were calculated for March through November. These figures are shown in the last column of Table 2. Although there is little change in the overall mean coefficients, Figure 2 shows the definite advantage of eliminating winter months, since they generally have higher coefficients than the remaining months.

The unusually high coefficients of variation for September are largely a result of steadily decreasing volumes throughout the month at most stations. Failure to eliminate from consideration holiday periods during the month further contributed to the variation of the weekday volumes. To determine the degree that September coefficients of variation were affected by the Labor Day traffic, two weekdays (September 4 and 7, 1964) were eliminated and coefficients recomputed for 24-hour weekday counts. The average decrease in the coefficients of variation was -4.7 percent. Similar decreases in variation for the other four count durations would be expected. Table 4 gives the results of the comparison in detail. Figure 4 shows a comparison monthly fluctuation of 24-hour weekday counts taken Monday through Friday. The September variation is still high. This must be attributed to a significant decrease in volumes during the month. Table 5 gives overall mean coefficients of variation for the period March through November, excluding September for all count durations other than 24-hour Monday through Friday. This shows that all coverage counts taken during this period, except 24 hours Monday through Friday, have coefficients of variation sufficiently low to produce satisfactory estimates of ADT.

The "Guide for Traffic Volume Counting Manual" suggests treating all roads with ADT less than 500 in a separate category. Of the 386 permanent recorders studied, fewer than 40 had ADT's less than 500. These were concentrated in Arkansas (21) and Oregon (15). Table 6 shows that very slight effect on the annual mean coefficients of eliminating these lower volume stations. The remaining three states had no more than three low-volume stations each. Therefore, the observations discussed can be considered representative of higher volume locations.

In 1946, Petroff (3) reported on the fluctuation of weekday volumes at locations where the ADT is less than 500 vpd. The study utilized data from 10 permanent recorders in northern states and 10 recorders in southern states. The mean annual coefficients of variation in the northern states for 24- and 48-hour counts were 24.96 percent and 19.50 percent, respectively. In the southern states, the coefficients were 19.32 percent and 14.85 percent.

Table 7 gives the mean monthly coefficients of variation for Arkansas and Oregon plus coefficients for three additional stations. Although the coefficients are not as high as those reported by Petroff, the difference of about 3 percent between the two count periods is comparable to Petroff's results. The coefficients in Table 7 produce the additional observation that ADT estimates with a standard deviation of ±10 percent or less are unlikely for low-volume stations, at least in these states, unless coverage counts are of 5 or 7 days duration.

CONCLUSIONS

The most important result of this study relates to the use of 24- or 48-hour coverage counts for ADT estimating purposes on rural roads with ADT's greater than 500 vpd. Coverage counts taken for 48 hours on weekdays will have a mean annual coefficient of variation of ±9.0 to ±9.5 percent when compared to the average weekday of the month. The study results strongly indicate that a coefficient of ±10 percent or less is not to be expected for coverage counts of 24 hours taken Monday through Friday. If these results are fairly representative of conditions in other states, 24-hour counts taken Monday through Friday should generally not be used to obtain ADT estimates with a relative error of ±10 percent. Although this study included only five states, the results appear definite enough to warn against use of 24-hour weekday coverage counts without sufficient proof that results will be satisfactory in the particular state.

The results also indicate that, if Fridays can be excluded from coverage counting, the coefficients of variation for 24-hour weekday counts will be comparable to 48-hour counts taken Monday through Friday. It should be noted that the full 24-hour Friday volumes were excluded in this part of the analysis. Although many states exclude Friday p.m. volumes, this practice was not tested. It would seem prudent, however, for individual states using Friday morning volumes to verify that they were not significantly different from those of other weekdays.

For estimating ADT on rural roads with ADT's under 500 vpd, it appears that neither 24- nor 48-hour counts should be used if the desired relative error of estimate of ADT is ±10 percent. In this case, either 5- or 7-day coverage counts are recommended. Many highway departments do not feel that it is necessary to maintain the same accuracy of ADT estimates for roads with ADT under 500 vpd as for the higher volume roads. Coverage counts of 48-hour duration on low-volume roads, having coefficients of variation of about ±12 percent when compared with the average weekday of the month, will probably produce ADT estimates with a standard deviation of about ±12 to 14 percent. Similarly, 24-hour coverage counts with coefficients of variation of approximately ±16 percent should produce ADT estimates with a standard relative error of about ±16 to 17 percent.

In scheduling coverage counts, most states, except those in the southern part of the country, exclude the winter months. Tables 2 and 4 do not indicate a marked decrease in annual coefficients of variation for the five states studied when winter months are excluded. Experience has shown that permanent recorders in most states are difficult to group following the U. S. Bureau of Public Roads "Guide for Traffic Volume Counting Manual" when winter months are included. The practice of excluding winter months from coverage-counting schedules, therefore, appears desirable in all but a few southern states.

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Appendix

DERIVATION OF FORMULA FOR USE WITH 24-HOUR COUNTS

N = number of 24-hour samples in a month,
X_i = ith 24-hour sample in a month,
$\bar{X}$ = average weekday of the month, and
σ = standard deviation.

$$\sigma^2 = \frac{\sum_{i=1}^{N} (X_i - \bar{X})^2}{N} = \frac{1}{N} \sum_{i=1}^{N} (X_i^2 - 2\,\bar{X}\,X_i - \bar{X}^2)$$

$$\sigma^2 = \frac{\sum_{i=1}^{N} X_i^2}{N} - 2\,\bar{X}\,\frac{\sum_{i=1}^{N} X_i}{N} + \frac{\sum_{i=1}^{N} \bar{X}^2}{N}$$

$$\sigma^2 = \frac{\sum_{i=1}^{N} X_i^2}{N} - 2\,\bar{X}\,\frac{\sum_{i=1}^{N} X_i}{N} + \frac{N\,\bar{X}^2}{N}$$

$$\sigma^2 = \frac{\sum_{i=1}^{N} X_i^2}{N} - 2\,\bar{X}^2 + \bar{X}^2$$

$$\sigma = \left[\frac{\sum_{i=1}^{N} X_i^2}{N} - \bar{X}^2 \right]^{1/2}$$

DERIVATION OF FORMULA FOR 48-HOUR, 5-DAY AND 7-DAY COUNTS

$X_i = \dfrac{\text{Vol } 1 + \text{Vol } 2 + \text{Vol } 3 + \ldots \text{Vol } n}{n}$ $\quad n = 2,\ 5$ or 7,
N = total number of possible samples in the month,
X = average of samples = $\dfrac{\sum_{1}^{N} X_i}{N}$, and
Y = average weekday or average day of the month (X is not necessarily equal to Y, but will be a close approximation to Y).

$$(\text{RM error})^2 = \frac{\sum_{1}^{N} (X_i - Y)^2}{N} = \frac{1}{N} \sum_{1}^{N} (X_i^2 - 2X_i\,Y + Y^2)$$

$$(\text{RM error})^2 = \frac{\sum_{1}^{N} X_i^2}{N} - 2\,Y\,\frac{\sum_{1}^{N} X_i}{N} + \frac{\sum_{1}^{N} Y^2}{N}$$

$$(\text{RM error})^2 = \frac{\sum_{1}^{N} X_i^2}{N} - 2\,Y\,X + Y^2$$

$$\text{Root mean square error} = \left[\frac{\sum_{1}^{N} X_i^2}{N} - 2\,X\,Y + Y^2 \right]^{1/2}$$

30th Peak Hour Factor Trend

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ABRIDGMENT

•USING counting station data from Pennsylvania, the 30th peak hour factor trend is compared to the urban-rural classification, the AADT group, the AADT/lane group, the trend of the AADT and New Jersey's 2.3 percent compounded reduction rate. Plotting the factor against time, a 1.4 percent compounded reduction rate fits the Pennsylvania data.

When this overall reduction rate is compared to the mentioned variables, it is found that the percent of stations with a decreasing factor: (a) is greater for rural roads than urban roads; (b) decreases as the AADT group increases; (c) decreases as the AADT/lane group increases; (d) is similar for roads with a changing AADT but is much lower for roads with no change in the AADT; and (e) increases as the factor group increases.

Paper sponsored by Committee on Highway Capacity.

Truck Equivalency

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ABRIDGMENT

•THE purpose of this study is to investigate the relationship of headways between passenger car following passenger car in predominantly passenger car lanes (less than 0.5 percent trucks) and headways between (a) passenger car following passenger car, (b) passenger car following truck, (c) truck following passenger car, and (d) truck following truck in an 80 percent truck lane and a 38 percent truck lane.

The average headways of vehicle queues of 11 vehicles is investigated to attempt to determine a relationship between the number of trucks in the queue and the average headway. The data on average headways were obtained by use of a 20-pen recorder, which recorded the headways between the various type vehicles under the mentioned conditions.

The following conclusions are made from an analysis of field data at the study location:

1. It appears that the percent of trucks in a traffic lane has an effect on the average headway of all type vehicles within the lane. The average headways appear to increase with an increase in percent trucks. However, a truck does not appear to be equivalent to as many as two passenger cars from a volume or headway standpoint.
2. It appears that the number of trucks in a queue of 11 vehicles has an effect on the average headway in the queue. The greater the number of trucks the larger the average headway. Again, however, a truck does not appear to be equivalent to as many as two passenger cars.

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