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Traffic Accident Studies--1958

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Contents

Traffic Accidents and the Quality of Traffic Flow

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This is a report of an attempt to find if there is a correlation between the quality of traffic flow and the frequency of highway accidents. The term "quality" characterizes the traffic stream and indicates the manner in which vehicles move.

In making the study three sections of highway with different accident frequencies per million vehicle-miles were selected for investigation. Two of the sections are two-lane and the third is three-lane. The accident frequencies and the corresponding quality indices were as follows:

Road	Quality Index	Accidents Per Million Vehicle Miles
A (two lane)	618	2.98
B (two lane)	1023	1.03
C (three lane)	1930	1.66

The three-lane road with the highest quality index did not have the lowest accident rate. This indicates that there is no direct correlation between the index and accident frequency. On the other hand, it could reveal that a three-lane road should not be compared with a two-lane. Furthermore, it could be that single vehicle accidents are a better basis for a comparison of the two types of roads. Using single vehicle accidents only, the following comparisons were obtained:

Road	Quality Index	Single Vehicle Accidents Per Million Vehicle-Miles
A (two lane)	618	1.28
B (two lane)	1023	0.36
C (three lane)	1930	0.25

The table shows that for single vehicle accidents the higher the quality the lower the accident frequency.

The fact that there were more accidents on the curving road than on the straighter one, points to the need for including change of vehicle direction as well as change of speed in the quality index.

The results of this limited study show that improving the quality of traffic flow should reduce accidents. Apparently the inherent characteristics of flow in a traffic stream tend to make it safe or hazardous.

● A RELATIONSHIP between the characteristics of highway traffic flow and the frequencies of traffic accidents has long been recognized, but up to the present time there has been no attempt to measure this relationship with any degree of precision. This is a report of a preliminary attempt to make such a measurement. The investigation is a part of the traffic accident research being conducted by the Transportation Institute of the University of Michigan.

This research had its inception in the recognition of the magnitude and the complexity of the motor vehicle accident problem, the solution of which was beyond the realm of existing programs of safety education, driver training and highway construction. The key to the solution of the problem, an adequate knowledge of the causes of accidents, was missing and could be obtained only through research. Traditionally, research is the responsibility of the University. In keeping with this responsibility, the initiation of the present research program was made possible by a state legislature grant to the University in the summer of 1956.

The research covered in this report is limited to: (1) the selection of three sections of highway with different accident frequencies; (2) the collection of data for the determination of the quality of traffic flow; and (3) a comparison of the quality index with accident frequency.

For the sections studied there is a definite correlation between the quality index and accident frequency. A greater range of indices must be examined before it can be determined whether the same relationship holds throughout the possible range. It is suspected that in urban areas where the quality of flow index can be quite low that the relationship between the index and the frequency of accidents may be different than for rural areas. In slow, congested traffic where the quality index is quite low it is logical to suspect that the severity if not the number of accidents decreases. There is, however, no reason to suspect that there is no relationship.

Once the relationship between the quality of flow and the frequency of accidents has been established it should become possible to predict the accident proneness of a highway. It is better to make improvements to reduce accidents before the accidents happen than after they occur.

QUALITY INDEX

An index measurement of the quality of traffic was developed at the Yale University Bureau of Highway Traffic in 1954.[1] This index was deemed the best one for initial trial in the present study. It is dimensionless, and simple in application. It yields accurate and unbiased measurements, and at the same time reflects the feelings of the driver. Subjectively it is a measure of the satisfaction of the driver in operating his vehicle; objectively, it is a measure of the characteristics of the traffic stream.

The quality of flow index "Q_F" as developed in the former study is equal to the product of a constant times the average speed divided by the change of speed per mile times the square root of the number of changes of speed per mile.

This statement may be expressed much more concisely by the use of symbols. Letting Q_F = quality of flow per mile, S = average speed, Δ_S=change of speed per mile, f = frequency of change of speed per mile and K = a constant of 1000 to prevent the value of Q_F from becoming a small fraction, the equation may be written:

$$Q_F = \frac{KS}{\Delta_S \sqrt{f}}$$

Letting L = distance and T = time and omitting K and f, since they are pure numbers, the dimension equation may be written:

$$\frac{Q_F}{L} = \frac{\frac{L}{T}}{\frac{\sum\left(\frac{L}{T} - \frac{L}{T}\right)}{L}} \quad \text{or} \quad Q_F = \frac{\frac{L}{T}}{\sum\left(\frac{L}{T} - \frac{L}{T}\right)} = \frac{\frac{L}{T}}{\frac{L}{T}}$$

wherein $\sum\left(\frac{L}{T} - \frac{L}{T}\right)$ = summation of speed change; $\sum\left(\frac{L}{T} - \frac{L}{T}\right) = \frac{L}{T}$, dimensionally, and is never zero for there is always some change of speed in any appreciable distance traveled. (See discussion at end of report.)

[1] "Quality of Traffic Transmission." Bruce D. Greenshields, HRB Proc. (Jan. 1955).

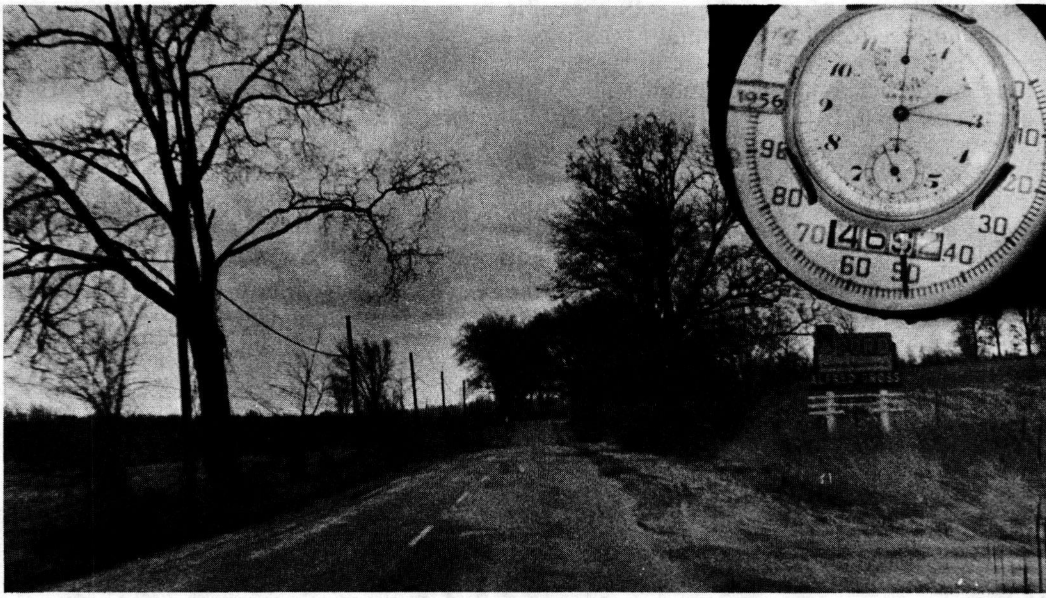

Figure 1. Picture taken with Markel camera showing chronometer and speedometer in upper right-hand corner.

After a study of the results obtained by this formula it was decided to try simplifying it by omitting the term giving the frequency of the speed changes, and decreasing the value of K. This simplification does not change the basic character of the index number. As may be seen by comparing the results given later in the report the simpler expression seems to be as accurate an indication of quality of flow as the other.

The denominator of either of the index numbers is a measure of the annoyance and frustration suffered by the driver as he is forced by highway deficiencies, conflicting traffic movements and congestion to stop, start , and change speed. The greater his speed changes the more annoyed he is.

COLLECTION OF FIELD DATA

While it is possible to obtain field data with present equipment, there is a need for development of new devices to give more complete information. It was indicated, for example, that it is desirable to record change of direction as well as change of speed. Since along with the measurement of quality of flow there is desired a measure of causes that affect flow there is needed a means of recording quickly and with sufficient accuracy the grade, curvature, roughness, and uneveness of highway surface.

At the beginning of the project the only equipment that was readily available and that seemed suitable for recording field data was a traffic camera obtained from Markel Service, Inc. This camera is mounted so as to take pictures through the windshield. In the corner of each picture, photographed by a separate lens, is the date, a watch, and a speedometer giving the speed of the car which contains the camera (Fig. 1). The speed of any chosen vehicle is obtained by trailing it and duplicating its speed as closely as possible.

In order to get an approximately continuous speed record, pictures were taken at two-second intervals. A shorter interval would have given a more nearly continuous record but it would have added to the cost and made the analysis more tedious. These intermittent pictures showed, along with the speed record, the road conditions and furnished a count of the on-coming traffic on the opposite lane. This count was found to be correct to within about 5 percent.

Later on in the work, the Highway Traffic Safety Center at Michigan State University cooperated on the project by lending their especially equipped test car for a series of runs. The part of the equipment applicable to the project was a recording speedometer. This recording speedometer was first designed and constructed for use on the Yale Bureau Project. It gives on a continuous chart the speed of the vehicle in which it is mounted. The speed is automatically plotted against time or distance, as desired. A set of six push buttons are connected to code pens, and may be used to record various items such as on-coming traffic or location of crossroads. A picture of the recorder is shown in Figure 2. A section of a recording is shown in Figure 3.

Starting at the bottom of the chart, pen A (line A) shows the time in six-second intervals; B shows one minute intervals; C gives the distance in 400-ft intervals; the irregular line in the center of the chart is continuous speed record; pens D, E, and F at the top are code pens that may be used as desired. Pen D for example shows points of curvature and the beginning and end of each run.

The data to be transferred from either the pictures or the speed record consists of the speed change per mile, and the number of speed changes per mile.

Starting at the left in Figure 3, the speed is 50 mph, then rises to 57 mph, changes and falls to 54 mph, and rises again to 70 mph, etc. The speed change per mile is the sum of the changes without regard to sign. The frequency is the number of speed changes per mile, a speed change being defined as a change from a rise to a fall in speed or the reverse. The average speed is obtained from the distance and time graphs.

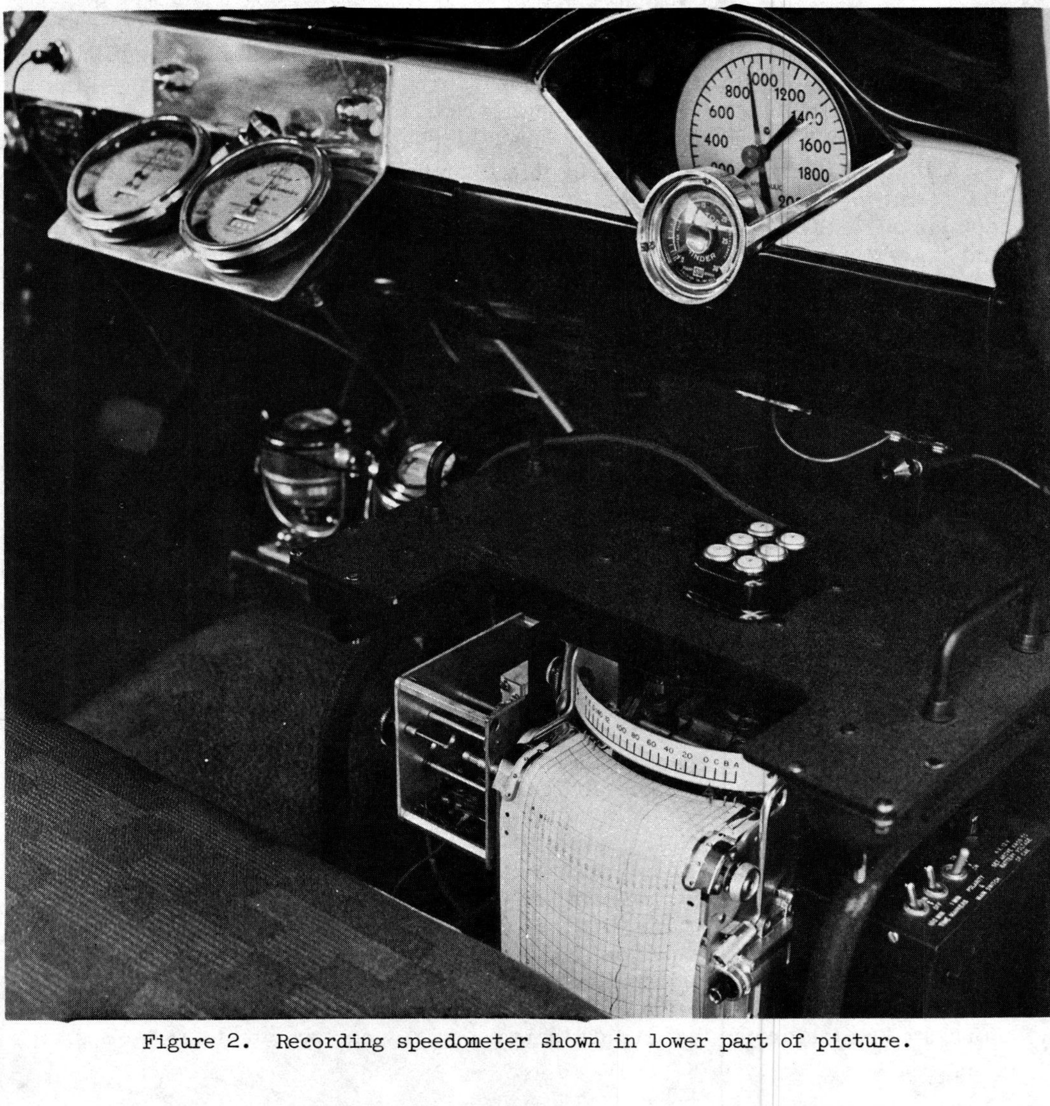

Figure 2. Recording speedometer shown in lower part of picture.

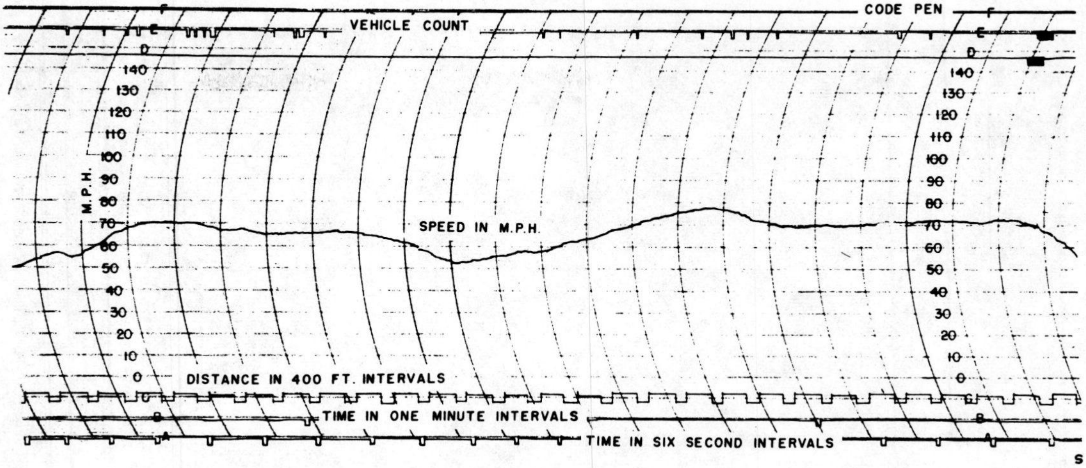

Figure 3. Traffic performance chart.

In the picture method the distance, time and speed is read from the pictures taken at two-second intervals. This discontinuous record gave a lower speed change than the continuous speed record. In order to bring the data to the same base all picutre data were prorated to equal the tape records. This ratio was based on the over-all average values and checked by picture and tape records taken simultaneously.

Before determining the correlations between the quality of flow and the frequency of accidents the sections of highway studied will be described and the accident experience for each section will be analyzed.

HIGHWAYS AND ACCIDENT EXPERIENCE

The traffic accident frequencies for the three sections of highway studied will first be discussed along with the traffic flow factors and then compared.

The section of the Dexter-Pinckney Road studied extends from the town of Dexter to to the McGregor Road, a distance of 5.7 mi (Fig. 4). This two-lane road, approxi-

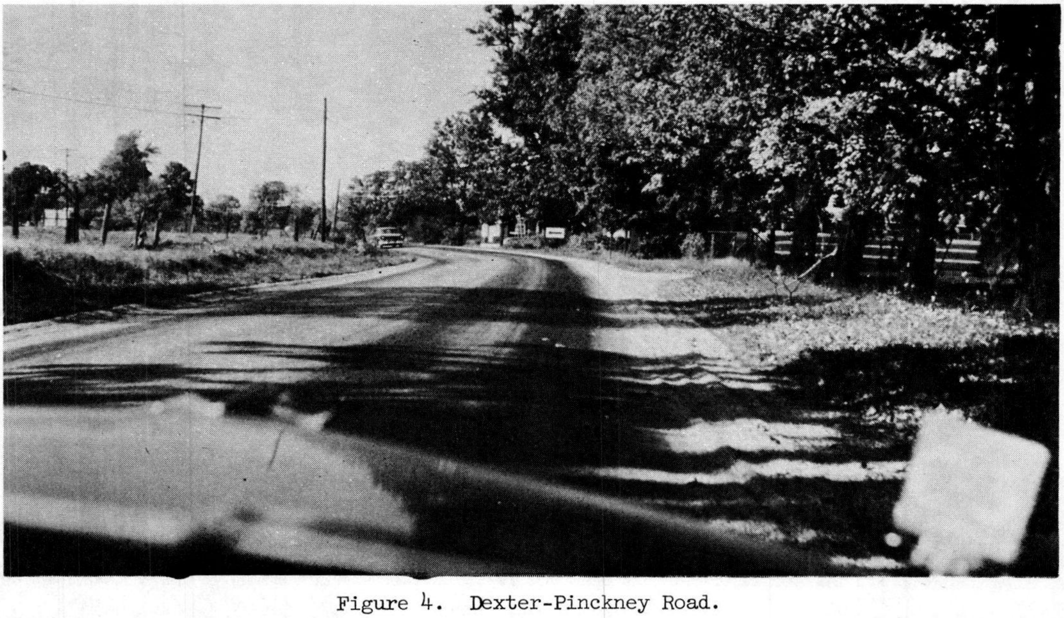

Figure 4. Dexter-Pinckney Road.

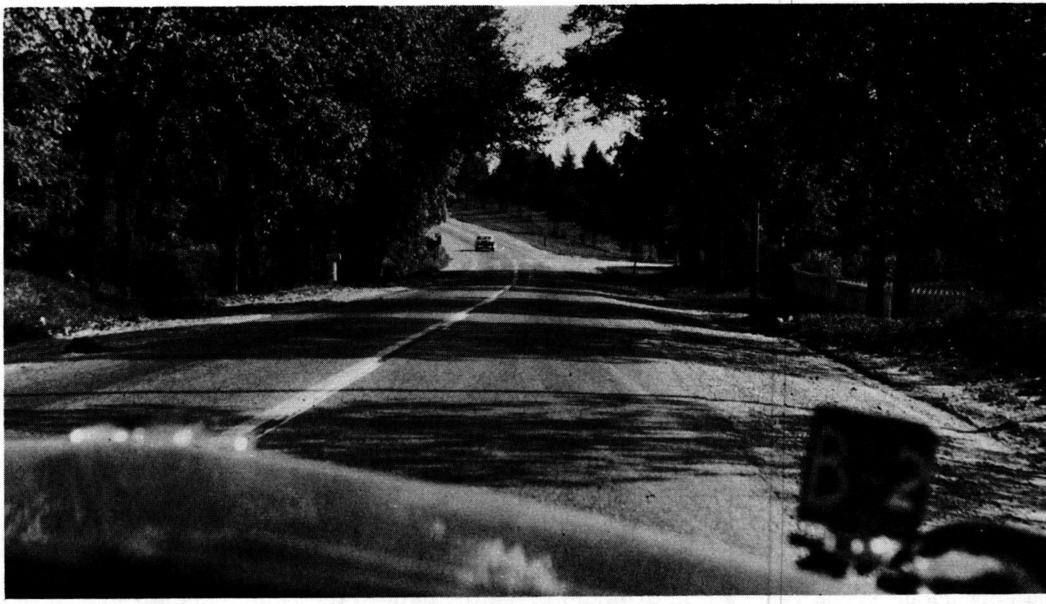

Figure 5. Plymouth Road, US 14.

mately 23 ft wide, has a very uneven blacktop surface. The road is curving and practically all of the section observed is signed for 45 mph safe speed.

The section of the Plymouth Road, US 14, studied extends eastward from Ann Arbor (Fig. 5) for a distance of 6.3 mi. It is a two-lane road approximately 20 ft wide, with blacktop surface in good condition.

The section of the Jackson Road, US 12, studied extends westward from Ann Arbor for a distance of about 6.7 mi (Fig. 6). This is a three-lane road approximately 31 ft wide, with concrete surface.

The type and frequency of highway accidents on the three highway sections is tabulated below. This information was taken from traffic accident records of the Michigan

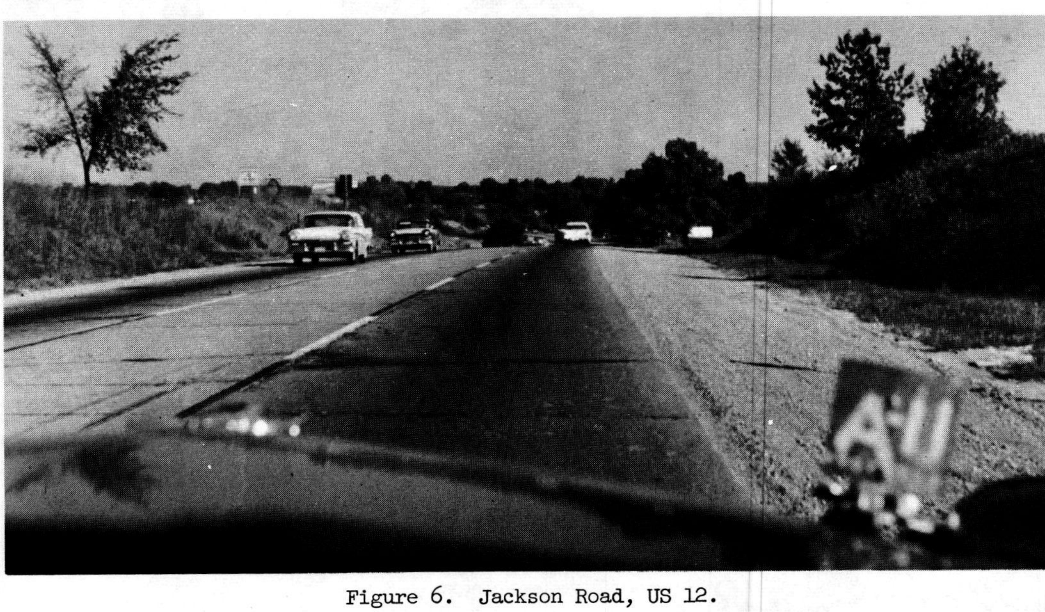

Figure 6. Jackson Road, US 12.

State Police. The total reported accidents, injuries and deaths for the three sections of highways are as follows:

Dexter-Pinckney Road	1953	1954	1955	1956
Traffic Deaths	0	0	1	4
Injuries	5	6	11	10
Accidents	14	15	29	25

Plymouth Road				
Deaths	0	4	2	1
Injuries	11	16	12	10
Accidents	25	27	23	20

Jackson Road				
Deaths	1	1	2	0
Injuries	32	25	35	18
Accidents	54	44	45	34

A detailed analysis of these accidents is shown in the following tabulation, the four-year records being combined.

PERCENTAGE OF ACCIDENTS OF EACH TYPE

	Dexter-Pinckney Road	Plymouth Road	Jackson Road
Type:			
Collision between vehicles	47.5	58.0	78.0
With fixed object	26.0	23.0	10.8
Ran-off-road	17.0	13.2	4.6
Collision with pedestrian	2.5	-	3.1
Overturning	6.5	5.5	3.6
Violations:			
1. Excessive speed	81.0	48.5	40.8
2. Disregard of officer or controlling device	3.8	15.4	1.1
3. Wrong side - not passing	7.7	10.2	1.1
4. Failure to grant right-of-way	-	12.8	12.5
5. Under influence of alcohol	-	-	12.8
6. Improper passing	3.8	5.1	-
7. Improper turns	-	5.1	4.7
8. Follow too closely	3.8	-	22.0
9. Failed to signal - improper signal	-	2.6	1.6
Road Alignment			
1. Straight road - level	49.5	43.0	75.0
2. Straight road - hillcrest	2.1	5.5	0.4
3. Straight road - on grade	8.5	21.9	20.2
4. Curve or turn - level	26.5	14.9	1.3
5. Curve or turn - hillcrest	0.0	1.0	0.4
6. Curve or turn - on grade	6.3	11.7	1.7
7. Not stated			
8. Other curves - level	5.3	2.4	-
9. Grade - level	2.1	-	-

Time:	Dexter-Pinckney Road	Plymouth Road	Jackson Road
Midnight to 12:59 a. m.	7. 4	4. 3	5. 4
1:00 a. m. to 1:59 a. m.	5. 0	3. 2	1. 5
2:00 a. m. to 2:59 a. m.	9. 9	3. 2	3. 0
3:00 a. m. to 3:59 a. m.	2. 5	3. 2	1. 5
4:00 a. m. to 4:59 a. m.	1. 3	-	3. 0
5:00 a. m. to 5:59 a. m.	1. 3	3. 2	2. 5
6:00 a. m. to 5:59 a. m.	2. 5	3. 2	2. 0
7:00 a. m. to 7:59 a. m.	1. 3	5. 3	3. 5
8:00 a. m. to 8:59 a. m.	8. 6	4. 3	3. 0
9:00 a. m. to 9:59 a. m.	2. 5	3. 2	4. 0
10:00 a. m. to 10:59 a. m.	2. 5	4. 2	4. 0
11:00 a. m. to 11:59 a. m.	2. 5	4. 3	5. 4
Noon to 12:59 p. m.			
1:00 p. m. to 1:59 p. m.	1. 3	2. 1	4. 4
2:00 p. m. to 2:59 p. m.	6. 6	8. 5	6. 9
3:00 p. m. to 3:59 p. m.	3. 7	6. 4	5. 4
4:00 p. m. to 4:59 p. m.	7. 4	2. 1	6. 4
5:00 p. m. to 5:59 p. m.	3. 7	6. 4	5. 4
6:00 p. m. to 6:59 p. m.	2. 5	3. 2	4. 0
7:00 p. m. to 7:59 p. m.	2. 5	3. 2	7. 5
8:00 p. m. to 8:59 p. m.	1. 3	4. 2	7. 5
9:00 p. m. to 9:59 p. m.	8. 6	2. 1	6. 4
10:00 p. m. to 10:59 p. m.	3. 7	4. 3	4. 4
11:00 p. m. to 11:59 p. m.	4. 9	4. 3	4. 4
Age of Drivers:			
0-14			
15-19	17. 5	1. 5	2. 8
20-24	22. 0	17. 1	13. 9
25-29	22. 0	18. 6	15. 9
30-39	16. 6	24. 6	2. 7
40-49	9. 6	20. 1	19. 6
50-59	8. 8	9. 7	14. 5
60-69	3. 5	6. 7	4. 0
69-70	-	-	0. 6
70 and over	-	1. 5	2. 0
Age of Vehicle:			
0-0. 9	14. 4	6. 9	1. 5
1-1. 9	23. 2	10. 3	4. 7
2-2. 9	13. 5	4. 8	9. 1
3-3. 9	12. 6	7. 6	7. 0
4-4. 9	12. 6	19. 2	18. 8
5-5. 9	10. 8	11. 7	11. 5
6-6. 9	5. 4	13. 7	11. 8
7-7. 9	1. 8	7. 6	11. 2
8-8. 9	-	9. 6	9. 1
9 and over	5. 4	8. 9	15. 0
Experience:			
0-6 mo.	3. 8	2. 5	0. 3
6. 1-1	4. 8	1. 6	1. 6
1. 1-2	3. 9	4. 2	3. 1
2. 1-3	8. 6	2. 5	2. 5
3. 1-4	3. 8	3. 3	5. 3
4. 1-5	6. 7	5. 8	4. 0
5. 1-6	4. 8	2. 5	3. 1
6. 1-7	4. 8	3. 3	4. 0
7. 1-8	4. 8	2. 5	2. 8

Experience (continued):	Dexter-Pinckney Road	Plymouth Road	Jackson Road
8.1-9	3.8	-	2.8
9.1-10	5.7	5.8	3.1
10.1-15	15.4	14.0	17.2
15.1-20	9.6	12.4	10.3
20.1-25	6.8	12.4	11.3
25.1-30	8.6	6.6	8.8
30.1-40	2.9	18.2	16.3
40.1 and over	11.0	3.3	3.4
Male	70.5	87.5	86.5
Female	29.5	12.5	13.5
Day:			
Sunday	27.4	18.1	15.4
Monday	10.7	9.6	16.9
Tuesday	4.8	10.7	10.0
Wednesday	9.5	12.8	7.0
Thursday	10.7	16.0	13.5
Friday	9.5	10.7	15.4
Saturday	27.4	22.4	22.0
Date:			
January	7.2	10.4	5.2
February	9.6	10.4	9.4
March	1.2	4.2	6.8
April	9.6	9.4	13.6
May	10.9	6.3	9.4
June	8.5	6.2	12.0
July	12.0	6.3	12.5
August	7.2	6.2	7.8
September	4.8	10.4	6.8
October	13.2	12.5	8.4
November	10.9	5.2	6.3
December	4.8	12.5	7.3
Condition of Driver:			
Defective vision (no glasses)	1.5	0.6	-
Same (wearing glasses)	4.6	3.0	4.9
Ill	-	0.6	-
Fatigued	-	2.9	-
Asleep	2.3	-	1.5
Other handicaps	1.6	-	0.5
Condition not known	7.8	9.8	6.0
Normal	62.0	63.5	61.0
Had been drinking	-	-	5.4
Drunk	3.1	3.5	1.0
Ability impaired	-	1.1	1.0
Ability not impaired	6.2	2.3	1.5
Not known whether impaired	3.1	2.9	2.5
Not known whether drinking	8.5	10.3	11.4
Vehicle Condition:			
Defective brakes	2.0	-	-
Improper lights	2.0	-	-
Steering	-	-	-
Tires	-	-	-
Windshield wiper	-	-	-
Other defects	11.0	0.7	8.1
No defects	52.0	33.4	38.8
Not known	45.0	66.0	52.5

	Dexter-Pinckney Road	Plymouth Road	Jackson Road
What Drivers Were Doing:			
1. Going straight ahead	61.0	60.5	64.0
2. Making right turn	6.0	2.0	0.8
3. Making left turn	4.3	11.9	6.1
4. Making U-turn	-	-	0.5
5. Slowing or stopping	-	2.6	2.8
6. Starting in traffic lane	-	-	-
7. Starting from parked position	-	0.7	0.5
8. Stopped in traffic lane	5.2	1.3	1.3
9. Parked	10.9	0.7	1.3
10. Backing	-	0.7	-
11. Passing or overtaking	3.5	4.0	8.8
12. Avoiding vehicle, obj. or pedestrian	2.6	3.3	1.3
13. Skidding - before applying brakes	3.5	4.0	2.6
14. Skidding - after applying brakes	12.1	8.6	8.5
15. Driverless moving vehicle	-	-	2.5
16. Hit and run	-	-	2.5
Point of Impact:			
Front	35.6	20.6	22.8
Right front	6.7	8.4	13.5
Left front	7.7	16.0	12.5
Right side	14.4	10.3	10.2
Left side	10.6	14.1	12.3
Rear	12.5	9.6	10.0
Right rear	1.0	5.2	5.6
Left rear	6.7	6.4	8.2
All sides - car rolled over	1.0	0.6	-
Vision Obscured:			
Windshields - snow, rain	7.9	11.9	5.1
Trees, crops	-	-	-
Buildings	-	-	-
Signboard	1.0	0.8	-
Embankment	-	-	1.3
Hill evert	-	-	1.3
Parked cars	2.0	-	1.0
Moving cars	8.9	1.6	9.3
Other	2.0	3.2	2.9
Not obscured	78.5	90.5	86.0
Traffic Control:			
Warning sign or signal	-	18.2	-
Stop sign or signal	58.5	59.0	8.4
Blinker	-	9.1	10.8
Speed zone	41.5	13.6	80.0
Weather:			
Clear	45.0	51.5	53.0
Cloudy	17.8	27.0	21.0
Rain	19.2	9.7	14.4
Snow	6.9	7.6	6.1
Fog	8.3	4.3	4.4
Sleet	2.8	-	1.1
Light Darkness - Not lighted	40.5	33.4	34.2
Daylight	49.5	56.0	56.5
Dusk	2.9	7.6	6.7

	Dexter-Pinckney Road	Plymouth Road	Jackson Road
Light Darkness (continued):			
Dawn	7.3	21.5	2.8
Road Conditions:			
Dry	53.5	60.0	65.5
Wet	26.6	21.2	22.0
Muddy	0.0	-	-
Snowy	10.6	8.8	3.3
Icy	9.4	10.0	9.3
Speed Stopped:			
0-5	7.9	4.3	5.2
5.1-10	5.6	4.3	1.8
10.1-15	1.1	5.1	3.7
15.1-20	2.2	2.5	2.1
20.1-25	10.1	2.5	4.3
25.1-30	3.4	2.5	3.4
30.1-35	1.1	4.3	5.8
35.1-40	13.5	11.9	9.2
40.1-45	11.2	16.1	10.1
45.1-50	19.1	9.3	12.9
50.1-55	9.0	18.6	15.3
55.1-60	6.8	5.9	16.0
60.1-65	4.5	4.3	5.5
65.1-70	3.4	3.4	3.4
70.1-80	-	3.4	0.9
80.1 and over	-	1.7	0.3

COMPARISON OF ACCIDENT RECORDS

A comparison of the traffic accident records on the three sections reveals some striking differences.

The approximate number of miles traveled per accident for the three roads were respectively, Dexter-Pinckney Road, 287,600 mi; Plymouth Road, 690,000 mi; and Jackson Road, 500,000 mi. These calculations were made from the number of accidents per year and the average daily traffic; 3,000 vehicles per day for the Dexter-Pinckney Road, 7,500 for the Plymouth Road and 9,000 for the Jackson Road, as reported in 1955 by the Traffic and Planning Division of the Michigan State Highway Department.

The accident frequencies per million vehicle miles and the corresponding quality indices are shown in the following table:

Road	Average Quality Index	Accidents Per Million Vehicle Miles
Dexter-Pinckney Road (two-lane)	618	3.48
Plymouth Road (two-lane)	1023	1.45
Jackson Road (three-lane)	1930	2.00

The single vehicle accidents which include "with fixed object," and "ran-off-road" amounted to 43.0 percent of the total accidents for the Dexter-Pinckney Road, 36.2 percent for the Plymouth Road and 15.4 percent for the Jackson Road. The Dexter-Pinckney Road, which is the most curving and has the most uneven surface has the worst record.

The accident frequencies for single vehicle accidents are compared in the following table:

Road	Average Quality Index	Single Vehicle Accidents Per Mil. Veh. Miles
Dexter-Pinckney Road	618	1.47
Plymouth Road	1023	0.52
Jackson Road	1930	0.31

The Dexter-Pinckney Road had the highest speed for the existing conditions. Excessive speed on this road accounted for 81.0 percent of the traffic violations, while the percentage on the Plymouth Road was 48.5 percent and on the Jackson Road, 40.8 percent.

The number of accidents on curve sections roughly paralleled the percentage of curvature, the respective percentages being as follows:

Dexter-Pinckney Road	34.8 percent
Plymouth Road	27.6 percent
Jackson Road	3.4 percent

The percentages of accidents occuring between midnight and 3:00 a.m. is perhaps significant.

Dexter-Pinckney Road	21.3 percent
Plymouth Road	10.6 percent
Jackson Road	9.9 percent

The percentage of accidents happening in the three hours after midnight is more than twice as much as on either of the other two sections of roads. Local comment has it that the presence of a roadhouse located just beyond the north end of the section studied has something to do with the accidents occuring about closing time for the roadhouse. If, however, the conditions of the drivers involved in accidents is compared it is found that there is no significant difference for this road and the other two sections. It may be mentioned that the Dexter-Pinckney is more of a summer resort than the other two. But the number of accidents happening during July, August and September is not greater for this section than for the other two:

Dexter-Pinckney Road	24.0 percent
Plymouth Road	22.9 percent
Jackson Road	27.1 percent

Another item of interest is the percentages of accidents corresponding to the ages of the drivers involved:

	19 yr (or under)	24 yr (or under)
Dexter-Pinckney Road	17.5 percent	39.5 percent
Plymouth Road	1.5 percent	18.6 percent
Jackson Road	2.8 percent	14.7 percent

It can be seen that more young drivers were involved in the accidents on the Dexter-Pinckney Road than the other two roads.

The percentages of accidents on the three roads for drivers with one year or less experience were as follows:

Dexter-Pinckney Road	8.6 percent
Plymouth Road	4.1 percent
Jackson Road	1.9 percent

This shows that not only were more young drivers involved in the total number of accidents on the Dexter-Pinckney Road, but also that more drivers with less experience were involved.

The weekend accidents were much higher for the Dexter-Pinckney Road. The percentages for the Dexter-Pinckney, the Plymouth and the Jackson roads were respectively, 54.8 percent, 40.5 percent, and 37.4 percent.

The items just mentioned include the most significant. Others of less significance may be found among those tabulated. These factors are to be considered in comparing the quality of flow on the three sections of highway with the accident frequencies. Certain human factors are to be taken into account in comparing the quality of flow on the three sections of highway with the accident frequencies. Some human factors, such as the age involved, could be deemed to be independent of the road, the explanation being that a younger than average group of people travel the Dexter-Pinckney Road. On the other hand, it could be that the younger driver has more trouble in driving over a winding road with an uneven surface. Without knowing the age distribution of the drivers on the roads it is impossible at present to properly evaluate the age factor.

Regardless of the possible variation in driving behavior between large groups of drivers it is believed that the over-all human characteristics of any two large groups are much the same.

In a broad sense the highway environment can account for differences in human characteristics. The average age of drivers on a large university campus must be younger than drivers at large.

The main purpose of this study has been the obtaining of the correlation between the quality index and the accident frequency. It has been assumed that the primary reason for the differences in the quality of flow has been the differences in the roadways and their immediate environment. It is recognized, nevertheless, that part of the variation in the quality of flow could be due to the driver characteristics. In any case the characteristics of the highway and of the driver are the causes, not measures, of the differences in the quality of driving. Whatever the reason, if low quality coincides with high frequency of accidents, quality may be used to anticipate highway mishaps.

The Quality Index and Highway Accidents

The average quality index Q_F for the Dexter-Pinckney Road is shown graphically. The average for the section studied is 618 with the variation for each half mile ranging from a low of 400 to a high of 840. The average index for Q'_F is much less than Q_F due to the method of calculating Q'_F. The omission of the factor, $\sqrt{f}$, in the denominator tends to increase the value of Q_F over Q'_F, but the change of the K value of from 1,000 to 100 tends to decrease the ratio. The average speed equals 47.7 mph.

The values from which the graphical representation is made up are shown in Table 1. The values are for each half mile beginning at the south end of this section of highway.

TABLE 1

DEXTER-PINCKNEY ROAD

Q Values	403	620	745	682	703	565	588	849	706	545	414	
Standard D	285	286	410	391	429	369	279	586	573	407	342	
Standard Error of Mean	51 3	52 2	72.6	69.1	75 8	66 3	48.5	108.9	102 9	74 4	63.5	
Q'	84	131	159	164	142	153	135	170	164	171	115	96
SD	68 0	55 7	85 2	89.0	63 3	89.6	62 1	71 7	66 0	85.6	73 8	99 8
SEofM	14 2	10 0	15.1	16.5	11.8	16.4	11.2	12 9	12 1	16 2	14.5	25 8
Speed	43.8	46 3	47.4	48 1	48 3	47 5	47 0	48.7	48.8	48.1	48 4	48.6
SD	6.70	6 65	6.22	7 37	6 53	7.15	6 32	7 85	7.47	8.18	8 78	8.34
SEofM	1 46	1 24	1 12	1.32	1.15	1.26	1 12	1 43	1 39	1 47	1 66	2 64

The quality of flow was calculated for each half mile with the thought that the frequency of accidents might vary directly with the fluction in the Q values. Apparently this is not the case as can be seen by comparing the quality with the accident frequency spotted in Figure 4. This could be due to the lack of sufficient runs or the fact that the fluction in the quality is not sensitive enough to respond to a change in road features in a short distance.

Quality of Flow on Plymouth Road

The average quality of flow on the Plymouth Road was 1023. The range for the half-time sections varied from 397 to 1684. The average Q'_F was 198 with a range from 141 to 252. The average speed was 47. 5 mph. These values are given in Table 2.

TABLE 2

PLYMOUTH ROAD

Q Values	397.0	836 7	1145 8	1208.4	1282.2	790.6	799.1	541 7	866 7	1498.6	1683 5	1223 2
SD	203 5	598.7	1224 3	774.2	865 1	486.5	854.5	386 9	1363 2	1478 1	1558.7	1051 8
SEofM	54.38	124.83	244.87	154.84	169.66	95.41	164.45	74 46	272.65	289.8	318 18	262 96
Q'	252	208	155	235	219	169	177	174	195	196	245	141
SD	103.8	119.5	63.2	215.0	161 6	58.5	110.8	87.7	191.7	143.6	213 0	69.2
SEofM	31 3	26 1	13.2	43.9	31.7	11 7	22.2	17 5	38.3	28 7	43 5	17.9
Speed	43.4	46 0	47.1	50 1	49.2	47 0	45.6	43.4	45 8	49.3	52.1	51.5
SD	7.19	4.53	4.43	6.37	6 08	6 03	4.45	6 69	5.78	6 55	6.13	5.65
SEofM	2 09	0.99	0.92	1.30	1 24	1.21	0.89	1.31	1 16	1 29	1 28	1 57

If the quality of flow values are compared with the spot locations of the accidents, it is found that there is no correlation that may be detected. But if the Q_F for the entire section of the Plymouth Road is compared with that of the Dexter-Pinckney Road there is found to be a fairly good correlation.

Quality of Flow on the Jackson Road

The quality of flow on the Jackson Road was the highest of the three sections of road, being 1930. Other flow factors are given in Table 3.

TABLE 3

JACKSON ROAD

Q Values	1657	1831	2200	2407	2293	1486	1302	1876	2779	2014	1745	1741	1550	2230
SD	1345	2413	1835	2008	2145	1083	879	1184	1826	1431	1480	1342	1217	2245
SEofM	293.5	433 4	314.7	339 4	362 6	180 5	142 6	192.1	296.2	235 2	243 3	223 7	205 7	478 7
Q'	267	297	330	285	353	404	315	225	261	387	371	368	334	277
SD	188.6	196 0	220 6	200.7	226 7	276 7	171.1	108 0	160 2	282 7	220 3	290 3	207 7	175 4
SEofM	39.3	32.7	36 3	33 0	36 8	44.9	27 5	17.3	26 3	47 8	37 2	49 1	36 7	39 2
Speed	55.0	52.4	52 9	54.9	54.4	54.2	53 6	52.2	53.9	55.3	56.6	55 8	58 1	58 0
SD	8 28	7.38	7.23	8 15	7 91	6 70	7 61	7.21	9.35	8.50	7 64	8 47	8 90	7 87
SEofM	1 73	1 26	1.22	1 34	1 28	1.10	1 23	1 17	1 56	1 43	1 29	1 43	1 59	1 76

A study of the data shows no definite correlation of the quality of flow with accident frequencies. But the over-all value shows a distinctly higher flow quality than either of the other two sections. This is a different type of road, being three-lane with little curvature. The accidents per million miles of travel are greater than on the Plymouth Road. This is due to the preponderance of multiple vehicle accidents. This is to be expected on a three-lane road. It could account for the fact that the accidents per miles traveled is higher on this road as compared to the Plymouth Road where the Q_F value is lower.

DISCUSSION

The findings indicate that the flow index does not have the same correspondence with accident frequency on a three-lane road as on a two-lane road. The three-lane road presents a hazard in the rivalry for the use of the middle lane. Perhaps the freedom to use the middle lane for passing gives the driver a false sense of security in its use. On a two-lane road the right-of-way is clearly understood.

Since the drive-ability features of a highway largely determine the quality of traffic flow that may lead to accidents it is essential that there be developed the devices needed to measure this drive-ability. The features of a roadway that influence drive-ability consist of its geometric design, its surface condition, and the appearance of the road and its immediate surroundings.

Since the volume of traffic not only increases the accident exposure but affects the quality of flow, the amount of traffic flow must be considered in any study of traffic accidents.

It became clear during the study that change of direction should be taken into account in the quality index. A vehicle rounding a curve at constant speed has an increase in velocity due to change in direction.

Speed alone gives no indication of the added effort of driving on a curving road, turning out to pass, or the dangers involved in these maneuvers. Change of direction as well as change of speed requires effort on the part of the driver and thus increases his annoyance. Apparently the less effort involved in driving the better the driver is satisfied.

Adding a factor to give the change in direction must be accomplished without altering the dimensionless character of the index. This can be done by expressing the change of direction in radians which have no dimension.

In order to avoid introducing the factor zero when there is no change of direction, the factor may be one, plus the change of direction. If there is no change, dividing the quality index by one does not change its value. This same reasoning applies to the change of speed.

To express this quality index symbolically, let Δ_θ = change in direction per mile, and the other symbols be as given at the first of the report. The index becomes:

$$Q_F = \frac{KS}{(1 + \Delta_s)\ (1 + \Delta_\theta)}$$

A means of recording the change of direction has not been devised but it should be possible. One means could be the recording of the amount of turning of the steering wheel.

It is believed that once a sufficient number of highways together with their accident records have been studied that standards of flow performance may be established, and that these standards may be used to predict the accident proneness of a roadway. It also follows that the features of a roadway that cause low quality of flow can be detected and removed or changed. It is hoped that the use of the quality of flow index may lead to the construction of safer highways.

Economic Costs of Motor Vehicle Accidents

ROBIE DUNMAN, Transportation Economist, Bureau of Publiċ Roads

In December 1947, the Highway Research Board recommended that the Bureau of Public Roads cooperate with the states in conducting economic cost of motor-vehicle accident studies. These studies are now underway in Massachusetts, New Mexico and Utah, and a fourth study is programed in Wisconsin. On the basis of preliminary discussion, it is anticipated that a fifth study will be started in Michigan early this year.

This paper uses data developed by the Massachusetts Department of Public Works and by the Massachusetts Registry of Motor Vehicles in cooperation with the Bureau of Public Roads. It is emphasized that the findings may not be typical or average for all states.

The number, direct cost, and average direct cost of motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953 are shown by accident type and severity of accident. Similar data are presented for accidents involving passenger cars in collision with other motor vehicles by type of collision, severity of accident, and for accidents involving passenger cars that were not in collision with other motor vehicles. Accident and cost rates—number of accidents and direct cost of accidents per hundred million vehicle-miles of travel—are included.

● THE PURPOSE of this paper is to present the costs of accidents in relation to the types of accidents in a way that will be helpful to all those groups and individuals that are trying to reduce traffic accidents and the economic loss thereof.

Procedure

Statistical studies of the economic cost of motor-vehicle accidents are based on a probability sample of the accident experience of vehicle owners. They are designed and conducted to be accurate within 10 percent. By means of mailed questionnaires and through personal interviews with the selected vehicle owners, their accident experience for 1 yr is obtained. From these data the direct cost of accidents is estimated and correlated with the more important characteristics of accidents including those characteristics peculiar to the highway and street facilities, the driver, and the vehicle. These studies are statewide and comprehensive. Because the data collected and analyzed in each state are so detailed an voluminous, this paper is confined to one segment of the comprehensive study of traffic-accident costs in Massachusetts during 1953.

Definitions

The narrative, tabulations, and charts that follow apply only to accidents in which passenger cars were involved. The accidents are "motor-vehicle traffic accidents" occurring on public roadways and involving motion.

"Direct costs" are defined as the money value of damages and losses to persons and property that were the direct result of these accidents and which might have been saved for the car owner had these accidents not occurred. Direct costs are composed of the money value of damage to property; hospitalization; doctors, dentists, and nursing service; ambulance use; medicine; work time lost; damages awarded in excess of other direct cost; attorney's services; court fees; and other miscellaneous but small items.

The type of collision was determined by the direction of travel of the vehicles involved before the collision, not by what took place because of efforts on the part of the drivers to avoid collision. Thus, any collision involving an intentional change of direction, such as a right, left, or U-turn, was classified as a turning movement, even though this may have resulted in a head-on or rear-end type of collision. Similarly, an angle collision resulted when two or more vehicles, each traveling in a straight line, came together at an intersection. Though one or both drivers swerved to avoid impact and only struck each other in a sideswipe fashion, this was still classified as an angle collision. On the same basis, a collision involving any vehicle entering or leaving a parking space was classified as a parking maneuver, though the vehicle struck, or was struck by another, in a head-on, rear-end, or sideswipe fashion. A sideswipe occurred only when one vehicle was overtaking another going in the same direction, or passing a vehicle traveling in the opposite direction, or passing a parked vehicle, and when one came too close to the other, either on a straight road or a curve.

TABLE 1

MOTOR-VEHICLE TRAFFIC ACCIDENTS INVOLVING PASSENGER CARS BY SEVERITY OF ACCIDENT IN MASSACHUSETTS DURING 1953

Severity of Accident	Number of Accidents	Percent of Total	Number of Accidents per 100 Million Vehicle-Miles
Fatal injury	315	0. 2	3[1]
Nonfatal injury	33, 270	25. 3	287
Property damage only	97, 915	74. 5	842
All accidents	131, 500	100. 0	1;132

[1] Rounded from 2. 7.

All other definitions are from the manual, "Uniform Definitions of Motor-Vehicle Accidents, 1947," prepared under the auspices of the National Conference on Uniform Traffic Accident Statistics and published by the Federal Security Agency, United States Public Health Service.

Background

In 1953 the population of Massachusetts was 4, 773, 000. There were 1,239,000 registered passenger cars and 1,858,000 licensed operators who drove their cars 11,628,000,000 vehicle miles over the Commonwealth's 24,500 miles of streets and highways. These passenger-car operators experienced 222,000 involvements in 131,500 accidents that resulted in a direct cost of $50,224,000.

ACCIDENT SEVERITY

All motor vehicle traffic accidents fall into one of three severity classes— property-damage-only accidents, nonfatal-injury accidents and fatal-injury accidents.

Table 1 reflects the accident experience of Massachusetts passenger-car operators during 1953 and it brings into focus the numerical relationship of accidents of different severity.

Injury to Fatal Accident Ratio

The 33,270 nonfatal-injury accidents and the 315 fatal-injury accidents shown in Table 1 are in the ratio of 106:1. This injury to fatal accident ratio of 106:1 is almost exactly 3 times the 35:1 injury to fatal accident ratio ordinarily used in estimating the cost of accidents. Whether or not this high injury to fatal accident ratio will hold in

TABLE 2

DIRECT COST OF MOTOR-VEHICLE TRAFFIC ACCIDENTS INVOLVING PASSENGER CARS BY SEVERITY OF ACCIDENT IN MASSACHUSETTS IN 1953

Severity of Accident	Total Direct Cost of Accidents	Percent of Total	Cost of Accidents per 100 Million Vehicle-Miles
Fatal injury	$ 1,642,000	3.3	$ 14,000
Nonfatal injury	28,688,000	57.1	247,000
Property damage only	19,894,000	39.6	171,000
All accidents	$50,224,000	100.0	$432,000

predominantly rural states will be known within a short time when results of the Utah and New Mexico studies are available.

Severity Class Relationships

It is apparent from Table 1 that 3 out of every 4 motor-vehicle traffic accidents result in property damage only, that 1 out of every 4 accidents results in a nonfatal injury and that only 1 in every 400 accidents results in a fatal injury.

It is also apparent from Table 1 that there are 106 times as many nonfatal-injury accidents and 311 times as many property-damage-only accidents as there are fatal-injury accidents per 100 million vehicle-miles of travel.

The accident severity rate for passenger cars was as follows: 1 fatal accident for every 3,935 passenger cars registered, 1 nonfatal-injury accident for every 37 cars registered and 1 property-damage-only accident for every 13 cars registered.

The accident severity rate for licensed passenger-car drivers was 1 fatal-injury accident for every 5,897 drivers, 1 nonfatal-injury accident for every 56 drivers, and 1 property-damage-only accident for every 19 drivers.

Accident Rates

In addition to the accidents per 100 million vehicle-miles shown many other useful rate characteristics may be readily established by relating the figures in Table 1 to such factors as population, number of licensed operators, and the number of passenger cars registered. For example, there was 1 fatal accident for every 14,200 persons. There was 1 nonfatal-injury accident for every 134 persons, and there was 1 property-damage-only accident for every 46 persons.

Total Direct Cost

The number and severity of motor-vehicle traffic accidents are both reflected in the direct cost figure presented in Table 2.

It should be remembered in considering these costs that they apply only to accidents involving passenger cars and that they do not include any indirect costs such as the present value of future earnings and the overhead cost of motor-vehicle accident insurance.

Direct Cost Relationships

When the number of accidents as shown in Table 1 was compared with the costs shown in Table 2, it was found that 1 fatal accident with a direct cost of $5,212 is the equivalent of either 6 nonfatal injury accidents with a direct cost of $862 each or 25 property-damage-only accidents with a direct cost of $203 each.

However, it is also apparent from Table 2 that on the basis of 100 million vehicle-miles of travel, property-damage-only accidents cost 12 times as much and nonfatal-injury accidents, 18 times as much as the cost of fatal accidents.

Furthermore, Table 2 shows that from the economic point of view alone, nonfatal-injury accidents are by far the most important. They account for 57 cents of every accident direct cost dollar. Property-damage-only accidents are of great economic importance too. They account for 40 cents of every accident direct cost dollar. The emotion-packed and often highly dramatized fatal accident is, from the economic point of view, relatively unimportant. It accounts for only 3 cents of every accident direct cost dollar. This does not minimize the personal tragedy of fatal accidents.

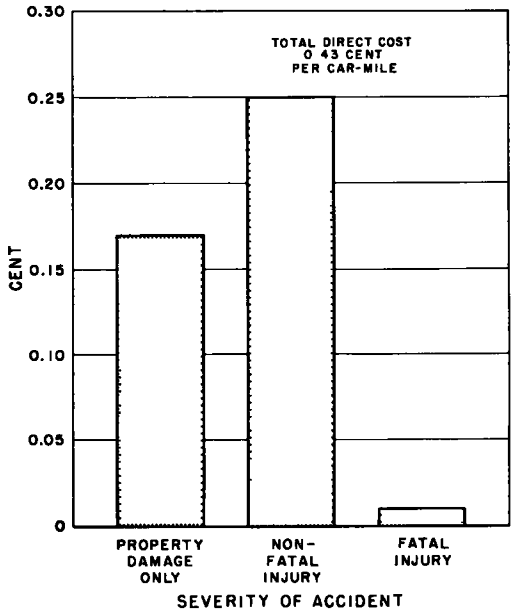

Figure 1. Direct cost of accidents of different severity per passenger car-mile of operation in Massachusetts in 1953.

Accident Cost Rates

By relating the direct costs in Table 2 to population, licensed operators, cars registered, and road mileage, accident cost rates were established as follows:

The per capita direct cost was $4. 17 for property-damage-only accidents, $6. 01 for nonfatal-injury accidents, and 34 cents for fatal-injury accidents, or a total accident direct cost of $10. 52 per capita.

The direct cost per licensed passenger-car operator was $10. 71 for property-damage-only accidents, $15. 44 for nonfatal-injury accidents and 88 cents for fatal-injury accidents, or a total accident direct cost of $27. 03 per passenger-car operator.

The direct cost per passenger car registered was $16. 05 for property-damage-only accidents, $23. 14 for nonfatal-injury accidents, and $1. 32 for fatal-injury accidents, or a total accident direct cost of $40. 51 per passenger car registered.

The direct cost per mile of road was $812 for property-damage-only accidents, $1,171 for nonfatal-injury accidents, and $67 for fatal-injury accidents, or a total accident direct cost of $2,050 per mile of road.

It is apparent from Figure 1 that the direct cost of operating a passenger car 1 mile was 17/100 of a cent for property-damage-only accidents, 25/100 of a cent for nonfatal-injury accidents, and 1/100 of a cent for fatal-injury accidents, or a total of 43/100 of a cent for each mile of passenger-car operation.

All motor-vehicle traffic accidents fall into one of five types—collision between motor-vehicles, collision with pedestrians, collision with fixed objects, collision with other objects, and the noncollision type accidents in which the vehicle turns over in the road or runs off the road without striking anything.

Numerical Relationships

It is evident from Table 3 that of all the accident types, collisions between motor-vehicles were by far the most numerous. More than 8 out of 10 of all the accidents involving passenger cars were of this type. Furthermore, out of a total of 1,132 accidents per 100 million vehicle-miles, 944 were passenger-car collisions with other motor-vehicles.

Second in importance from the standpoint of number of accidents were passenger-car collisions with objects, fixed and otherwise. This type of accident accounted for 1 out of 10 of all the accidents in which passenger cars were involved.

Ranked according to number of accidents, passenger-car collisions with pedestrians

TABLE 3

MOTOR-VEHICLE TRAFFIC ACCIDENTS INVOLVING PASSENGER CARS BY TYPE OF ACCIDENT IN MASSACHUSETTS DURING 1953

Type of Accident	Number of Accidents	Percent of Total	Number of Accidents per 100 Million Vehicle-Miles
Collision with:			
Other motor vehicles	109, 700	83. 4	944
Pedestrians	5, 900	4. 5	50
Fixed objects	7, 300	5. 5	63
Other objects	6, 400	4. 9	56
Noncollision accidents	2, 200	1. 7	19
All accidents	131, 500	100. 0	1, 132

were third in importance. About 1 in every 20 accidents involving passenger cars were of this type.

Least in importance from the standpoint of numbers alone were noncollision accidents. They account for less than 1 in 50 of all the accidents in which passenger cars were involved.

However, the number of accidents alone is not a true measure of the relative economic importance of accidents of different types—both the number and the severity of accidents must be considered to accurately measure economic importance.

Accident Rates

By comparing the number of accidents as shown in Table 3 with the population, passenger cars registered, and number of licensed drivers, accident rates were computed as follows:

There was 1 collision between motor vehicles for every 44 persons, 1 collision with pedestrians for every 815 persons, 1 collision with fixed objects for every 658 persons, one collision with other objects for every 740 persons, and 1 noncollision accident for every 2,150 persons.

There was 1 collision between motor vehicles for every 11 passenger cars, 1 collision with pedestrians for every 212 passenger cars, 1 collision with fixed objects for every 171 passenger cars, 1 collision with other objects for every 192 passenger

TABLE 4

DIRECT COST OF MOTOR-VEHICLE TRAFFIC ACCIDENTS INVOLVING PASSENGER CARS BY TYPE OF ACCIDENT IN MASSACHUSETTS IN 1953

Type of Accident	Total Direct Cost of Accidents	Percent of Total	Cost of Accidents per 100 Million Vehicle-Miles
Collision with:			
Other motor vehicles	$41, 816, 000	83. 3	$360, 000
Pedestrians	3, 375, 000	6. 7	29, 000
Fixed objects	3, 023, 000	6. 0	26, 000
Other objects	673, 000	1. 3	6, 000
Noncollision accidents	1, 337, 000	2. 7	11, 000
All accidents	$50, 224, 000	100. 0	$432, 000

cars, and 1 noncollision accident for every 558 passenger cars.

There was 1 collision between motor vehicles for every 17 drivers, 1 collision with pedestrians for every 317 drivers, 1 collision with fixed objects for every 256 drivers, 1 collision with other objects for every 288 drivers, and 1 noncollision accident for every 837 drivers.

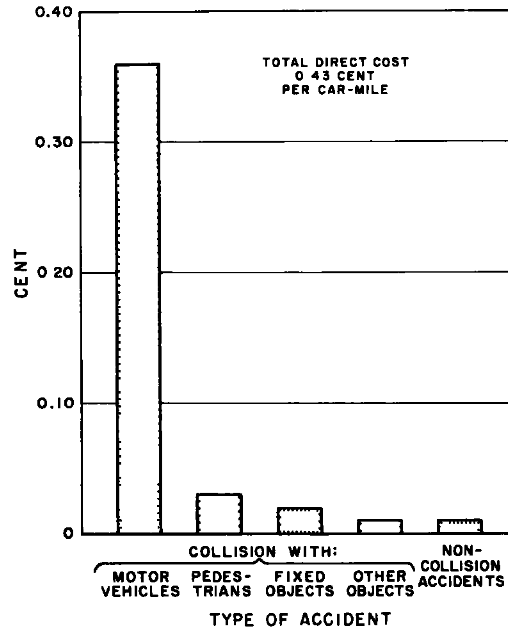

Figure 2. Direct cost of different types of accidents per passenger car-mile of operation in Massachusetts in 1953.

Cost Relationships

Using the number of accidents as shown in Table 3 and the direct costs shown in Table 4, the average direct cost for each type of accident was found to be as follows: collisions between passenger cars and/or other motor vehicles, $381; collisions of passenger cars with pedestrians, $577; collisions of passenger cars with fixed objects, $416; passenger car collisions with other objects, $104; and passenger car noncollision accidents, $553.

These average costs reflect the severity of accidents of different types. They do not reflect economic importance. Using them as a measure of severity it was found that passenger-car collisions with pedestrians were the most severe type of accident. Other types of accidents were ranked as follows: noncollision accidents second, passenger-car collisions with fixed objects third, collision between passenger cars and/or other motor vehicles fourth, and passenger-car collisions with other objects fifth.

The overriding economic importance of the collisions between motor vehicles is made clear by Table 4. Out of every accident direct cost dollar 83 cents goes for this type of accident. On the basis of 100 million vehicle-miles of travel, collisions between passenger cars and/or other motor-vehicles cost five times as much as all other types of accidents combined.

Second in economic importance was the passenger-car collision with pedestrians. Out of every accident direct cost dollar 7 cents was chargeable to this type of accident.

The collision with fixed objects ranked third in economic importance. Out of every accident direct cost dollar 6 cents was expended for this type of accident.

The fourth place in economic importance was occupied by the noncollision type of accident. Out of every accident direct cost dollar 3 cents was expended for this type of accident.

The collision with other than fixed objects was of the least economic importance of all. Only 1 cent of every accident direct cost dollar was expended for this type of accident.

Accident Cost Rates

By relating the direct cost figures in Table 4 to the population, licensed operators, cars registered, and road mileage figures, accident cost rates were established as follows:

The per capita direct cost was $8. 76 for collisions with other motor vehicles, 71 cents for collisions with pedestrians, 63 cents for collisions with fixed objects, 14 cents for collisions with other objects, and 28 cents for noncollision accidents.

The direct cost per licensed passenger-car operator was $22. 51 for passenger car collision with other motor vehicles, $1. 82 for collision with pedestrians, $1. 63 for

TABLE 5

NUMBER OF COLLISIONS BETWEEN PASSENGER CARS AND/OR OTHER MOTOR VEHICLES BY TYPE OF COLLISION IN MASSACHUSETTS IN 1953

Type of Collision	Number of Collisions Between Motor Vehicles	Percent of Total	Number of Collisions Between Motor Vehicles per 100 Million Vehicle-Miles
Angle	53, 200	48. 5	459
Rear-end	22, 500	20. 5	193
Head-on	12, 800	11. 7	110
Sideswipe - same direction	7, 100	6. 5	61
Parking maneuver	5, 200	4. 7	44
Turning movement	4, 800	4. 4	41
Backing in traffic lane	2, 600	2. 4	22
Sideswipe - opposite direction	1, 500	1. 3	13
All collisions between motor vehicles	109, 700	100. 0	943

collision with fixed objects, 36 cents for collision with other objects, and 72 cents for noncollision accidents.

The direct cost per passenger car registered was $33. 73 for passenger-car collision with other motor vehicles, $2. 72 for collision with pedestrians, $2. 44 for collisions with fixed objects, 54 cents for collisions with other objects, and $1. 08 for noncollision accidents.

The direct cost per mile of road was $1,707 for passenger-car collisions with other motor vehicles, $138 for collisions with pedestrians, $123 for collisions with fixed objects, $27 for collisions with other objects, and $55 for noncollision accidents.

The total passenger-car miles of travel was divided into the accident direct cost as shown in Table 4 to calculate the direct cost of each type of accident per mile of passenger-car operation. The results are presented in Figure 2.

Figure 2 indicates that the cost of operating a passenger car 1 mile was 36/100 of a cent for collisions between motor vehicles, 3/100 of a cent for passenger-car collisions with pedestrians, 2/100 of a cent for collisions with fixed objects, 1/100 of a cent for collisions with other objects, and 1/100 of a cent for noncollision accidents or a total accident direct cost of 43/100 of a cent per mile.

COLLISIONS BETWEEN MOTOR VEHICLES

All collisions between passenger cars and/or other motor vehicles fall into one of the eight collision types shown in Table 5.

Collision Type Relationships

In Table 5 the different types of collisions are listed in the order of their numerical importance. Angle collisions were by far the most numerous. Approximately half of the collisions between passenger cars and/or other motor vehicles were angle collisions.

Rear-end collisions ranked second. About 1 out of every 5 collisions was a rear-end collision.

The head-on collision ranked third. Approximately one in every 9 collisions was a head-on collision.

These three collision types—angle, rear-end, and head-on combined—accounted for 8 out of 10 of all the collisions between passenger cars and/or other vehicles.

There were less than ½ as many rear-end collisions and approximately ¼ as many head-on collisions as there were angle collisions per 100 million vehicle-miles of travel. On this same basis there were more rear-end collisions than there were collisions of the following five types combined: sideswipes in the same direction, parking-manuever collisions, turning-movement collisions, backing-in-traffic-lane collisions and sideswipes in the opposite direction. In fact, these five types together accounted for only about 1 in 5 of all the collisions between passenger cars and/or other motor vehicles.

Collision Rates

By comparing the number of collisions as shown in Table 5 with the population, vehicles registered, and drivers licensed, collision rates were established as follows:

For every 90 persons there was 1 angle collision. For every 212 persons there was 1 rear-end collision, and for every 373 persons there was 1 head-on collision. For every 225 persons there was a collision of one or another of the remaining five types of collisions between passenger cars and/or other motor vehicles.

For every 23 passenger cars there was 1 angle collision. For every 55 passenger cars there was one rear-end collision and for every 97 passenger cars there was 1 head-on collision. For every 58 passenger cars there was a collision of one or another of the remaining five types of collision between passenger cars and/or other motor vehicles.

For every 35 drivers there was 1 angle collision. For every 83 drivers there was 1 rear-end collision and for every 145 drivers there was 1 head-on collision. For every 88 drivers there was a collision of one or another of the remaining five types of collisions between passenger cars and/or other motor vehicles.

Total Direct Cost

It is evident from Table 6 that angle collisions rank far above all other types in economic importance. Almost 42 percent of the total direct cost of collisions between passenger cars and/or other motor vehicles result from angle collisions.

The rear-end collision is of major economic importance too. It ranks second and accounts for almost 26 cents of every dollar expended for collisions between passenger cars and/or other motor vehicles.

Also of considerable economic importance is the head-on collision, it ranks third

TABLE 6

DIRECT COST OF COLLISIONS BETWEEN PASSENGER CARS AND/OR OTHER MOTOR VEHICLES BY TYPE OF COLLISION IN MASSACHUSETTS IN 1953

Type of Collision	Total Cost of Collisions Between Motor Vehicles	Percent of Total	Cost of Collisions Between Motor Vehicles per 100 Million Vehicle-Miles
Angle	$17,385,000	41.6	$150,000
Rear-end	10,842,000	25.9	93,000
Head-on	9,078,000	21.7	78,000
Sideswipe - same direction	1,958,000	4.7	17,000
Parking maneuver	599,000	1.4	5,000
Turning movement	1,114,000	2.7	10,000
Backing in traffic lane	133,000	0.3	1,000
Sideswipe - oposite direction	706,000	1.7	6,000
All collisions between motor vehicles	$41,815,000	100.0	$360,000

and accounts for almost 22 cents of every dollar expended for collisions between passenger cars and/or other motor vehicles.

The five remaining types of collisions between passenger cars and/or other vehicles are of considerably less economic importance. Together they account for less than 11 cents of every dollar expended for collisions between passenger cars and/or other motor vehicles.

Their rank in economic importance among all collisions between passenger cars and/or other motor vehicles was: sideswipes in the same direction, fourth; turning-movement collisions, fifth; sideswipes in the opposite direction, sixth; parking-maneuver collisions, seventh; and backing-in-traffic-lane collisions, eighth.

By dividing the costs as shown in Table 6 by the number of collisions as shown in Table 5 the average cost of each type of collision was computed with results as follows:

Angle collisions, $327; rear-end collisions, $482; head-on collision, $709; sideswipes in the same direction, $275; parking-maneuver collisions, $115; turning-movement collisions, $232; backing-in-traffic-lane collisions, $51; and sideswipes in the opposite direction, $471.

The direct cost of operating a passenger car 1 mile as shown in Figure 3 was 15/100 of a cent for angle collisions, 9/100 of a cent for rear-end collisions, 8/100 of a cent for head-on collisions, and 4/100 of a cent for all other types of collisions between passenger cars and/or other motor vehicles combined. The total direct cost of all types of collisions between passenger cars and/or other motor vehicles was 36/100 of a cent per mile of passenger-car operation.

PASSENGER CAR ACCIDENTS OTHER THAN COLLISIONS BETWEEN MOTOR VEHICLES

Passenger-car accidents other than collisions between motor vehicles are ordinarily classified into 8 types as follows: (1) collisions with fixed objects, (2) collisions with pedestrians, (3) collisions with bicycles, (4) collisions with animals or animal-drawn vehicles, (5) collisions with railroad trains, (6) collisions with streetcars, (7) collisions with other objects, and (8) noncollision accidents.

However, since accident types 3, 4, 5, and 6 together accounted for less than 3,000 of the almost 22,000 passenger-car accidents other than collisions between motor vehicles, these 4 accident types were combined with other objects in Tables 7 and 8.

Numerical Relationships

Of the 21,800 accidents other than collisions between passenger cars and/or other motor vehicles, 1 out of every 10 was a noncollision type in which the vehicle turned over in the road or ran off the road without striking anything. In 9 out of every 10 of them a passenger car struck something other than another motor vehicle.

In approximately 1 out of every 4 of these accidents a passenger car struck a pedestrian. In about 1 out of 3 of them a passenger car struck a fixed object, and in about 3 out of 10 of them a passenger car struck one of the miscellaneous group of things listed under other objects.

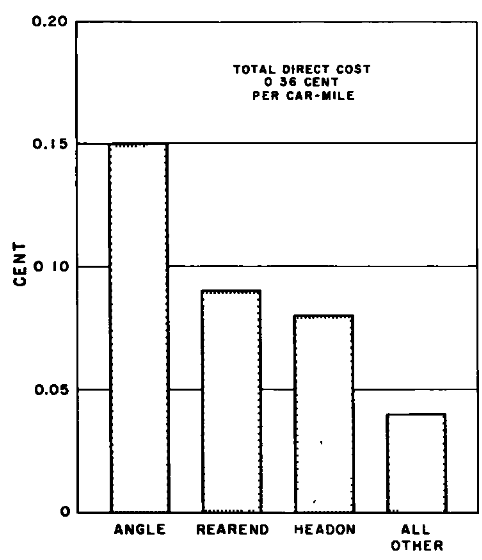

Figure 3. Direct cost of collisions between passenger cars and/or other motor vehicles per passenger car-mile of operation in Massachusetts in 1953.

TABLE 7

NUMBER OF ACCIDENTS INVOLVING PASSENGER CARS OTHER THAN COLLISIONS BETWEEN MOTOR VEHICLES BY TYPE OF ACCIDENT IN MASSACHUSETTS IN 1953

Type of Accident	Number of Accidents Other than Collisions Between Motor Vehicles	Percent of Total	Number of Accidents Other than Collisions Between Motor Vehicles per 100 Million Vehicle-Miles
Passenger-car collisions with:			
Pedestrians	5,900	26.9	50
Fixed objects	7,300	33.9	63
Other objects	6,400	28.9	55
Total	19,600	89.7	168
Noncollision accidents	2,200	10.3	20
All accidents other than collision between motor vehicles	21,800	100.0	188

Accident Rates

By comparing the number of accidents as shown in Table 7 with the population, passenger cars registered, and licensed drivers, accident rates were computed as follows:

There was 1 passenger-car collision with a pedestrian for every 809 persons, 1 collision with a fixed object for every 654 persons, 1 collision with other objects for every 746 persons, and 1 noncollision accident for 2,170 persons.

There was 1 passenger car collision with a pedestrian for every 315 drivers, 1 collision with a fixed object for every 255 drivers, 1 collision with other objects for every 290 drivers, and 1 noncollision accident for every 845 drivers.

TABLE 8

DIRECT COST OF ACCIDENTS INVOLVING PASSENGER CARS OTHER THAN COLLISIONS BETWEEN MOTOR VEHICLES BY TYPE OF ACCIDENT IN MASSACHUSETTS IN 1953

Type of Accident	Total Cost of Accidents Other than Collisions Between Motor Vehicles	Percent of Total	Cost of Accidents Other than Collisions Between Motor Vehicles per 100 Million Vehicle-Miles
Passenger-car collisions with:			
Pedestrians	$3,375,000	40.1	$29,000
Fixed objects	3,023,000	36.0	26,000
Other objects	673,000	8.0	6,000
Total	7,071,000	84.1	61,000
Noncollision accidents	1,337,000	15.9	11,000
All accidents other than collisions between motor vehicles	$8,408,000	100.0	$72,000

There was 1 passenger-car collision with a pedestrian for every 210 cars registered, 1 collision with a fixed object for every 170 cars registered, 1 collision with other objects for every 194 cars registered, and 1 noncollision accident for every 563 cars registered.

Total Cost

The costs shown in Table 8 are 16.7 percent of the total direct cost of all the motor-vehicle traffic accidents experienced by Massachusetts passenger-car drivers during 1953.

Cost Relationship

Among the four types of accidents—passenger car collisions with pedestrians, passenger-car collisions with fixed objects, passenger-car collisions with other objects, and noncollision accidents, collisions with pedestrians were of the greatest economic importance. Collisions with fixed objects ranked second, noncollision accidents ranked third, and collisions with other objects ranked last.

The average direct cost of passenger-car collisions with a pedestrian was $572.

The average direct cost of a passenger-car collision with fixed objects was $414.

The average direct cost of a passenger-car noncollision accident was $608.

The average direct cost of a passenger-car collision with other objects as used in Tables 7 and 8 was $105.

Cost Rates

By dividing the costs as shown in Table 8 by the population, passenger cars registered, and the number of licensed drivers cost rates were computed as follows:

For passenger-car collisions with pedestrians the per capita direct cost was 71 cents. The cost per passenger car registered was $2.72 and per licensed driver, $1.82.

For passenger-car collisions with fixed objects the per capita direct cost was 63 cents. The cost per passenger car registered was $2.44, and per licensed driver, $1.63.

For passenger-car collisions with other objects the per capita direct cost was 14 cents. The cost per passenger car registered was 54 cents, and per licensed driver was 36 cents.

For passenger-car noncollision accidents the per capita direct cost was 28 cents. The cost per passenger car registered was $1.08, and per licensed driver, 72 cents.

SUMMARY

The essence of this paper is presented in Figure 4.

Severity-Class Dollar

The severity class dollar as diagramed in Figure 4 is representative of the total 50,224,000 accident direct cost dollars. This diagram shows the economic importance of nonfatal-injury accidents to be greater than that of fatal accidents and property-damage-only accidents combined. The diagram also portrays the minor economic role of fatal accidents and the major economic impact of the great number of relatively inexpensive property-damage-only accidents.

Accident-Type Dollar

The accident type dollar as diagramed in Figure 4 is also representative of the total 50,224,000 accident direct cost dollars. This diagram brings accidents of different types into proper economic perspective. It illustrates the overriding economic importance of the collision between motor-vehicles type of accident and from the economic point of view, the relatively little importance of passenger-car collisions with other objects.

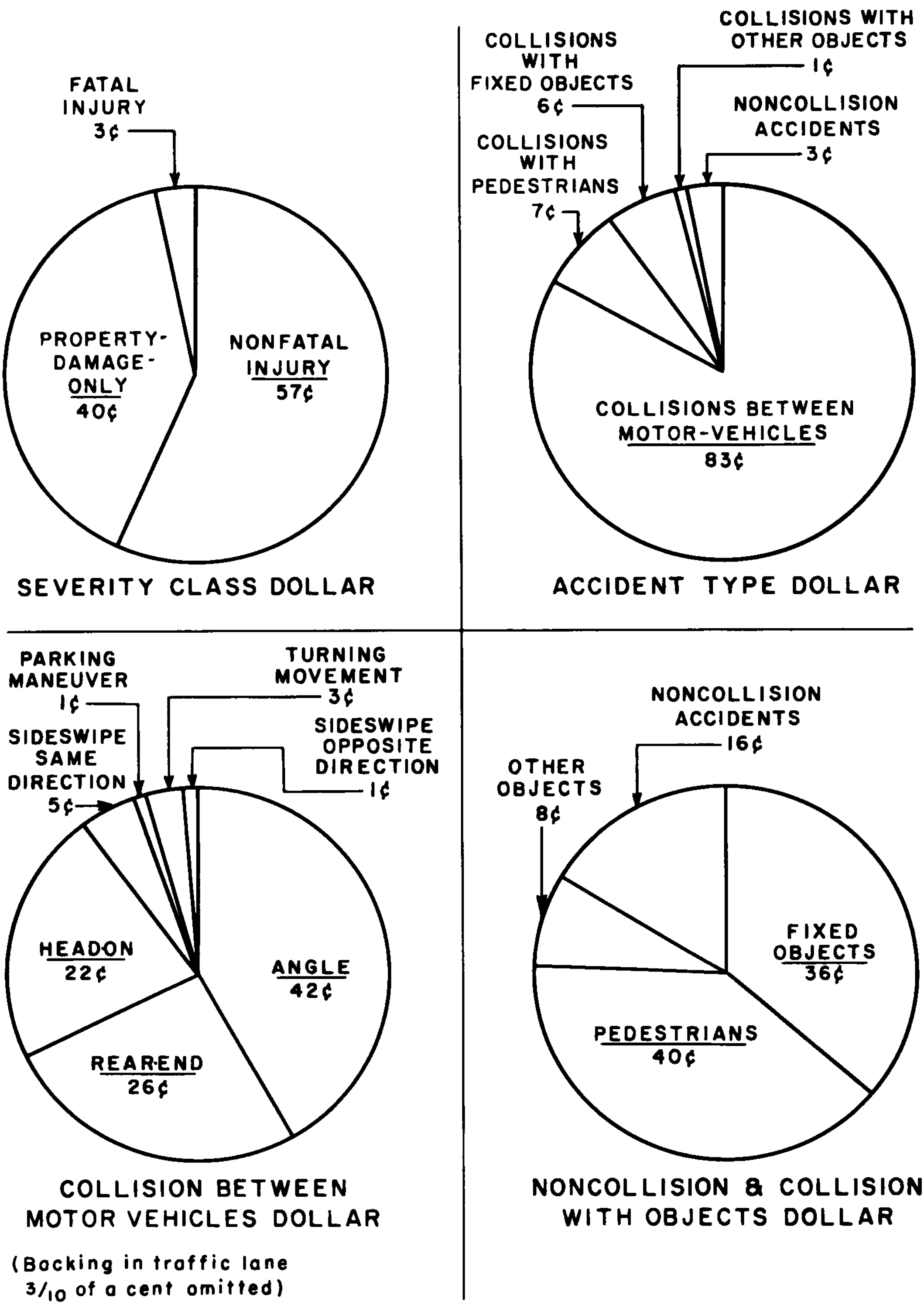

Figure 4. Where the accident direct cost dollar goes, Massachusetts, 1953.

Collisions-Between-Motor-Vehicles Dollar

The collisions-between-motor-vehicles dollar as diagramed in Figure 4 is the equivalent of the 83-cent segment of the accident-type dollar. It is representative of the 41,816,000 accident direct cost dollars expended for collisions between motor vehicles. This diagram illustrates the great economic importance of angle, rear-end, and head-on collisions and shows the remaining five collision types to be of little economic importance. These remaining five collision types together account for only 10.8 percent of the direct cost of collisions between motor vehicles.

Noncollision and Collision-with-Objects Dollar

The noncollision and collision-with-objects dollar as diagramed in Figure 4 is the equivalent of a 17-cent segment of the accident type dollar. It is representative of the 8,408,000 accident direct cost dollars expended for noncollision accidents and passenger-car collisions with objects other than another motor vehicle.

The diagram shows that of the four types of accidents included in the group, passenger-car collisions with pedestrians are, from the economic point of view, the most important and passenger-car collisions with other objects are the least important.

Statistical Evaluation of Traffic Accident Severity

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This paper is an attempt to create a scale of numerical values to be applied to traffic accidents, assigning a number to each accident consistent with the severity of the property damage or human injury involved.

A scale is set up which is based upon monetary damage values and American Standard injury classifications currently used in industrial injury study.

The scale was applied to 1,253 accidents at the Lincoln Tunnel during 1953, 1954, and 1955. Accident severity is compared on a monthly, daily, and hourly basis with accident frequency, accident rate and vehicular volume to determine the relationship of severity to other variables. Severity is compared with accident rate on the basis of weather, road, and light conditions.

The results are the following:

1. Severity as a monthly, daily, or hourly pattern does not in general follow the movements of accident frequency or accident rate.
2. Severity of accidents increases with a decrease of natural light.
3. Severity increases with poor road conditions and also with bad weather.

THE EXTENT to which severity is considered in studying traffic accident statistics at present is to classify accidents as property damage, personal injury, or fatal. No one will argue that locations, or times of day, week or year, in which fatalities have been prevalent should not be the first to receive consideration as to remedial and preventive measures. It also seems logical to turn the attention to areas with high frequencies of injury accidents. But the location with a great proportion of extremely disabling injuries is put, by this system, on a completely equal footing with the area commonly producing many minor bumps and bruises.

Fatality rates are, of course, useful as a measure of severity. However, at locations with few or no fatal accidents each year they are meaningless, in which case the number or rate of personal injury accidents becomes the yardstick of severity. The range in this classification, as in the classification of property damage, makes them rather crude.

In this paper an attempt is made to develop a numerical rating system, to be applied to individual traffic accidents, with the purpose of obtaining a more realistic picture of severity and, thereby, hazard or risk. It is felt that the true measure of risk at a location involves more than simply chance or risk of occurrence; it should give an indication of the hazard of degree of damage or injury. However, the concept of a severity-frequency "index" or "ratio" is beyond the scope of the present paper.

The basis of this approach is the assumption that the reporting of both injury and property damage is at a reasonably high level of accuracy. Otherwise there is no benefit in using anything other than the commonly accepted three classifications.

It was found, at least for the facility studied here, that a graph of the numbers of personal injury accidents is very similar to that of accident frequency; and that plotting either the proportion of personal injury accidents to the total number, or the rate of personal injury accidents on a volume basis, gives a picture which seems to lie somewhere between the accident frequency graph and the graph of severity values presented here.

SEVERITY SCALE

The first assumption made in creating a numerical scale was that while the relative position of accidents on the scale is all-important, the actual numbers assigned need have no relation to the true cost or effect of the accident, as long as the purpose in mind is merely comparison. The number need only separate the accident from other less or more severe accidents. With this in mind, a numerical range of 0 to 1,000 was set arbitrarily, with the upper limit equated to a fatality.

The next essential was to delimit the damage and injury classifications. In the field of industrial accident study, methods are already in use for evaluating injury. The source for the injury classifications used in this paper is, therefore, the "American Standard Method of Recording and Measuring Work Injury Experience," (American Standard Z 16.1-1954, American Standards Association), now used in industrial accident study. With some modifications and additions, the classifications and their relative positions used in the scale are as follows:

Death is any fatality resulting from a traffic accident injury regardless of the time intervening between injury and death.

Permanent Total Disability is any injury other than death which permanently and totally incapacitates a person from following any gainful occupation, or which results in the loss of or complete loss of use of any of the following in one accident:

1. Both eyes;
2. One eye and one hand, or arm, or leg, or foot; and
3. Any two of the following not on the same limb: hand, arm, foot, or leg.

Permanent Partial Disability is any injury other than death or permanent total disability which results in the complete loss of use of any member or part of a member of the body, or any impairment of the functions of the body or part thereof, regardless of any pre-existing disability of the injured member or impaired body function.

The following injuries are not classified as permanent partial disability:

1. Loss of fingernails or toenails;
2. Loss of tip of finger without bone involvement;
3. Loss of teeth;
4. Disfigurement;
5. Strains or sprains which do not cause permanent limitation of motion; and
6. Simple fractures to the fingers and toes; also such other fractures as do not result in permanent impairment or the restriction of normal function of the injured member.

Temporary Total Disability is any injury which does not result in death or permanent impairment, but which renders the injured person unable to perform his regular employment activity on any one or more days subsequent to the date of injury.

Medical Treatment Injury is an injury which does not result in death, permanent impairment, or temporary total disability, but which requires medical treatment (including first aid).

Negligible Injury is an injury which raises no doubt in the mind of either the person injured or a witnessing officer as to its lacking the severity to require medical treatment, and includes the following:

1. Bruises, bumps and slight blows;
2. Contusions;
3. Slight sprains and strains; and
4. "Shaking up" (minor shock).

In order to break down injury classifications from the general to the more specific injuries sustained, recourse was had to a schedule of payment allowances of a medical-surgical payment plan. The costs and periods of hospital care assigned to different injury treatments were taken as indicative of the relative severity of each condition.

As to property damage accidents, the amount of damage in terms of estimated monetary cost of repairs provides a convenient scale. At the point at which the property damage grouping merges with that of injury, it was decided that the upper limit of property damage should coincide with a lower level of personal injury. While it is difficult to accept any level of property damage severity as equivalent to even the slightest personal injury, at the level of damage where a vehicle is totally or almost totally demolished it seems reasonable to assume that the lack of injury in such a case is due almost entirely to chance.

The complete scale evolved is given below:

CLASSIFICATION		VALUE
Death		1000
Permanent Total Disability		800 - 975
Loss both eyes		975
Loss one eye and one hand, arm, leg, or foot		900 - 975
Loss two hands, arms, feet, legs		800 - 900
Permanent Partial Disability		600 - 800
Loss one eye		775
Loss one arm above elbow		750
Loss one leg above knee		750
Loss one arm below elbow		700
Loss one leg below knee		650
Loss one foot, hand		625
Strains, sprains, fractures with permanent impairment		600 - 625
Temporary Total Disability		400 - 600
Fractures: compound		550 - 600
simple		500 - 550
Rank:	Thigh	
	Lower leg	
	Forearm and upper arm	
	Vertebra	
	Neck	
	Lower jaw	
	Pelvis	
	Collarbone	
	Finger, toe	
	Nose	
Dislocations		450 - 500
Rank:	Hip	
	Shoulder	
	Ankle	
	Jaw	
Cuts, shock, etc., within this classification		400 - 450
Medical Treatment Injury		300 - 400
Cuts, bruises, shock, blows examined and diagnosed by doctor		
Negligible Injury		200 - 300
Slight bumps, bruises ignored by injured		
Property Damage $2,000 to $3,000		200 - 300
Property Damage $10 to $2,000		1 - 200

BASIC DATA AND METHOD

The records used in this report were accident records of the Port of New York Authority for the Lincoln Tunnel, covering the three years 1953, 1954 and 1955.

Each record was reviewed individually, a numerical value assigned to it consistent with the severity scale previously described, and the following additional data were taken: hour of day, day of week, month, light condition (daylight, semidark, dark—street lights on, artificial—in tunnel), weather (clear, overcast, fog, rain, snow or sleet), and road condition (dry, wet, snowy, icy).

It is believed that inaccuracy in the original data used is as close to a minimum as is possible in accident reporting. The nature of a facility such as the Lincoln Tunnel does not permit parties to an accident to escape notice by a police officer. Any incident, however minor, which causes a driver to stop his vehicle will bring an officer to investigate.

The accuracy of estimating damage incurred can be questioned, and is potentially the most inaccurate part of the reporting procedure. Comparison, however, of original estimated damage to Port Authority vehicles involved in accidents, with the actual cost of repair, has shown a reasonable level of accuracy.

The reporting of personal injury may also be questioned. However, the arrangements at Port Authority facilities are such that only those injured persons who specifically refuse to be examined by a physician do not have their injuries diagnosed professionally.

It was decided that the severity of an accident is a function of the speed of impact, angle of collision, and type of accident. The number of persons in a vehicle, or the number of vehicles involved, serve only to distort the true level of severity of that specific type of accident. For this reason the severity value assigned to an accident is that of the greatest damage done to a single vehicle or the highest degree of injury to a single person.

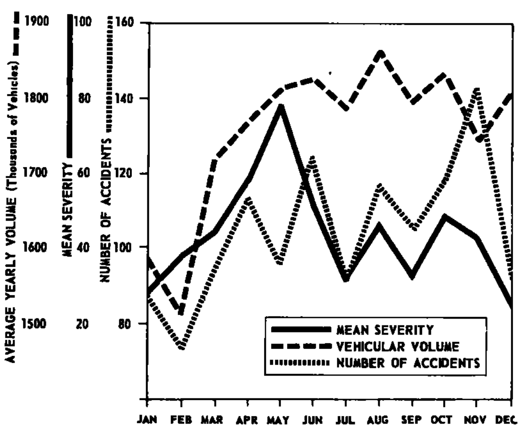

Figure 1. Severity, volume and accident frequency by month.

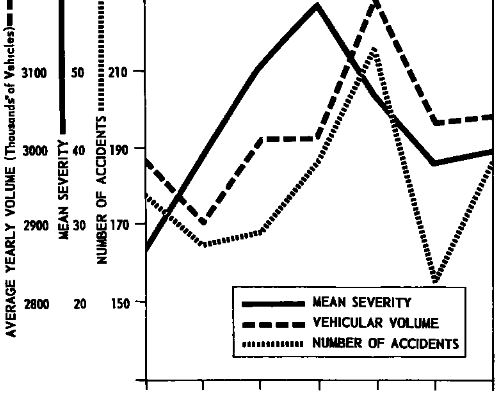

Figure 2. Severity, volume and accident frequency by day of week.

The final decision made was in selecting a representative severity value. Because of the equal weight given each severity value for accidents occurring in an hour, day or month, it was considered reasonable to use the arithmetic mean of all values. The mean severity is the index used in this study.

APPLICATIONS

The applications of the scale which were chosen as illustrations are comparisons of mean severity with vehicular volume, accident frequency, and accident rate by month, day, and hour. The hazard rating of accident severity is then compared with that of accident rate for different light, road and weather conditions.

Monthly Variation

In order to arrive at as accurate a monthly, or seasonal, pattern of accident severity as possible, severity values for the three years studied were combined. These values are plotted in Figure 1 together with number of accidents and vehicular volume. Since it was found that accident rate parallels the movements of number of accidents for this particular sample, it is not plotted here.

Figure 1 shows severity to have its peak in the spring, while the greatest number of accidents occurs in the winter

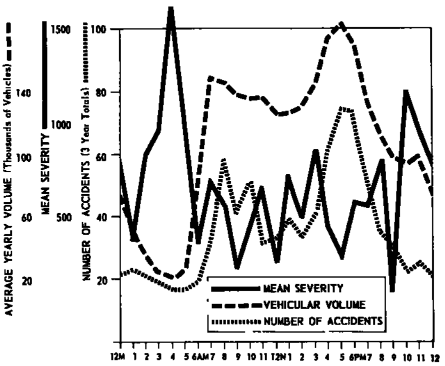

Figure 3. Severity, volume and accident frequency by hour of day.

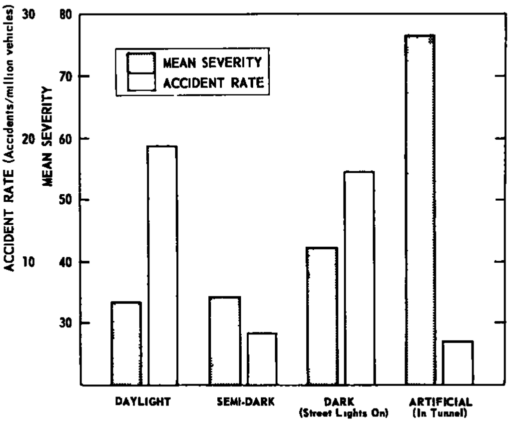

Figure 4. Severity and accident rate by light condition.

months. In this case severity follows the rise in volume from February to April, while accident frequency falls off from March to April. The rise in frequency in October and November during the falling off in volume is not reflected by severity.

Daily Variation

Again, values for the three years were combined, and are shown in Figure 2 compared to volume and accident frequency. Accident frequency follows volume, rising to a peak on Friday, while severity has its peak on Thursday, falling with the additional volume occurring on Friday.

Hourly Variation

Figure 3 compares hourly severity values with volume and accident rate. Severity has its peak during the early morning hours, reflecting the rise in accident rate. Number of accidents is low during this period, following the pattern of volume.

Between 5 a.m. and 3 p.m., severity fluctuates in a manner similar to accident frequency. During the 3 p.m. to 7 p.m. peak volume period, however, accident frequency and accident rate follow this rise while severity falls to a low point.

Variation of Severity with Light Condition

Figure 4 shows the rise in the severity level from daylight through artificial (in tunnel) light conditions. Accident rate, however, indicates semidark, dark and artificial as less hazardous than daylight.

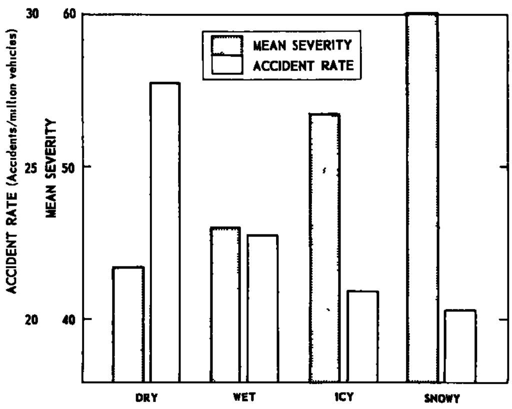

Figure 5. Severity and accident rate by road condition.

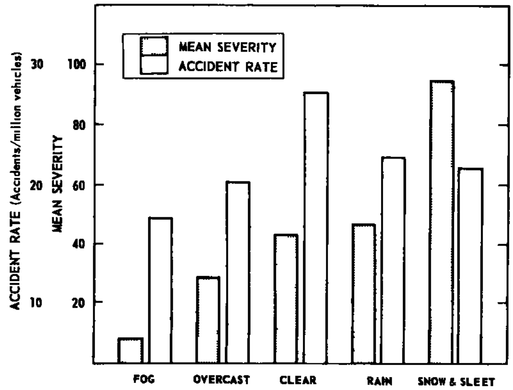

Figure 6. Severity and accident rate by weather condition.

in fact, it gives the in-tunnel condition the best rating, while on the basis of severity it has the worst.

Variations with Road Conditions

Figure 5 gives the rise in severity from dry to wet to icy to snowy. Accident rate gives a completely opposite hazard rating, rising from snowy to icy to wet to dry.

Variation with Weather Condition

Figure 6 has severity rising from fog to overcast to clear to rain to snow or sleet. Accident rate gives clear as most hazardous, followed by rain, snow or sleet, overcast, and fog, in that order.

CONCLUSIONS

For the period and facility considered in this paper, the following conclusions may be drawn:

1. The monthly pattern of severity does not reflect that of accident frequency. The peak in severity occurs in the spring months; that of frequency in the winter months.
2. Severity rises to a peak on Thursdays, falling toward Sunday. The accident rate peak occurs on Fridays.
3. The hourly pattern of severity indicates a peak in the early morning hours 2 a.m. to 6 a.m. Severity follows the movement of volume and accident rate thereafter, until the evening volume and number of accidents peak period, 3 p.m. to 8 p.m. During this time severity shows a depression, falling from 3 p.m. until 5 p.m. and rising until 8 p.m.
4. Severity of accidents increases with decrease of natural light until its maximum, within the tunnel. Accident rate for these same conditions ascribes greater hazard to daylight.
5. Severity increases with poor road conditions, snowy being rated most hazardous. Accident rate gives its lowest hazard rating to snowy, increasing to a maximum for dry.
6. Severity increases with bad weather. As to possibility of occurrence, however, accident rate shows clear weather to be most hazardous.

RECOMMENDATIONS

No doubt there is a great deal of room for improvement in the details of the method outlined here, but a perfected index combining severity and frequency should be of use in directing engineering and enforcement activities to those times and locations which merit priority of attention.

The combining of these two variables seems a logical next step, with possible application of statistical quality-control methods.

An Analysis of One-Car Accidents

RICHARD W. BLETZACKER, Research Associate; and
THOMAS G. BRITTENHAM, Research Associate, Engineering Experiment Station, Ohio State University

Subsequent to a study concerning causation factors in one-car accidents an approach has been developed which proposes a general mathematical expression of the interrelationship of the three major factors involved in every accident, namely, the driver, the highway and the vehicle. The methodology of data collection developed and utilized in the previous study is suggested as a means of amassing sufficient data to permit the correlations necessary to evaluate the constants and express the variables in the proposed equation.

If experimental data verify the proposed concept, a means of predicting accident probability would be at hand. Testing methods and devices could then be developed to predict accident probability for individual drivers. The mathematical expression would also be valuable for estimating the potential effect of highway improvements on accident probability.

A method of classifying driver characteristics, which are factors of accident causation, to enable an intelligent estimate of the cost and effect of correction programs is proposed. The ultimate objective of any comprehensive research in highway safety should be directed toward developing the requisite knowledge to accomplish the maximum effect with the funds available for any recommended corrective program.

The potential of a study based on the concept and methodology proposed here justifies the recommendation that such a project should be inaugurated and pursued to the ultimate end.

● THE CIVIL ENGINEERING Research Group of the Engineering Experiment Station at Ohio State University has been engaged in studies in the field of one-car accidents during the past three years. The data collection and initial analyses were sponsored by the Ohio Department of Highway Safety. The presentation of data and the discussions of findings of previous analyses were contained in reports (1, 2, 5) published by the Engineering Experiment Station and theses (3, 4) by staff members of the Station. A summary of the final report (5) was published as a non-technical version entitled "Highways or Dieways?", Circular 59 (6) of the O. S. U. Engineering Experiment Station.

The present study is sponsored by a supplemental grant from the Station. Robert F. Baker, Associate Professor of Civil Engineering, is project supervisor and Emmett H. Karrer, Professor of Highway Engineering, is consultant. Richard W. Bletzacker, Research Associate is the principal investigator, and Thomas G. Brittenham, Research Associate, is the sociological consultant.

BACKGROUND

The basic philosophy of the study is that all one-car accidents are the result of the failure of the driver to exercise proper judgment. The factors which affect the driver's ability to exercise proper judgment are the combined characteristics of the driver, the highway, and the vehicle. The accidents caused by "acts of God" would necessarily be excluded from consideration, because they could be considered unavoidable. These accidents would include such conditions as: trees blown down immediately in front of or upon a moving vehicle; broken steering mechanism or faulty brakes on a new vehicle; and pavement "blow-ups" (rigid pavement expansion failures) that occur simultaneously with the approach of the vehicle.

If the basic concept can be accepted, the following corollaries may be developed: (1) the perfect driver on the perfect highway will not have an accident; (2) the perfect driver on the imperfect highway will not have an accident; (3) the imperfect driver on the perfect highway may have an accident; and (4) the imperfect driver on the imperfect highway may have an accident. The term perfect driver as used here is defined as a driver who is exercising the proper judgment required to cope with any driving situation. The term imperfect driver is defined as a driver who at a given instant fails to exercise the proper judgment required to cope with the driving situation. The perfect highway is defined as a roadway which does not present any unsafe characteristics with which drivers must cope. The imperfect highway is a roadway which does present unsafe characteristics.

The basic concept and the four corrollaries, as applied to one-car accidents, may be expressed in the general mathematical expression:

$$a = x^{y^{-n}} \quad (1)$$

in which

a = driver judgment
x = evaluation of driver characteristics
y = evaluation of highway and vehicle characteristics
n = a constant that reflects the relative importance of the highway and vehicle.

Driver's judgment, as used here, is defined as that collection of driver actions which are the result of his evaluation of proper driving procedures required to cope with any driving situation. The quantitative evaluations of x and y must result in values between the limits of zero and one for the mechanics of the equation to follow the proposed concept.

OBJECTIVES

The objective of the study is to develop further a method of evaluating the interrelationship of the major variables involved in highway accidents, namely, the driver, the highway, and the vehicle. It is further proposed that with this technique an index of accident probability of an individual driver could be developed.

PROCEDURES

Eq. 1 is an attempt to express the interrelationship of the three major variables and their combined effect on driver judgment. The exact form of the expression and the evaluation of the constant n must be developed from and based on experimental data.

The data available from the previous studies included 183 one-car accidents, for which a driver interview was conducted, as well as an investigation of the highway characteristics at the accident location. The previous study was not specifically designed to yield the information necessary to make the evaluation proposed herein and, therefore, a rather subjective study of the data and subsequent evaluation were necessary. In order to illustrate the methodology of developing the interrelationship of the major variables, an analysis of each of the 183 one-car accidents was made by evaluating the driver characteristics and the highway characteristics.

The driver interview schedule contained over 80 questions, but, for the purpose of this study, only 27 were considered pertinent. Of interest here are only those factors which effect a negative or subtractive influence on the driver's ability to cope with the driving situation faced at the time of the accident. The data shown in Table 1 were classified as physiological, psychological, trip data, driver training and experience, and driver action, opinion, and attitude. Although the opinion and attitude information could not be shown to have contributed directly to the accident it was particulary useful and often significant as corroborating material.

In order to assess the influence of each negative driver characteristic, the following qualitative terms were used: (1) primary factor, (2) secondary factor, and (3) tertiary

TABLE 1

CLASSIFICATION OF NEGATIVE DRIVER CHARACTERISTICS

I. Physical Characteristics
   A. Age
   B. Amount of sleep prior to trip
   C. Alertness on trip
   D. How driver felt on trip
   E. Amount of alcohol consumed on trip
   F. Illness or injuries (recent)

II. Psychological Characteristics
   A. Anxiety on trip
   B. Personality type

III. Trip Data
   A. Familiarity with the highway
   B. Number of close calls encountered on trip

IV. Driver Training and Experience
   A. Method of learning to drive
   B. Driving examination information
   C. Length of experience with driving license
   D. Average mileage driven
   E. Number of previous accidents
   F. Number of previous arrests

V. Driver Action
   A. Distractions within vehicle
   B. Distractions outside of vehicle
   C. Average speed on trip
   D. Speed at time of accident
   E. Reason for arrest (this accident)

VI. Driver's Opinion
   A. Judgment as to speed at time of accident
   B. Driver's opinion of cause of accident
   C. Amount of alcohol that can be consumed without affecting driving
   D. Driver's opinion of his ability as a driver
   E. Driver's opinion of his need for improvement
   F. Attitude toward driving as measured by Siebrecht Test

factor. A primary factor was defined as any driver condition, situation, or behavior which directly contributed to the cause of the accident, that is, without the presence of this (these) factor(s), the accident could not have occurred. A secondary factor was any driver condition, situation, or behavior which indirectly contributed to the cause of the accident. These factors were considered to be conducive to creating a hazardous or unsafe driving situation. A tertiary factor was described as a driver condition, situation or behavior which constituted or reflected an undesirable driving situation but no direct relationship could be shown between it and the accident.

Each of the 183 one-car accidents was reviewed and the evidence of each negative driver characteristic was evaluated as to its effect, that is, primary, secondary, or tertiary. The degree of the effect of a particular factor was not constant; for example, the fact that a driver had two beers might be a tertiary factor in some cases yet might be evaluated as a secondary factor in another instance. Each primary factor was arbitrarily assigned a numerical value of 1.0, each secondary factor a value of 0.33, and each tertiary factor a value of 0.1.

The numerical values in each accident were summed, and converted to values of x by the equation:

$$x = e^{-1.8z} \tag{2}$$

in which

e = base of natural logarithms (2.71828)

z = sum of negative driver characteristic factors

Eq. 2 was developed to convert the value of z to values of x which followed the tenets of the proposed concept.

Obviously, had some other method of evaluating the driver characteristics been used which would have yielded numerical values of x between zero and one, Eq. 2 would not have been necessary. However, the method used here seemed to have certain advantages and permitted a reasonable degree of objectivity in making the required evaluation.

The report of the highway conditions at each accident location was studied and an evaluation of the highway characteristics was made. The evaluation was based upon the geometric standards of the AASHO (7) for design speeds of 60 mph, lane widths of 12 ft, and shoulder widths of 7 ft. The condition of the highway from the standpoint of maintenance was also considered, as were adequate warning and advisory signing.

Table 2 lists the highway characteristics under the two major classifications of design geometrics and maintenance. Each major classification was given equal weights with a potential value of 100. The minor classifications and their subdivisions were subjectively assigned weighted values, as indicated after each characteristic. The

TABLE 2

CLASSIFICATION OF ROADWAY CHARACTERISTICS

I. Geometrics - Design			100
A. Alignment		40	
1. Vertical	10		
2. Horizontal	30		
a. Safe speed			
b. Passing sight distance			
c. Obstructions			
d. Distractions			
B. Pavement Width		20	
C. Shoulder Width		10	
D. Change in Geometric Conditions		30	
II. Maintenance			100
A. Road Surface Conditions		30	
1. Pavement type	10		
2. Rideability	20		
B. Road Friction		20	
1. Surface roughness	5		
2. AASHO brake distance	10		
3. Surface condition	5		
C. Shoulder Conditions		20	
1. Relative elevation	10		
2. Surface type	5		
3. Surface irregularity	5		
D. Warning Conditions		30	
1. Sign adequacy	15		
a. Size			
b. Condition			
c. Type			
d. Obstructions			
2. Center line marking adequacy	15		
a. Condition			

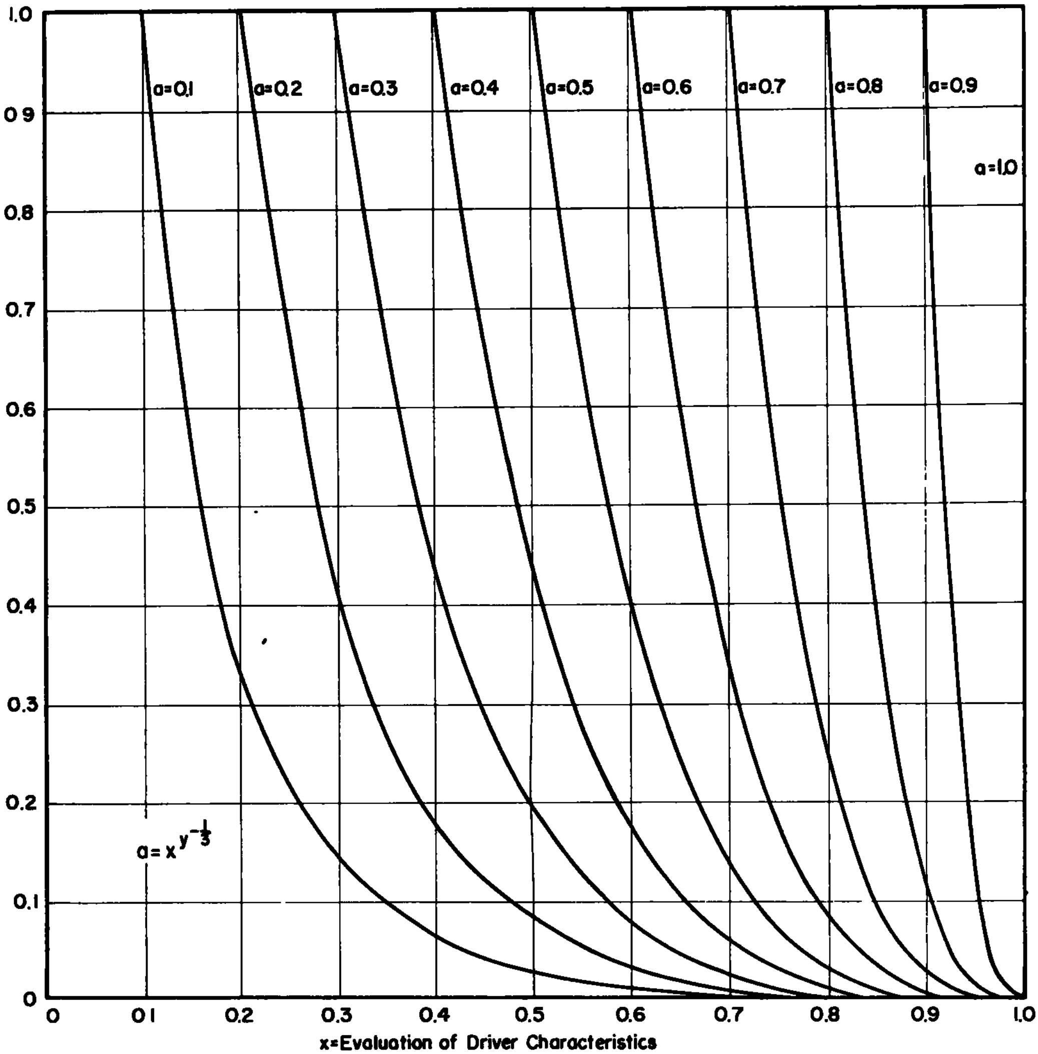

Figure 1. Family of curves of driver ability.

evaluation of the design standards and the maintenance conditions for each accident location were combined by a geometric rather than a straight arithmetic mean, and then divided by 100 to reduce the value within the limits of zero to one. The geometric mean was used because of the consideration that a well designed road poorly maintained, or a poorly designed road well maintained was not equivalent to some average road with average maintenance.

The previous study did not provide any significant data on vehicle characteristics, and no attempt was made to evaluate this variable in the present analysis. No inference should be made, however, that the vehicle influence would not enter into future studies.

The accomplished evaluations of driver and highway characteristics indicate that there were various factors in each characteristic which combined in such a way as to cause an accident. The inference is that some drivers were exercising better judgment than others and that accidents occurred on highways with evaluations encompassing the whole range from safe to unsafe. Data are needed which will permit analyses yielding quantitative values of judgment. To fit an equation to the data would then be relatively

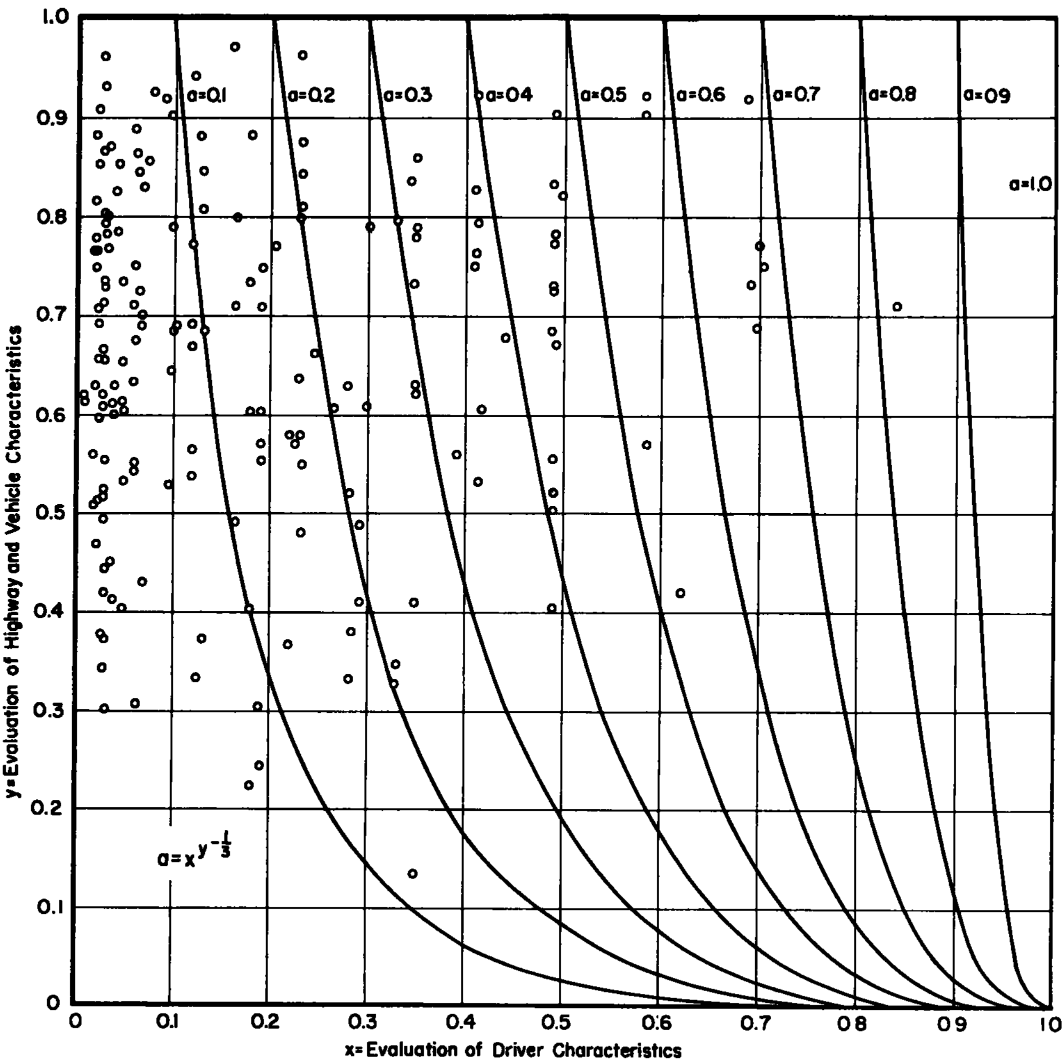

Figure 2. Plot of 183 one-car accidents.

simple and, specifically, it is proposed that the equation would follow the general form of Eq. 1.

Assuming a value of n, and for various values of a, the relationship between x and y is illustrated in Figure 1. By definition, the perfect driver is depicted when a = 1, which can only be valid when x = 1. In this instance, y has no influence. If the value of driver judgment is thought of as similar to topographic elevations, then the values of a = 0.1, a = 0.2, etc., are actually contours joining points of equal elevation or in this case equal driver judgment.

The values of x and y in each of the 183 one-car accidents were plotted, as shown in Figure 2, on the same graph containing the family of curves illustrated in Figure 1. There is no intention to infer that this evaluation would result, necessarily, in n being equal to ⅓. These curves were merely used to illustrate the principle.

DISCUSSION OF METHODOLOGY

The proposed concept and methodology appears to have merit in the analysis of a number of problems encountered in highway safety. The utilization of Eq. 1 is dependent on establishing a means of properly evaluating x and y in such a way as to yield some level of driver judgment demonstrated in each driving situation. Data could be collected on a number of driving situations in which some of the factors could be held constant, in some cases resulting in accidents and the rest in non-accidents. An example is a male driver who, on a clear day, failed to negotiate a sharp curve at 45 mph. An interview produced data on factors which influenced this driver's judgment. If a group were stationed at this location and attempted to match the apparent factors found in the accident situation, data might be gleaned which would point up the influence of the unsimilar factors. With sufficient data of this type collected, correlations could be performed which should produce significant results. Obviously, this procedure would entail a rather comprehensive and long-range project but there is good reason to believe that the cost would be justified.

In analyzing data collected on accident and non-accident driving situations, it is reasonable to expect that the accident situations would plot in the lower ranges of a values while the non-accident situations would plot in the higher ranges. Of interest is the fact that, in Figure 2, assuming Eq. 1 to be valid in the form indicated, 96.7 percent of the accidents occurred when the driver's judgment was less than 0.6, 94.5 percent when a was less than 0.5, and 87.5 percent when a was less than 0.4.

For the one-car accident case there must be a curve which would delineate between that value of driver judgment which will result in an accident and that which will be sufficient to avoid an accident. This curve will probably lie somewhere in the middle range of a. Such delineation may actually be a band of some width but this situation will result from an inaccurate evaluation.

It is also reasonable to expect that the necessary data on multi-car accidents could be analyzed by the concept proposed here. There would not, however, be a clear delineation between the value of a which would result in and the one which would be free of an accident. The theory of probability could be utilized to predict how many times a near-accident situation would have to be presented to a driver capable of a certain level of judgment before an accident might occur. This technique would permit the development of testing devices and procedures which would evaluate an individual and yield a determination of his accident potential.

The shape of the curves produced by Eq. 1 is of interest to the highway engineer. This concept could permit estimates of the highway improvements on driver judgment. Assume $n = \frac{1}{10}$ and the range of questionable driver judgment were between $a = 0.5$ and $a = 0.6$. For drivers who had driver characteristics (x values) of 0.6, a would equal 0.5 when $y = 0.046$, whereas, y would equal 1.0 when $a = 0.6$. This situation indicates that a vast change in the combined highway and vehicle characteristics would have to be made to improve driver judgment by only ten percent. If $n = 1.0$ is assumed and the other conditions constant, the same improvement of a would be effected by changing y from 0.735 to 1.0. Such a change might be economically feasible in this instance. This principle is shown by comparing the families of curves illustrated in Figures 3 and 4.

A concept accepted and used in the field of instrument surveying to classify the inaccuracies of measurements (namely, errors, mistakes, and blunders) could be used to classify the detractive characteristics of the driver which restrict the driver's ability to exercise proper judgment. Driver error is defined as the static physical, psychological, or mental characteristics of a human which inhibit the judgment required to correctly cope with a driving situation. Driver mistake constitutes the failure of a human to have knowledge of the laws, rules, principles, and procedures required to cope with a driving situation. Driver blunder involves the transitory physical, psychological, or mental characteristics of a human which inhibit the judgment required to cope correctly with a driving situation.

Driver error reduces any human from perfection to some idealized level which he can maintain "most of the time." Those officials responsible for granting driving licenses must decide the minimum level which can be tolerated. Having established this level, the highways and vehicles must be designed to permit the minimum driver to operate a vehicle without undue jeopardy.

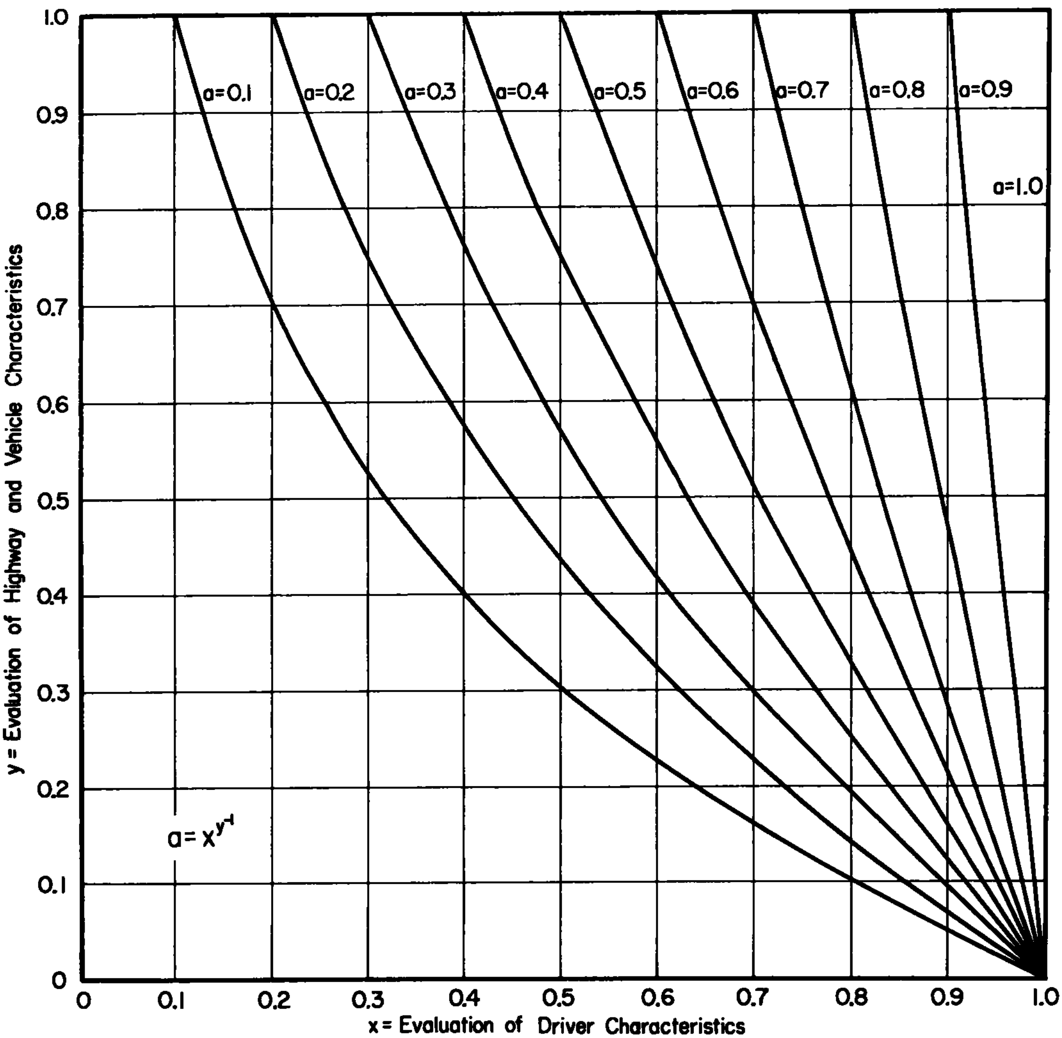

Figure 3. Family of curves of driver ability (n=1.0).

Driver error is the primary subtractive factor in evaluating a human's maximum capacity as a driver. Certain characteristics are capable of correction, such as faulty eyesight by use of glasses, but, in the main, driving error must be compensated for since correction is not generally possible. The compensation would take the form of driving-habit adjustments or limitations when or where a driver may operate a vehicle.

Driver mistake is the second subtractive factor in evaluating a human's maximum capcity as a driver. Obviously, an individual must possess a minimum knowledge to obtain a license to drive. However, this minimum is insufficient to permit a driver to cope with every driving situation. The driver who does not know, for instance, how to correct for a skid on a slippery pavement, or how to resist centrifugal force when he finds he is exceeding the safe speed around a curve has little chance to cope with these situations. The knowledge therefore is no assurance that he will possess the required skill to handle the situation, but it necessarily precedes skill.

The third subtractive factor in evaluating driver characteristics, driver blunder, is of two types: (1) periodic lapses into thoughtless or reckless actions caused by

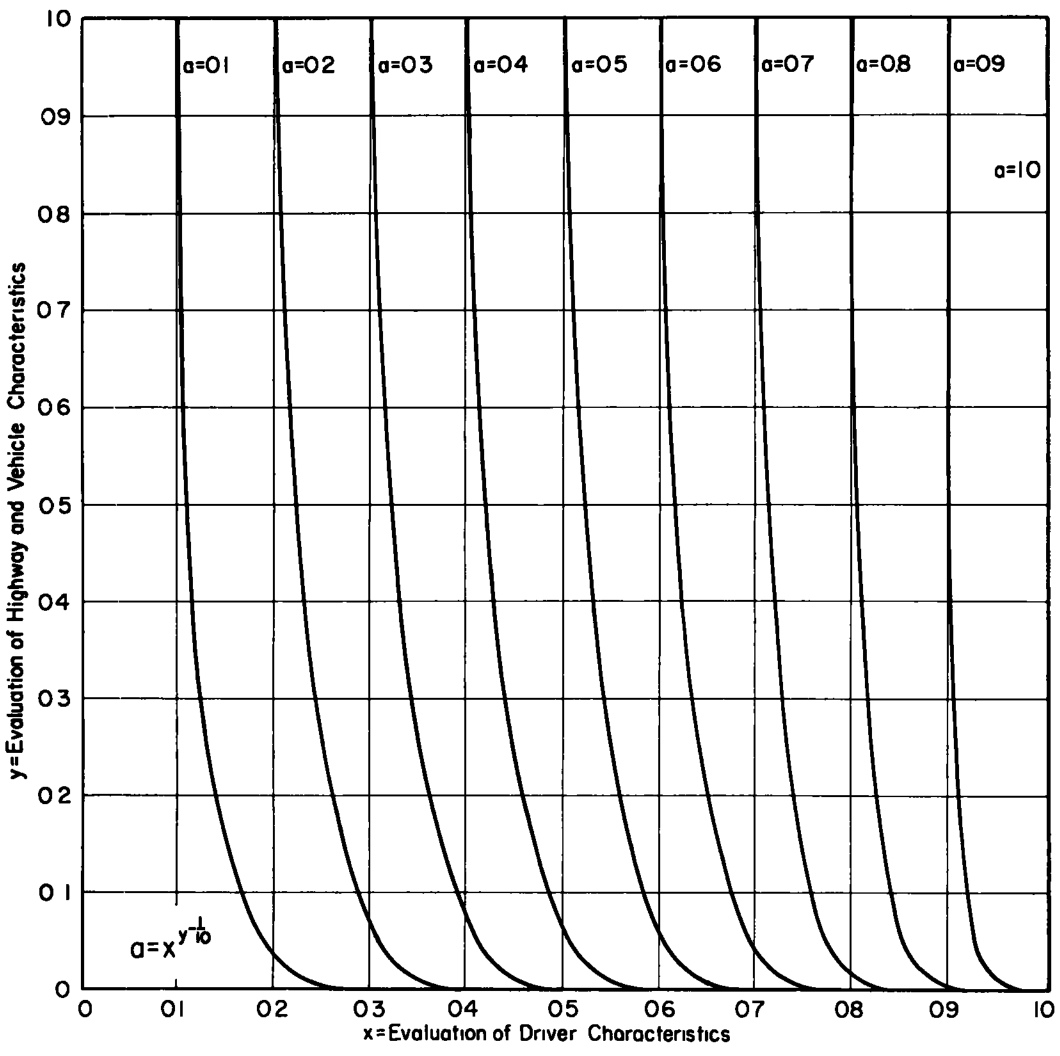

Figure 4. Family of curves of driver ability (n=0.1).

transitory characteristics, and (2) regular disregard of the laws, rules, or principles of safe driving. Correction of the first type might be accomplished to a degree by properly motivating the drivers. However, a certain amount of mental lapse is inherent in any human. The second type presents a problem for the enforcement agency and the courts.

Accident data classified as suggested here might permit insight into the most effective methods of implementing a highway safety program. Such a program usually includes education, enforcement, driver examination, and training. The ideal solution is to know which area to emphasize and to what degree.

CONCLUSIONS AND RECOMMENDATIONS

In the considered opinion of the authors, the proposed concept is a valuable base upon which to institute significant research in the field of highway safety. The methodology, incoporating a confidential personal interview with drivers and an investigation of highway conditions at accident sites, is a fruitful means of data collection. Data should be obtained on non-accident and near-accident situations as well as upon actual accidents. The proposed study should be confined to the one-car accident in the initial phases, but in any expansion multi-car accidents should be included.

The solution of the problem or, more realistically, the reduction of highway accidents lies in the efficient utilization of available funds. It would be foolhardy, for example, to expend billions of dollars on highway construction solely for the purpose of reducing accidents before knowing how fruitful the expending of comparable funds for driver training and education would be. It is essential, therefore, to conduct studies in accident causation directed toward a means of evaluating the interrelationship of the driver, the highway, and the vehicle so as to provide the most effective methods of accident reduction.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to certain individuals whose assistance was invaluable in developing the proposed concept. The supervisor of the study, Robert F. Baker, Associate Professor of Civil Engineering, guided the development of the basic philosophy and the interpretations of the proposed analyses. Emmett H. Karrer, Professor of Highway Engineering, co-supervisor of the previous studies, also contributed guidance in this study.

Dr. E. Q. Moulton, Professor of Civil Engineering, suggested ideas which led to development of the mathematical expression. Robert S. Green, Executive Director of the Engineering Experiment Station, implemented the financial support for this analysis.

Special mention should be made of the staff of the Civil Engineering Research Group for its aid in tabulating and computing data, and reproducing this paper.

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Predicting Traffic Accidents from Roadway Elements on Urban Extensions of State Highways

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The investigation described in this report represents research by the Oregon State Highway Department to develop equations which can be used to predict accidents on the urban extensions of the State Highway System from roadway elements such as ADT, commercial and residential units and driveways, intersections, signalized intersections, indicated speed, pavement width, effective lane width, and the number of lanes.

The study utilized a sample of 426 sections with a total length of 186.4 mi. Data were analyzed by subgrouping the sections by number of traffic lanes and ADT groupings. Within these groups additional subgroups of urban extensions were studied for suburban, corporate, business, residential and mixed culture sections. The analysis used multiple correlation techniques with the end result of the analysis being a series of equations which indicated the relationships of the various roadway elements to accident rates on urban and subgroups of urban extensions of highways in Oregon.

The more important conclusions which can be drawn from the study are as follows:

1. Motor vehicle accident rates are related to certain physical features of urban extensions of the highway system. This relationship is strong enough in the higher ADT ranges to make it possible to predict accident rates with a reasonable degree of accuracy on the basis of known physical features.
2. Accident rates on low volume roads do not have a strong relationship with any roadway feature.
3. Motor vehicle accident rates increase when: (a) The number of commercial units adjacent to the section increases. (b) The number of traffic signals increases. (c) The number of intersections increases. (d) The indicated speed decreases. (e) The average daily traffic increases. (f) The pavement width increases.

● THE CHANGES in recent years of highway construction to higher design standards requiring many miles of expressways and freeways have emphasized the deficiency of benefit studies used as an aid in selecting one of two or more alternate routes between common termini. Present methods of analysis provide a means for converting to monetary values benefits derived through savings in time, distances traveled, and other operating costs. However, no measure has been used for the increased safety from reduction in accidents resulting from access control and elimination of crossings at grade. Proper evaluation of benefits from expressways and freeways should include the benefits derived from the reduction in motor vehicle accidents.

The first step necessary to convert the reduction in accidents to a monetary value is a reasonable estimate of the number of accidents which will occur on both the proposed improvement and the old obsolete highway. This study was undertaken to determine if some means could be developed for predicting the number of accidents which could be expected on a given section of urban or suburban section of highway from information readily available.

A previous study, "Predicting Traffic Accidents from Roadway Elements - Rural Two-Lane Highways with Gravel Shoulders,"[1] developed a procedure for predicting accidents with reasonable accuracy for rural 2-lane highways with gravel shoulders. The characteristics of urban extensions of the state highway system from the standpoint of both design and roadside culture adjacent to the highways are quite different from rural highways. It was, therefore, felt that the formulas developed for rural highways could not be applied to urban sections. For this reason, a separate study was conducted for the urban extensions of the highway system.

In the use of accident prediction formulas for determination of motor vehicle user benefits, it is desirable to limit the roadway elements used in the prediction to those which can be readily determined. To this end the field data were confined to those that were readily available. Roadway elements which were considered, and for which data were obtained in the field, include the number of commercial units, commercial driveways, residential units, residential driveways, intersections, channelized intersections and traffic signals, the indicated speed, the pavement width, width of each shoulder, and number of traffic lanes.

Field data were obtained for 466 urban sections of highways. Of these, 426 sections permitted parallel parking on both sides, and the remaining 40 sections were distributed among those having no parking, angle parking, and various combinations of parking. To provide the maximum uniformity of sections, the study was confined to the 426 sections permitting parallel parking. The remaining 40 sections did not provide a large enough sample for a separate analysis. Because of the variable nature of the roadside culture and roadway elements on the urban sections of highways, it was not feasible to select sections of uniform length for this study. The sections varied in length from those as short as $\frac{1}{10}$ mile up to sections 2.1 miles long. The 426 sections had a total length of 186.4 miles or an average length of 0.4 mile per section.

The data were analyzed on several different breakdowns. The anticipated accident experience on 2- and 4-lane highways should be quite different. It was, therefore, considered necessary to initially consider the urban sections separately for 2- and 4-lane highways. The previous study in rural areas showed that the most reliable prediction equations were developed with a breakdown of sections into various traffic volume groupings. This then became the second major grouping in this analysis.

Prediction equations were developed for the urban sections based on the foregoing subgrouping. To further increase the accuracy of the prediction equations, the sections were subdivided between corporate and suburban sections. During the collection of the field data each section was classified by its cultural composition as residential, business, or mixed. These classifications were used for an additional subgrouping of the urban sections.

The data were analyzed by statistical methods to determine the quantitative relationship between accidents and the various roadway elements. The prediction equations were developed from multiple regression techniques. It is possible that more refined data may greatly improve the accuracy of prediction equations, however, such refinement was not deemed desirable for this particular study. The data used in the study were based on an average traffic volume and accident data for 1954 and 1955. Because of the variable length of the sections, it was not desirable to use number of accidents for this analysis, therefore, they were converted to accidents per million vehicle-miles. All regression equations provide answers in terms of accidents per million vehicle-miles.

DATA SOURCES

Field Work

The field data were obtained on state primary and secondary urban highways, in-

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[1] Bulletin 158, Highway Research Board

cluding 2- and 4-lane sections. Any section which had new construction since 1954, the first year for which accident data were used in the analysis, was eliminated.

The field examiners recorded the following information with the use of a multiple bank counter:

1. Commercial Units (CU)
2. Commercial Driveways (CDW)
3. Residential Units (RU)
4. Residential Driveways (RDW)
5. Intersections (INT)
6. Traffic Signals (SIG)
7. Channelized Intersections (CI)

All of the above data were converted to rates (that is, driveways per mile). In addition to the above rate per mile data, the following information was also obtained in the field:

1. Indicated Speed (SP)
2. Pavement Width (PA)
3. Shoulder Width (SH)
4. Number of Lanes
5. Median Width
6. Effective Lane Width (ELA)

The sections may have had curbs, gravel shoulders, or paved shoulders; however, no distinction was made in this regard except to take these factors into account in the calculation of effective lane widths. Finally, space was provided for recording codes for urban, suburban, and culture type—that is, business, residential, or mixed, and a parking code.

A detailed description of the field procedure, along with a sample field sheet appears in Appendix A.

Traffic Volume Data

The average daily traffic (ADT) was taken from the Traffic Volume Tables for 1954 and 1955.[2] The average for the 2 years was used in the study. As indicated earlier, traffic accident predictions are better if the data are grouped by ADT ranges. For this study arbitrary ADT ranges were used as follows: on 2-lane sections 5,000 and under, 5,000 - 9,999, and 10,000 and over. The corresponding ranges for 4-lane sections were less than 9, 000, 9,000 - 17,999, and 18,000 and over.

Although it is probable that the volume of traffic on cross streets and driveways might be closely related to the accident experience on corporate and suburban sections, complete information on these traffic volumes was not available and therefore could not be included in the report.

Accident Data

The accident data used in this study were available in the Accident Analysis Section of the Traffic Engineering Division, Oregon State Highway Department. The accident records for the years 1954 and 1955 were employed.[3]

The lack of uniform section lengths did not permit direct comparison of the various sections based on the total number of accidents on each section. It was therefore necessary to adjust each section for this difference in section length. Because of the relatively high relationships between accidents and average daily traffic, it was decided to convert the accidents for each section to accidents per million vehicle-miles, which provided the measure for direct comparison of accident experience on each section.

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[2] Oregon State Highway Department Technical Report No. 55-1 and 56-1, respectively.
[3] Oregon State Highway Department Technical Report No. 55-2 and 56-2, respectively.

ANALYSIS

Previous study on rural sections indicated that considerable advantage could be gained by developing accident prediction equations for subgroups of the study sections. Several arrangements of subgroups were studied to obtain the best results possible. No attempt will be made here to describe the statistical techniques used in the analysis of the data, however, a description of these statistical techniques will be found in Appendix C.

The first step in the analysis was to group the data by 2- and 4-lane sections. Within these major groups subgroups were made for the selected ADT ranges. Additional subgroupings were made by studying separately corporate and suburban sections. Suburban sections were defined as those sections which were outside the coporate limits of the city, but which had an indicated speed posted. Although it was realized that this definition would have some shortcomings inasmuch as the corporate limits do not normally correspond with a change in roadside culture, they did lend themselves to subdividing sections which were independent of the observer's judgment.

A subgrouping of urban sections by roadside culture was also studied. During the

TABLE 1

DISTRIBUTION OF STUDY SECTIONS BY NUMBER OF LANES, ADT RANGES AND CULTURE TYPE

ADT Range	Area of Culture	Number of Sections	Total Length of Sections (mi)	Average Length of Sections (mi)
		2 Lanes		
Under 5,000	Urban	130	45.2	0.3
	Suburban	35	17.6	0.5
	Corporate	95	27.6	0.3
	Business	30	-	-
	Residential	46	-	-
	Mixed	54	-	-
5,000 - 9,999	Urban	140	54.8	0.4
	Suburban	33	18.8	0.6
	Corporate	107	36.0	0.3
	Business	52	-	-
	Residential	33	-	-
	Mixed	55	-	-
10,000 and over	Urban	26	18.6	0.7
	Suburban	2	1.1	0.5
	Corporate	24	17.5	0.7
	Business	11	-	-
	Residential	5	-	-
	Mixed	10	-	-
All	Corporate Portland	22	15.8	0.7
	Corporate Non-Portland	204	66 0	0.3
		4 Lanes		
Under 9,000	Urban	54	16.4	0.3
	Suburban	9	3.7	0.4
	Corporate	45	12.7	0.3
	Business	35	-	-
	Residential	8	-	-
	Mixed	11	-	-
9,000 - 17,999	Urban	50	31.2	0.6
	Suburban	17	12.2	0.7
	Corporate	33	19.0	0.6
	Business	29	-	-
	Residential	3	-	-
	Mixed	18	-	-
18,000 and over	Urban	26	20.0	0.8
	Suburban	5	2.8	0.6
	Corporate	21	17.2	0.8
	Business	20	-	-
	Residential	3	-	-
	Mixed	3	-	-
All	Corporate Portland	32	25.6	0.8
	Corporate Non-Portland	67	22.6	0.3

cluding 2- and 4-lane sections. Any section which had new construction since 1954, the first year for which accident data were used in the analysis, was eliminated.

The field examiners recorded the following information with the use of a multiple bank counter:

1. Commercial Units (CU)
2. Commercial Driveways (CDW)
3. Residential Units (RU)
4. Residential Driveways (RDW)
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5. Median Width
6. Effective Lane Width (ELA)

The sections may have had curbs, gravel shoulders, or paved shoulders; however, no distinction was made in this regard except to take these factors into account in the calculation of effective lane widths. Finally, space was provided for recording codes for urban, suburban, and culture type—that is, business, residential, or mixed, and a parking code.

A detailed description of the field procedure, along with a sample field sheet appears in Appendix A.

Traffic Volume Data

The average daily traffic (ADT) was taken from the Traffic Volume Tables for 1954 and 1955.[2] The average for the 2 years was used in the study. As indicated earlier, traffic accident predictions are better if the data are grouped by ADT ranges. For this study arbitrary ADT ranges were used as follows: on 2-lane sections 5,000 and under, 5,000 - 9,999, and 10,000 and over. The corresponding ranges for 4-lane sections were less than 9,000, 9,000 - 17,999, and 18,000 and over.

Although it is probable that the volume of traffic on cross streets and driveways might be closely related to the accident experience on corporate and suburban sections, complete information on these traffic volumes was not available and therefore could not be included in the report.

Accident Data

The accident data used in this study were available in the Accident Analysis Section of the Traffic Engineering Division, Oregon State Highway Department. The accident records for the years 1954 and 1955 were employed.[3]

The lack of uniform section lengths did not permit direct comparison of the various sections based on the total number of accidents on each section. It was therefore necessary to adjust each section for this difference in section length. Because of the relatively high relationships between accidents and average daily traffic, it was decided to convert the accidents for each section to accidents per million vehicle-miles, which provided the measure for direct comparison of accident experience on each section.

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[2] Oregon State Highway Department Technical Report No. 55-1 and 56-1, respectively.
[3] Oregon State Highway Department Technical Report No. 55-2 and 56-2, respectively.

ANALYSIS

Previous study on rural sections indicated that considerable advantage could be gained by developing accident prediction equations for subgroups of the study sections. Several arrangements of subgroups were studied to obtain the best results possible. No attempt will be made here to describe the statistical techniques used in the analysis of the data, however, a description of these statistical techniques will be found in Appendix C.

The first step in the analysis was to group the data by 2- and 4-lane sections. Within these major groups subgroups were made for the selected ADT ranges. Additional subgroupings were made by studying separately corporate and suburban sections. Suburban sections were defined as those sections which were outside the coporate limits of the city, but which had an indicated speed posted. Although it was realized that this definition would have some shortcomings inasmuch as the corporate limits do not normally correspond with a change in roadside culture, they did lend themselves to subdividing sections which were independent of the observer's judgment.

A subgrouping of urban sections by roadside culture was also studied. During the

TABLE 1

DISTRIBUTION OF STUDY SECTIONS BY NUMBER OF LANES, ADT RANGES AND CULTURE TYPE

ADT Range	Area of Culture	Number of Sections	Total Length of Sections (mi)	Average Length of Sections (mi)
		2 Lanes		
Under 5,000	Urban	130	45.2	0.3
	Suburban	35	17.6	0.5
	Corporate	95	27.6	0.3
	Business	30	-	-
	Residential	46	-	-
	Mixed	54	-	-
5,000 - 9,999	Urban	140	54.8	0.4
	Suburban	33	18.8	0.6
	Corporate	107	36.0	0.3
	Business	52	-	-
	Residential	33	-	-
	Mixed	55	-	-
10,000 and over	Urban	26	18.6	0.7
	Suburban	2	1.1	0.5
	Corporate	24	17.5	0.7
	Business	11	-	-
	Residential	5	-	-
	Mixed	10	-	-
All	Corporate Portland	22	15.8	0.7
	Corporate Non-Portland	204	66 0	0.3
		4 Lanes		
Under 9,000	Urban	54	16.4	0.3
	Suburban	9	3.7	0.4
	Corporate	45	12.7	0.3
	Business	35	-	-
	Residential	8	-	-
	Mixed	11	-	-
9,000 - 17,999	Urban	50	31.2	0.6
	Suburban	17	12.2	0.7
	Corporate	33	19.0	0.6
	Business	29	-	-
	Residential	3	-	-
	Mixed	18	-	-
18,000 and over	Urban	26	20.0	0.8
	Suburban	5	2.8	0.6
	Corporate	21	17.2	0.8
	Business	20	-	-
	Residential	3	-	-
	Mixed	3	-	-
All	Corporate Portland	32	25.6	0.8
	Corporate Non-Portland	67	22.6	0.3

TABLE 2

ZERO ORDER CORRELATIONS BETWEEN ACCIDENT RATES AND ROADWAY ELEMENTS

	Accident Rate - Roadway Elements Correlations									
Study Group	ADT	CU	CDW	RU	RDW	INT	SIG	SP	PA	ELA
2 Lane, Under 5,000 ADT										
Urban	0.03	0.26	0.13	-0.06	-0.14	0.04	0.06	-0.06	0.37	0.22
Suburban	0.06	-0.10	-0.06	-0.11	-0.10	-0.06	[1]	0.32	0.13	0.30
Corporate	0.03	0.33	0.20	-0.08	-0.15	0.01	0.04	-0.08	0.37	0.20
Business	-0.03	0.29	0.10	-0.27	-0.25	-0.06	-0.03	-0.10	0.40	0.27
Residential	0.04	0.19	0.06	0.38	-0.01	-0.03	0.03	-0.13	0.52	0.28
Mixed	0.07	-0.03	-0.08	-0.03	-0.18	0.16	0.28	0.11	0.21	0.00
2 Lane, 5,000 - 9,999 ADT										
Urban	0.33	0.66	0.26	-0.15	-0.18	0.45	0.49	-0.32	0.39	0.19
Suburban[2]										
Corporate	0.32	0.71	0.24	[3]	[3]	0.42	0.48	-0.35	0.36	0.16
2 Lane, 10,000 and over, ADT										
Urban	0.42	0.67	0.44	-0.07	-0.11	0.53	0.85	-0.44	0.37	0.02
Suburban[2]										
Corporate	0.44	0.67	0.46	[3]	[3]	0.51	0.85	-0.41	0.37	0.02
2 Lane, 5,000 and over ADT										
Suburban	0.10	0.15	0.41	0.04	0.05	0.52	[1]	-0.02	0.16	0.28
Business	0.47	0.67	0.13	0.07	0.06	0.55	0.78	-0.39	0.43	0.13
Residential	0.45	0.02	-0.09	0.32	0.15	0.27	0.01	0.08	0.10	-0.02
Mixed	0.49	0.46	0.23	0.33	0.22	0.64	0.27	-0.20	0.42	0.09
2 Lane, All ADT's										
Corporate Portland	0.54	0.68	0.60	[3]	[3]	0.53	0.87	-0.39	0.25	-0.17
Corporate Non-Portland	0.10	0.47	0.21	-0.08	-0.16	0.16	0.27	-0.20	0.37	0.22
4 Lane, Under 9,000 ADT										
Urban	0.05	0.39	0.20	-0.13	0.04	0.13	0.79	-0.11	0.07	-0.18
Suburban[2]										
Corporate	0.14	0.42	0.29	-0.17	0.03	0.11	0.80	-0.06	-0.08	-0.21
Business	0.03	0.35	0.12	0.12	0.28	0.12	0.81	-0.08	0.03	-0.23
Residential & Mixed	0.07	-0.11	0.01	-0.20	0.26	-0.04	0.56	0.20	0.20	-0.05
4 Lane, 9,000 - 17,999 ADT										
Urban	0.27	0.62	0.30	0.09	-0.02	-0.39	0.78	-0.45	0.36	0.05
Suburban[2]										
Corporate	0.24	0.59	0.18	[3]	[3]	0.17	0.76	-0.41	0.25	0.04
4 Lane 18,000 & over ADT										
Urban	0.69	0.66	0.29	0.27	0.18	0.51	0.71	-0.72	0.57	0.49
Suburban[2]										
Corporate	0.64	0.60	0.20	[3]	[3]	0.33	0.72	-0.72	0.49	0.50
4 Lane, 9,000 & over ADT										
Business	0.49	0.66	0.14	0.11	-0.10	0.39	0.79	-0.56	0.35	0.14
Residential & Mixed	0.67	0.49	0.34	0.37	0.19	0.51	0.74	-0.36	0.68	0.07
4 Lane, All ADT's										
Suburban	-0.02	0.36	0.09	0.37	0.22	0.69	0.62	-0.36	0.52	0.17
Corporate Portland	0.64	0.58	0.29	[3]	[3]	0.27	0.78	-0.46	0.48	0.44
Corporate Non-Portland	0.19	0.47	0.23	-0.17	-0.01	0.12	0.71	-0.17	-0.07	-0.18

Note Roadway elements considered for regression equations are underlined

[1] Insufficient sample to compute simple correlation

[2] Insufficient sample sections for computation of reliable regression equations. Data combine with other ADT ranges

[3] Preliminary analysis indicated low correlations, therefore, computations were not completed.

field analysis, each section was classified by the roadside culture—that is, business, residential, or mixed, with the distinction being that if the roadside culture was composed of 75 percent business establishments it was typed business, and if the roadside culture was 75 percent residential it was typed residential. Sections with mixed culture were typed mixed.

A final subgrouping of the corporate sections was made to determine if the large Portland Metropolitan Area had characteristics which would make prediction in that area more reliable than predictions in the other corporate areas of the state.

A distribution of the study sections by number of lanes, ADT ranges, and area or culture type is shown in Table 1. Also shown in this table are the total lengths of the sections and average lengths of the sections for urban groupings, suburban and corporate groupings. In the analysis of the data, it was decided that any group with less than 20 sections would not provide an adequate sample to develop reliable regression equations. A review of this table indicates that there were only two suburban 2-lane sections with an ADT of 10,000 and over. This did not provide an adequate sample for computation for regression equations; therefore, they were combined with the next lower ADT group in the analysis. A similar procedure was used for other groupings which had inadequate samples.

The first step in the statistical analysis was to compute the zero order correlation coefficient between accident rates and each of the roadside elements. The results of these computations are shown in Table 2. In examining this table, a positive correlation indicates that accident rates increase with an increase in the roadway elements, whereas the negative correlation indicates that accident rates increase with a decrease in roadway elements. Perfect correlation between the factors would be indicated by a value of one, and no correlation would be indicated by a value of zero.

The ADT had, with two exceptions, a positive correlation with accident rates. This relationship varies from practically no correlation to fairly good correlation. It will be observed that the subgroupings, corporate, suburban, business, residential, and mixed, do not make any substantial change in the correlation between accident rates and the average daily traffic. The correlation between accident rates and ADT increased for both 2- and 4-lane sections for the higher ADT groups, with generally higher correlations for all groupings of 4-lane sections.

The number of commercial units per mile, with three exceptions, was positively correlated with accident rates. Here again, the correlations for subgroupings do not materially better the correlation of urban sections. This roadway element had some fairly good correlations with accident rates, and as a result was used more frequently than any other element in the development of regression equations.

The number of commercial driveways was generally positively correlated with accident rates. Although fairly good correlations were found, it will be noted that commercial driveways were not as highly correlated with accident rates as the number of commercial units.

The number of residential units per mile, similarly, was evenly divided between positive and negative correlations. With four exceptions the correlations were low and could not be used for the development of regression equations. Residential units were used only in some regression equations for the subgroupings suburban, residential, or mixed culture. In a few instances preliminary investigation indicated that very low correlations would be obtained; therefore, the correlation with accident rates was not computed.

The number of residential driveways per mile had in all cases a very low correlation with accident rates, and did not in any grouping have a high enough correlation so that it could be considered in developing regression equations. This element was the worst predictor of accident rates of any studied.

TABLE 3

ZERO ORDER CORRELATIONS BETWEEN ACCIDENT RATES AND ROADWAY ELEMENTS
(Weighted for Length)

Study Group	Accidents - Roadway Elements Correlations									
	ADT	CU	CDW	RU	RDW	INT	SIG	SP	PA	ELA
2 Lane, Under 5,000 ADT										
Suburban	0 01	-0.20	-0.22	-0.24	-0 19	-0 01	[1]	0 41	0.43	0 49
Corporate	-0 14	0 18	0 15	[2]	[2]	0 07	0 04	-0 08	0.27	0 03
2 Lane, 5,000 - 9,999 ADT										
Corporate	0 35	0.77	0 34	[2]	[2]	0.44	0.59	-0 32	0.46	0 22
2 Lane, 10,000 & over ADT										
Corporate	0 32	0.76	0 50	[2]	[2]	0.66	0.89	-0 64	0.62	0.17
2 Lane, 5,000 & over ADT										
Suburban	0.08	0 26	0.41	0.09	0.08	0.66	0.34	-0 07	0 22	0.38
2 Lane, All ADT's										
Corporate Portland	0 29	0 68	0.42	[2]	[2]	0 60	0.86	-0.55	0 36	-0.20
Corporate Non-Portland	0 09	0 46	0.26	[2]	[2]	0 24	0 38	-0.24	0.39	0 19
4 Lane, Under 9,000 ADT										
Corporate	0 05	0.63	0 20	-0 13	0 05	0 12	0 70	-0 34	-0 12	-0 36
4 Lane, 9,000 - 17,999 ADT										
Corporate	0.34	0.71	0 38	[2]	[2]	0.47	0.82	-0 58	0.32	0 03
4 Lane, 18,000 & over ADT										
Corporate	0.54	0.71	0.53	[2]	[2]	0 52	0.79	-0.73	0.52	0.30
4 Lane, All ADT's										
Suburban	-0.08	0.37	0 07	0.62	0 52	0 67	0 48	-0 48	0.64	0.19
Corporate Portland	0.48	0 71	0 54	[2]	[2]	0 54	0.81	-0 63	0 54	0 30
Corporate Non-Portland	-0.29	0.31	0 18	[2]	[2]	0 08	0.34	-0 18	-0 13	-0 38

Note Roadway elements considered for regression equations are underlined
[1] Insufficient sample to compute simple correlation
[2] Insufficient sample sections for computation of reliable regression equations. Data combine with other ADT ranges.

TABLE 4

MULTIPLE CORRELATIONS BETWEEN ROADWAY ELEMENTS AND ACCIDENT RATES

Study Group	Best Predictors	Coefficient of Multiple Correlation Un-Weighted	Coefficient of Multiple Correlation Weighted	Standard Error of Estimate Un-Weighted	Standard Error of Estimate Weighted	Ratio of Standard Error of Estimate to the Mean Un-Weighted	Ratio of Standard Error of Estimate to the Mean Weighted
2 Lane, Under 5,000 ADT							
Urban	[1]	[1]	[2]	[1]	[2]	[1]	[2]
Suburban	(RU), (SP), (ELA)	0.37	0.52	5.26	8.85	1.57	1.24
Corporate	CU, PA	0.39	[2]	17.43	[2]	1.74	[2]
Business	[1]	[1]	[2]	[1]	[2]	[1]	[2]
Residential	RU, PA	0.55	[2]	6.76	[2]	1.11	[2]
Mixed	[1]	[1]	[2]	[1]	[2]	[1]	[2]
2 Lane, 5,000 - 9 999 ADT							
Urban	ADT, CU, INT, SIG, SP, PA	0.74	[2]	6.44	[2]	0.69	[2]
Suburban[3]							
Corporate	(ADT), (CU), (CDW), (INT), (SIG), (SP), (PA)	0.75	0.82	6.82	6.11	0.68	0.62
2 Lane 10,000 & over ADT							
Urban	ADT, CU, CDW, INT SIG SP	0.89	[2]	8.07	[2]	0.42	[2]
Suburban[3]							
Corporate	(ADT), CU, (CDW), (INT), (SIG), (SP), (PA)	0.89	0.93	7.98	5.44	0.39	0.34
2 Lane, 5,000 & over ADT							
Suburban	(CDW), (INT), (SIG), (ELA)	0.66	0.75	3.58	2.95	0.56	0.44
Business	ADT, CU. INT, SIG, SP, PA	0.86	[2]	8.25	[2]	0.53	[2]
Residential	ADT, RU	0.49	[2]	4.56	[2]	0.64	[2]
Mixed	ADT, CU, RU, INT, PA	0.69	[2]	5.25	[2]	0.62	[2]
2 Lane, All ADT's							
Corporate Portland	(ADT), (CU), (CDW), (INT), (SIG), (SP), (PA)	0.92	0.91	7.05	5.48	0.37	0.36
Corporate Non-Portland	(CU), PA	0.48	0.46	13.23	14.28	1.29	1.38
4 Lane, Under 9,000 ADT							
Urban	CU, SIG	0.84	[2]	7.75	[2]	0.59	[2]
Suburban[3]							
Corporate	(CU), (SIG), (SP), (ELA)	0.86	0.80	7.67	8.17	0.57	0.49
Business	CU, SIG	0.86		8.46	[2]	0.55	[2]
Residential & Mixed	[1]	[1]		[1]	[2]	[1]	[2]
4 lane, 9,000 - 17,999 ADT							
Urban	CU, INT, SIG, SP, PA	0.81	[2]	5.95	[2]	0.42	[2]
Suburban[3]							
Corporate	(ADT), (CU), (CDW), (INT), (SIG), (SP)	0.78	0.88	6.40	4.55	0.40	0.28
4 Lane, 18,000 & over ADT							
Urban	ADT, CU, INT, SIG, SP, PA	0.89	[2]	6.30	[2]	0.29	[2]
Suburban[3]							
Corporate	(ADT), (CU), (CDW), (INT), (SIG), (SP), (PA)	0.88	0.92	6.20	5.33	0.25	0.25
4 Lane, 9,000 & over ADT							
Business	ADT, CU, INT, SIG, SP, PA	0.87	[2]	6.30	[2]	0.33	[2]
Residential & Mixed	ADT, CU, INT, SIG, PA	0.86	[2]	4.78	[2]	0.35	[2]
4 Lane, All ADT's							
Suburban	(CU), RU, (RDW), (INT), (SIG), (SP), (PA)	0.90	0.88	3.56	3.40	0.34	0.33
Corporate Portland	(ADT), (CU), (CDW), (INT), (SIG), (SP), (PA)	0.84	0.90	7.37	5.28	0.36	0.28
Corporate Non-Portland	(ADT), (CU), (SIG), (ELA)	0.74	0.58	9.33	31.92	0.63	1.40

Note. Best predictors underlined for unweighted, parenthesis for weighted.

[1] No prediction equations computed because the zero order correlations indicated insignificant correlations

[2] No prediction equations computed

[3] Insufficient sample sections for computation of reliable regression equations

The number of intersections per mile was normally positively correlated with accident rates. Here again subgrouping of urban sections did not materially improve the correlations. Fairly good correlations were found for some subgroups, and this element was used frequently in the development of the regression equations.

The number of traffic signals per mile was with one exception always positively correlated with accident rates. This element had one of the highest correlations with accident rates of any of the elements considered, and was exceeded only by commercial units in the number of groups in which it was used for computation of regression equations. This element again did not show appreciable increase in correlations for the subgroups of the urban sections. In general, correlations for 4-lane sections were much higher than for 2-lane sections. This is probably accounted for, in part, by the traffic volume warrant used for determining the need for traffic signals. Although this element had one of the highest correlations with accident rates, it also had more variability than the other elements, and for this reason it was not one of the most reliable predictors of accident rates. In a number of subgroups there was an insuficient number of traffic signals for the computation of the zero order correlation.

The indicated speed as posted in the field showed in most subgroups a negative relationship with accident rates. The relationship was good enough in about one-half of the cases for inclusion in the regression equations. It appears from these correlations that increasing speed had a tendency to decrease the accident rates. However, the practice in Oregon has been, in part at least, to establish speed zones and post reduced indicated maximum speeds in those sections which have an abnormal accident experience. This procedure undoubtedly was responsible in a large part for the negative correlations obtained in that fewer adjustments for speed zones were established in sections with a relatively low accident rate, whereas sections of high accident rates were posted for reduced speeds.

The pavement width showed positive relationship with accident rates. However, the relationship varies considerably and was normally fairly low. Pavement width was included in several of the regression equation.

The effective lane width was generally positively correlated; however, there were frequent negative correlations. The correlation of this element was very small, and therefore this element was infrequently included in the regression equations.

The variable lengths of study sections indicated that a bias could be present in the analysis by giving undue consideration to short sections and insufficient consideration to relatively long sections. Therefore, a separate analysis was made on the suburban and corporate subgroups to test the presence of a length bias. For this analysis each study section was divided into a series of sections $\frac{1}{10}$ mile long. The roadway elements

TABLE 5

REGRESSION EQUATIONS, ACCIDENT RATES FROM ROADWAY ELEMENTS

Study Group	Regression Equations
2 Lane, Under 5,000 ADT	
Urban	[1]
Suburban	A = -9.67 + 0.14 SP + 0.73 ELA
Corporate	A = -6.28 + 0.08 CU + 0.50 PA
Business	[1]
Residential	A = -8.56 + 0.05 RU + 0 48 PA
Mixed	[1]
2 Lane, 5,000 - 9,999 ADT	
Urban	A = -7 54 + 0.09 ADT + 0.12 CU + 0.36 INT + 0 94 SIG + 0 06 SP - 0.01 PA
Suburban	[2]
Corporate	A = -4.44 + 0 09 ADT + 0 15 CU + 0 26 INT + 0.72 SIG + 0.02 SP - 0.08 PA
2 Lane, 10,000 and over ADT	
Urban	A = -18.21 + 0 09 ADT + 0.25 CU + 0.07 CDW + 0.41 INT + 3.87 SIG - 0 16 SP
Suburban	[2]
Corporate	A = -1 01 + 0.04 ADT + 0.02 CU + 0.12 CDW + 0.38 INT + 3.98 Sig - 0.15 SP
2 Lane, 5,000 and over ADT	
Suburban	A = -4.42 + 0.09 CDW + 0.57 INT + 0 29 ELA
Business	A = -2.66 + 0.10 ADT + 0.11 CU + 0.10 INT + 2.66 SIG - 0.16 SP + 0 05 PA
Residential	A = -2.18 + 0.11 ADT + 0.03 RU
Mixed	A = -5 88 + 0.05 ADT + 0.01 CU + 0 03 RU + 0.64 INT + 0.08 PA
2 Lane, all ADT's	
Corporate Portland	A = 13.45 - 0.03 ADT + 0.11 CU + 0.66 CDW - 0.33 INT + 4.27 SIG - 0 12 SP
Corporate Non-Portland	A = -1 10 + 0 14 CU + 0.19 PA
4 Lanes, Under 9,000 ADT	
Urban	A = 4 60 + 0.07 CU + 6 78 SIG
Suburban	[2]
Corporate	A = 3.04 + 0.08 CU + 6 78 SIG
Business	A = 2.61 + 0.08 CU + 6.88 SIG
Residential and Mixed	[1]
4 Lanes, 9,000 - 17,999 ADT	
Urban	A = 7.93 + 0 04 CU + 0.03 INT + 2 70 SIG - 0 10 SP + 0.05 PA
Suburban	[2]
Corporate	A = 7.89 + 0.04 CU + 2.47 SIG
4 Lanes, 18,000 and over ADT	
Urban	A = 1.79 + 0.18 ADT + 0.04 CU + 0.23 INT + 0.80 SIG - 0 70 SP - 0.09 PA
Suburban	[2]
Corporate	A = 33.57 + 0.14 ADT + 0.03 CU + 1.42 SIG - 0.94 SP - 0 32 PA
4 Lanes, 9,000 and over ADT	
Business	A = 37.19 + 0 04 ADT + 0.05 CU - 0 16 INT + 2.11 SIG - 0.40 SP - 0 35 PA
Residential and Mixed	A = -11 26 + 0 07 ADT + 0.16 CU + 0.09 INT + 2.19 SIG + 0.35 PA
4 Lanes, all ADT's	
Suburban	A = -26.97 + 0 04 CU + 0.04 RU + 0.32 INT + 2.88 SIG + 0 07 SP + 0.54 PA
Corporate Portland	A = 5.65 + 0.06 ADT + 0.08 CU + 2.44 SIG + 0.20 SP - 0.31 PA
Corporate Non-Portland	A = 6 55 + 0.06 CU + 3.80 SIG

[1] No prediction equations computed because the zero order correlations indicated insignificant correlations

[2] No prediction equations computed.

TABLE 6

REGRESSION EQUATIONS, ACCIDENT RATES FROM ROADWAY ELEMENTS
(Weighted for Length)

Study Group	Regression Equations
2 Lane, Under 5,000 ADT	
Suburban	A = -29.98 + 0.01 RU + 0.23 SP + 2.37 ELA
Corporate	[1]
2 Lanes, 5,000 - 9,999 ADT	
Urban	A = -3.40 + 0.05 ADT + 0.20 CU - 0.07 CDW + 0.32 INT + 1.09 SIG + 0.12 SP - 0.12 PA
2 Lanes, 10,000 and over ADT	
Urban	A = 3.76 - 0.01 ADT + 0.11 CDW + 0.34 INT + 4.25 SIG - 0.10 SP + 0.02 PA
2 Lanes, 5,000 and over ADT	
Suburban	A = -1.99 + 0.08 CDW + 0.68 INT + 0.94 SIG + 0.05 ELA
2 Lanes, all ADT's	
Corporate Portland	A = 26.50 - 0.07 ADT + 0.10 CU + 0.04 CDW - 0.07 INT + 4.62 SIG - 0.13 SP - 0.26 PA
Corporate Non-Portland	A = 3.81 + 0.18 CU
4 Lanes, Under 9,000 ADT	
Urban	A = 35.30 + 0.09 CU + 3.02 SIG - 0.12 SP - 3.02 ELA
4 Lanes, 9,000 - 17,999	
Urban	A = -1.43 + 0.05 ADT + 0.06 CU + 0.08 CDW + 0.06 INT + 2.13 SIG - 0.04 SP
4 Lanes, 18,000 and over ADT	
Urban	A = 13.17 + 0.10 ADT - 0.03 CU = 0.25 CDW - 0.39 INT + 2.40 SIG - 0.71 SP - 0.02 PA
4 Lanes, all ADT's	
Suburban	A = -23.06 + 0.03 CU + 0.11 RDW + 0.32 INT + 2.63 SIG + 0.04 SP + 0.48 PA
Corporate Portland	A = 14.05 + 0.04 ADT - 0.04 CU + 0.24 CDW - 0.09 INT + 3.07 SIG - 0.46 SP - 0.05 PA
Corporate Non-Portland	A = 162.34 - 0.32 ADT + 0.04 CU + 6.23 SIG - 11.27 ELA

[1] No prediction equations computed because the zero order correlations indicated insignificant correlations.

were prorated to each of these sections of uniform length. The zero order correlations between accident rates and roadway elements for the study sections weighted for length variation are shown in Table 3.

In general, the same comments apply to this table as applied to the zero order correlations in the unweighted data. In some instances the correlations between accident rates and roadway elements were increased by the weighting process; on the other hand, there were other cases where the correlations were not as good. In general, very little benefit was realized by adjusting the study sections to compensate for the varying length of sections.

From the zero order correlations referred to in Table 2, regression equations and coefficients of multiple correlation were computed for the various groupings of urban sections. Table 4 shows the coefficients of multiple correlation, the standard error of estimate, and the ratio of the standard error of estimate to the mean for these various groupings of urban sections. In addition, information is shown for the original data and the data compensated for length variations. Also shown are the roadway elements which were the best predictors of accident rates for each of the various subgroups.

As a guide for determining which regression equations would be acceptable for use, it was arbitrarily decided that the ratio of the standard error of estimate to the mean should be no larger than 0.5. This means that two-thirds of the time the regression equations can give predicted accident rates which will be in error less than 50 percent of the mean accident rate. This appears to be quite a large allowable error. On the other hand, the nature of the data employed in the study was such that relatively large errors must be used if any usable results are to be developed.

A review of Table 4 indicates that urban groupings of the original data gave fairly reliable regression equations, except for 2-lane highways with less than 5,000 vehicles per day. For this particular group, the zero order correlations were too low to warrant development of equations.

The subgrouping of urban sections by suburban, corporate, business, residential, and mixed culture did in some cases improve the result of the regression equations. However, this slight improvement did not warrant the additional time and effort necessary to make the subdivision. A study of the results of the unweighted data and the

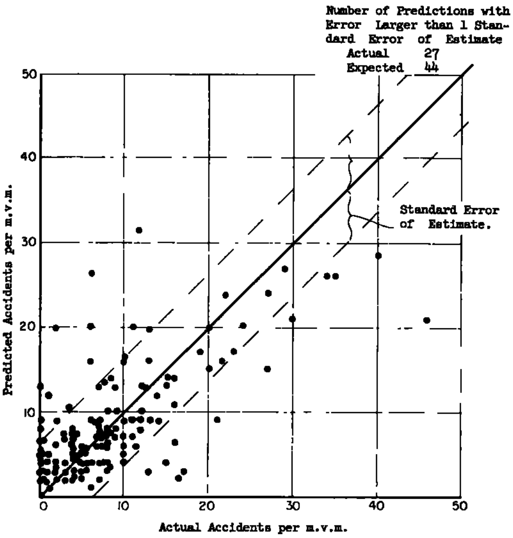

Figure 1. Comparison of predicted accident rates and actual accident rates in two-lane urban sections. (5,000 - 9,999 ADT)

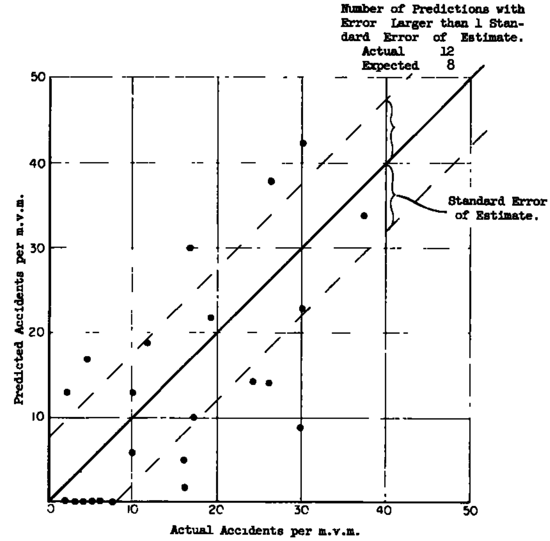

Figure 2. Comparison of predicted accident rates and actual accident rates in two-lane urban sections. (10,000 and over ADT)

weighted data indicate, in most cases, slightly better results for the weighted data than for the unweighted data. However, here again it did not appear that there was sufficient improvement to warrant the additional work required for this refinement.

Regression equations developed for the unweighted data are shown in Table 5, and the regression equations for the data weighted for length variations are shown in Table 6. In the development of these regression equations, only those elements which had

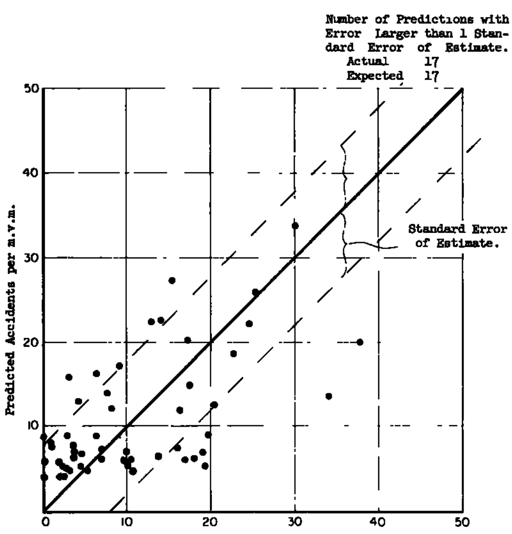

Figure 3. Comparison of predicted accident rates and actual accident rates in four-lane urban sections. (Under 9,000 ADT)

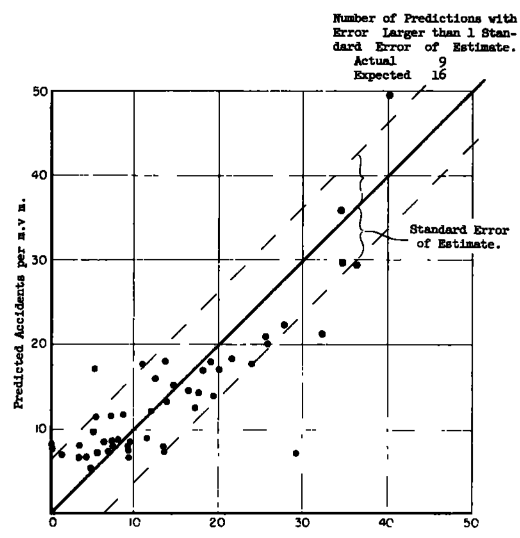

Figure 4. Comparison of predicted accident rates and actual accident rates in four-lane urban sections. (9,000 - 17,999 ADT)

zero order correlations greater than 0. 30 were considered. It will be noted in a few instances that roadway elements were indicated as being used in the development of the regression equations, however, they did not appear in the equation. This was the result of the coefficient for that element being less than 0. 005.

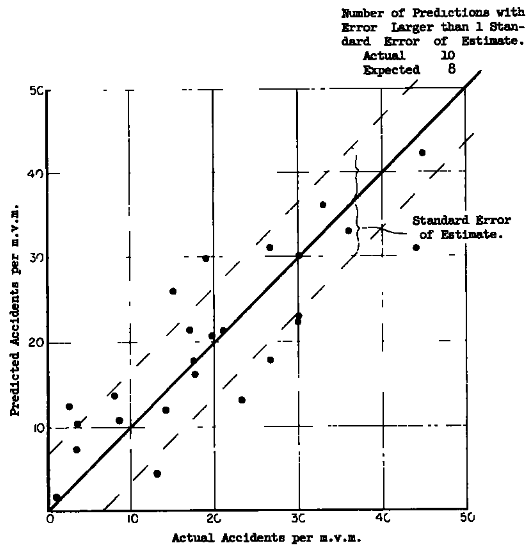

Figure 5. Comparison of predicted accident rates and actual accident rates in four-lane urban sections. (18,000 and over ADT)

COMPARISON OF PREDICTIONS WITH ACTUAL CASE HISTORIES

The regression equations developed in this study were employed to predict the accident rate which would be expected to occur on the urban sections within the various ADT groups.

Figures 1 through 5 show the comparison of predicted accident rates and actual accident rates. Also shown on the figures is the standard error of estimate for each equation. For normal distribution, 68 percent of the predictions should fall within the range of ± one standard error of estimate. For the five equations for which comparisons were made, the predicted values within the range of one standard error of estimate exceeded expectations for two of the equations, and were equal or less than expectations for the other three. The difference between the actual values within the range and the expected values was generally quite small and indicated that the theoretical standard error of estimate could be used with a reasonable degree of accuracy.

SUMMARY

1. Accident rate predictions are better for 4-lane highways than for 2-lane highways, and further are better for those sections with higher ADT.
2. Subgrouping of the urban sections by suburban, corporate, business, residential or mixed culture does not provide better predictions of motor vehicle accident rates.
3. Compensating for varying length of study sections does not provide better predictions of accident rates.
4. The most important roadway element in predicting traffic accident rates for urban sections is the number of commercial units adjacent to the section. The number of commercial units becomes a better predictor as the number of traffic lanes increases and as the average daily traffic increases.
5. The second most important roadway element in predicting traffic accident rates in the number of traffic signals which becomes a better predictor as the number of traffic lanes increases and as the average daily traffic increases.
6. Other important elements for predicting accident rates in order of importance are: the number of intersections, the indicated speed, the average daily traffic, and pavement width. Average daily traffic becomes a better predictor as the traffic volumes increase.
7. The number of commercial driveways, residential units and residential driveways, and the effective lane width are relatively unimportant roadway elements for predicting traffic accident rates. Commercial driveways can be used only for 2-lane sections. Residential driveways are of no value for predicting accident rates.
8. The following equations can be used to predict accident rates on urban sections of the state highway system:

2-Lane, 5,000 - 9,999 ADT

$$A = -7.54 + 0.09\ ADT + 0.12\ CU + 0.36\ INT + 0.94\ SIG + 0.06\ SP - 0.01\ PA$$

2-Lane, 10,000 or Over ADT

$$A = -18.21 + 0.09\ ADT + 0.25\ CU + 0.07\ CDW + 0.41\ INT + 3.87\ SIG - 0.16\ SP$$

4-Lane, Under 9,000 ADT

$$A = 4.60 + 0.07\ CU + 6.78\ SIG$$

4-Lane, 9,000 - 17,999 ADT

$$A = 7.93 + 0.04\ CU + 0.03\ INT + 2.70\ SIG - 0.10\ SP + 0.05\ PA$$

4-Lane, 18,000 or Over ADT

$$A = 1.79 + 0.18\ ADT + 0.04\ CU + 0.23\ INT + 0.80\ SIG - 0.70\ SP - 0.09\ PA$$

in which:

A = Accidents per million vehicle miles.
ADT = The average daily traffic divided by 100.
CU = The number of commercial units per mile.
CDW = The number of commercial driveways per mile.
INT = The number of intersections per mile.
SIG = The number of traffic signals per mile.
SP = The indicated speed.
PA = The pavement width in feet.

CONCLUSIONS

1. Motor vehicle accident rates are related to certain physical features of urban extensions of the highway system. This relationship is strong enough in the higher ADT ranges to make it possible to predict accident rates with a reasonable degree of accuracy on the basis of known physical features.
2. Accident rates on low volume roads do not have a strong relationship with any roadway feature.
3. Motor vehicle accident rates increase when:
    a. Number of commercial units adjacent to the section increases.
    b. Number of traffic signals increases.
    c. Number of intersections increases.
    d. Indicated speed decreases.
    e. Average daily traffic increases.
    f. Pavement width increases.

Appendix A

SOURCE OF RAW DATA

The raw data employed in this study were derived from 2 major sources. The first source was obtained by field inventory, and included roadway, parking, and culture characteristics of the sections. The second source of data was obtained from office records.

Field Data

The field workers worked in pairs, one of them driving while the other tabulated the roadway elements. Among the elements for which data were obtained were the number of commercial units and driveways, residential units and driveways, intersections, traffic signals, and channelization. Multiple bank calculators were used to facilitate this portion of the field work. In addition to the above tabulations, it was necessary to make periodic stops to measure pavement and shoulder widths each time there was an arterial change along the route.

Also recorded were the cultural classifications of each section. Three major groups were used for this—business, residential, and mixed. Based upon this criterion, if 75 percent of the buildings in the area were for business purposes it was classified as business, if 75 percent of the buildings were residential the section was classified as residential, and any combination of business and residential establishments between these two values was classified as mixed.

Information was also obtained on parking, whether it was permitted, and also the type of parking whether angle, parallel, or mixed. A sample field sheet is shown in Figure 6. The location for which the data were obtained would be indicated by the highway number appearing in the upper left-hand corner and milepost readings which were recorded in the extreme right-hand column under "Other Characteristics." The body of the table provided space for recording the following information in order: frequency

Highway No. 15

Commercial Units	Commercial Dwys.	Residential Units	Residential Dwys.	Inter-sec-tions	Sig-nals	Culture	Roadway Characteristic	Curb or Shldr.	Speed Indi-cated	Other Character-istics and Comments
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
3	5	1	1	I : 4 C : 0	0	B R M	PP PP AP AP NP NP 23'	RS : 5 LS : 6	25	M.P. : 92.92 at C.L. SISTERS Median Wid. 0 M___ U___ M.P. : 92.73
3	4	1	1	I : 1 C : 0	0	B R M	PP PP AP AP NP NP 48'	RS : 11 LS : 0 (curb)	25	M.P. : 92.73 50' beyond Larch Median Wid. 0 M___ U___ M.P. : 92.66
17	7	3	2	I : 3 C : 0	0	B R M	PP PP AP AP NP NP 48'	RS : 0 (curb) LS : 0 (curb)	25	M.P. : 92.66 50' beyond Fir St. Median Wid. 0 M___ U___ M.P. : 92.48
3	4	7	1	I : 2 C : 0	0	B R M	PP PP AP AP NP NP 44'	RS : 6 LS : 0 (curb)	25	M.P. : 92.48 50' beyond Ash St Median Wid. 0 M___ U___ M.P. : 92.39
8	12	2	1	I : 1 C : 0	0	B R M	PP PP AP AP NP NP 44'	RS : 6 LS : 4	25	M.P. : 92.39 50' beyond Oak St Median Wid. 0 M___ U___ Pine St 0.07 · M.P. : 92.27

Figure 6. Urban roadway elements study by Oregon State Highway Department.

of commercial units and commercial driveways, residential units and residential driveways.

Under the column headed "Intersections" opposite "I" were indicated the number of intersections, and opposite "C" the number of channelized intersections. Col. 6 was used for recording the number of traffic signals in the section. Col. 7 was used for the culture characteristics by circling "B" for business, "R" for residential, and "M" for mixed. Space was provided in Col. 8 for recording the lane width and type of parking. The lane width was recorded to the nearest foot just above the arrow. At the ends of the arrow appear the letters PP, AP, and NP, provided for recording of parallel parking, angle parking, and no parking, respectively, for each side of the road. In Col. 9 space was provided for recording the curb and shoulder characteristics of the particular section, with shoulder width indicated for both right and left shoulders. Zero for shoulder width was used to indicate a curve section. Col. 10 provides space for recording the posted speed for each section. Col. 11 provides space for recording the termini of each section by milepost. Space was also provided for a very brief description of the termini and to indicate median widths and, further, whether the medians were mountable or unmountable.

In addition to the field data sheets described above, the field workers carried a plat map of the city in which they were working; with a colored pencil they marked the route they traveled and indicated breaks between consecutive sections. It was necessary for the field workers to terminate a given section with any change in the field characteristics from the previous section; that is, the field workers terminated a given section if there was a change from angle to parallel parking and from parallel to no parking, or when the shoulder width changed from one section to another, or if there was a difference in culture characteristics, lane width, or indicated speed. These section breaks were necessary to obtain, to the extent possible, homogeneous characteristics of roadway elements for each section. As an example, it would be impossible to relate indicated speeds to accident rates if there were no consistent values of indicated speed for the given section.

The above paragraphs describe the data which were obtained during the field procedures. However, the data were very incomplete with regard to certain roadway elements. For example, commercial driveways and units, residential driveways and units, and intersections were recorded with regard to their absolute frequency. No attempt was made in the field to distinguish between commercial driveways which might handle as many as 500 cars per day and those that might not be used more than ten times per week. Also, with regard to commercial units and residential units, the only thing that was noted was the number of entrances facing the roadway. Thus a 4-unit apartment with 1 exit was counted as 1 residential unit, whereas a 4-unit apartment with 4 exits was counted as 4 units. The situation was often inaccurate with regard to commercial units, since a 10-story building might have only one street entrance while a one-story building might have two or more entrances. The former was counted as 1 commercial unit, whereas the latter was counted as 2 or more commercial units if it housed a separate business for each entrance.

With regard to intersections, no record was made of the type of intersection—that is whether it was a "T" or a "X" type. Furthermore, traffic volume on the side streets was not recorded nor utilized in any part of the analysis.

Office Data

The data gathered in the field as described previously were transcribed in the office onto code sheets (Fig. 7). In addition to the highway number and beginning milepost number reading in Cols. 2 and 3 of the code sheet, an identifying code was used in Col. 1 to identify the city, county, and population group of the city.

Col. 4 shows the length of each section in $1/100$ mile, and Col. 5 shows the average traffic volume in hundreds of vehicles per day. Cols. 6 through 9 code the frequency of the respective roadway elements per mile of section. These values were obtained by multiplying the reciprocal of the length of the section by the frequency of these elements as indicated on the field sheet. For example, if a section was $1/10$ mile long and

City Code County-City Population (1)	Highway No. (2)	Beg. M.P. (3)	Length .01's mi. (4)	ADT (5)	Comm. Units/mi. (6)	Cdws/mi. (7)	Resid. Units/mi. (8)	Rdws/mi. (9)	Urb. Code (10)	Cult. Code (11)	Inter./mi. (12)	Signals per mi. (13)	Channels per mi. (14)	Speed (15)	Pavement (16)	Shldr. (17)	Lane No. (18)	Mdn. Width (19)	Mdn. Code (20)	Dark Code (21)	Ef. Lane (22)	Non-inter. accidents per m.v.m. (23)	Inter. accidents per m.v.m. (24)
09011	015	09227	012	016	067	100	017	008	1	0	008	000	000	25	44	05	2	00	0	0	19	00000	00000
	015																						
09011	015	09239	009	018	033	044	078	011	1	2	022	000	000	25	44	03	2	00	0	0	17	00850	00850
09011	015	09248	018	020	094	039	017	011	1	2	017	000	000	25	49	00	2	00	0	0	16	00000	00382
09011	015	09266	007	019	043	057	014	014	1	0	014	000	000	25	48	06	2	00	0	0	22	00000	00000
09011	015	09273	019	018	016	026	005	005	1	0	021	000	000	25	23	06	2	00	0	0	10	00000	00000

Figure 7. Urban roadway elements study by Oregon State Highway Department.

had 3 commercial driveways, the commercial driveways per mile would be 30. Col. 10 was used to indicate whether a section was within or without the corporate limits of a city by coding zero for suburban and one for corporate sections.

The culture classification of each section was coded in Col. 11 with 0, 1, and 2 as the code for business, residential, and mixed, respectively. Cols. 12 through 14 were used for recording intersectional and traffic signal data. For these items the frequency per mile was coded. Cols. 15 through 19 were coded direct from the field sheets. Col. 20 was used to indicate whether a median was mountable or unmountable with a code of zero or one, respectively. The type of parking for each section was indicated in Col. 21 by using a code of zero for parallel, one for angle, and two for no parking; and three for no parking on one side and parallel on the other, and four for angle parking and parallel parking combined.

Col. 22 indicated to the nearest foot the effective lane width. This value was obtained by taking the total pavement width less the distance necessary for the type of parking which was permitted, divided by the number of lanes. Eight ft was allowed for parallel parking and 18 ft for angle parking on each side of the street. Thus, for a roadway 60 ft from curb to curb which allowed angle parking on both sides, the effective lane width would be 60 ft minus 36 ft divided by 2 lanes, for an effective lane width of 12 ft. If a section had a shoulder width of more than 6 ft, it was assumed that parking would be on the shoulder and no reduction was made in the pavement width.

The accident data recorded in Cols. 23 and 24 were obtained from office records. The frequency of accidents on each section was converted into accident rates and appears in terms of accidents per million vehicle miles. Accident rates were entered separately for intersectional and non-intersectional accident rates.

Appendix B

IBM PROCEDURE

The data on the completed code sheets were punched onto IBM cards. The code sheets were retained as duplicate records, but no further direct work involved them. The first step in the IBM procedure was the computation of the total accident rate, that is, the sum of the intersectional and non-intersectional accident rates. The IBM calculating punch type 602-A was used in the summation, and in this way the total accident rates were punched on the card from which the intersectional and non-intersectional rates were read. In the next step of the IBM computations, the sums of squares and the sums of the cross products among all the factors were developed. The 602-A punched these sums in summary cards following the deck of data cards. This same deck of data cards was then processed from the IBM accounting machine which printed the totals of each factor. In these two operations, all the data necessary in the computation of correlation coefficients were made available (assuming the sample size was known). In actual practice these operations were preceded by a stratification on the basis of desired grouping by means of the IBM sorter.

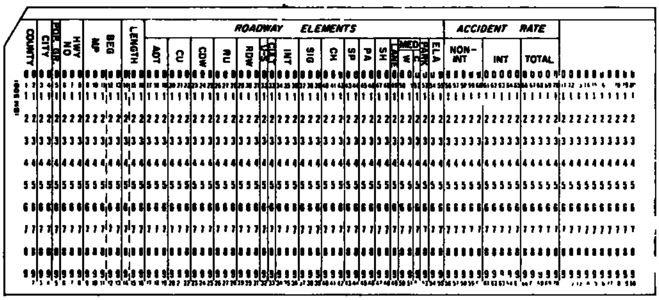

Figure 8. IBM card layout—roadway elements study.

A sample IBM card appears in Figure 8. A detailed description of the information appearing there is as follows: county, city, population codes (Cols. 1 through 5), highway number (Cols. 6 to 8), beginning M. P. (Cols. 9 through 13), length of section in hundredths of a mile (Cols. 14 through 16), ADT (Cols. 17 through 19), commercial units per mile (Cols. 20 to 22), commercial driveways per mile (Cols. 23 through 25), residential units per mile (Cols. 26 through 28), residential driveways per mile (Cols. 29 through 31), urban code (Col. 32), culture code (Col. 33), intersections per mile (Cols. 34 through 36), signals per mile (Cols. 37 through 39), channels per mile (Cols. 40 to 42), speed limit (Cols. 43, 44), pavement width (Cols. 45 through 46), shoulder width (Cols. 47 through 48), number of lanes (Col. 49), median width (Col. 50, 51), median code (Col. 52), parking code (Col. 53), effective lane (Cols. 54 and 55), nonintersectional accidents per million vehicle miles (Cols. 56 through 60), intersectional accidents per million vehicle miles (Cols. 61 through 65), and total accidents per million vehicle miles (Cols. 66 through 70). As mentioned before, the data in Cols. 1 through 65 were key punched onto the card, whereas the data in Cols. 66 through 70 were calculated using the 602-A and punched onto the same card.

Appendix C

MULTIPLE CORRELATION TECHNIQUE

The analytic procedures employed in this study were attempts to evaluate the individual contributions of various roadway factors and ultimately the joint effects of the several roadway elements taken together upon the accident experience. The multiple correlation technique is ideally suited to this end since it is designed to provide an estimate of the over-all relationship between several factors and a given factor. In this

TABLE 7

ILLUSTRATION OF MULTIPLE CORRELATION TECHNIQUE

Part 1 Intercorrelations, means, and standard deviations of the data of the multiple correlation illustrations.

Two-Lane Urban, 10,000 and over ADT

	Intercorrelations (r)							
	X_3 (CU)	X_4 (CDW)	X_5 (INT)	X_6 (SIG)	X_7 (SP)	X_1 (A)	M_x Mean	σ_x Standard Deviation
X_2 (ADT)	0 1794	0 0748	0.1795	0 1470	-0.1502	0.4190	119.231	16 439
X_3 (CU)	-	0.5751	0 6251	0.6259	-0.4283	0 6684	53.615	49.516
X_4 (CDW)	-	-	0 4314	0.3279	-0.5343	0.4386	31 538	18.731
X_5 (INT)	-	-	-	0 3861	-0.4102	0.5327	15.577	6 968
X_6 (SIG)	-	-	-	-	-0 3391	0.8485	2.462	3 003
X_7 (SP)	-	-	-	-	-	-0.4363	31 538	7.308
X_1 (A)	-	-	-	-	-	-	19.055	17.571

TABLE 8

ILLUSTRATION OF MULTIPLE CORRELATION BY THE DOOLITTLE METHOD
Two-Lane Urban and Suburban, ADT 10,000 and over
Part 2

Column Number		2	3	4	5	6	7	1	
Variable		ADT	CU	CDW	INT	SIG	SP	A	Check Sum
Row	Instructions								
A	r_2K	1.0000	0.1794	0.0748	0.1795	0.4170	-0.1502	0.4190	2.1195
B	$A \div (-A_2)$	-1.0000	-0.1794	-0.0748	-0.1795	-0.4170	0.1502	-0.4190	-2.1195
C	r_3K	(0.1794)	1.0000	0.5751	0.6251	0.6259	-0.4283	0.6684	3.2456
D	$A \times B_3$	(-0.1794)	-0.0332	-0.0134	-0.0322	-0.0748	0.0269	-0.0752	-0.3802
E	$C + D$		0.9678	0.5617	0.5929	0.5511	-0.4014	0.5932	2.8654
F	$E \div (-E_3)$		-1.0000	-0.5804	-0.6126	-0.5694	0.4148	-0.6129	-2.9607
G	r_4K	(0.0748)	(0.5751)	1.0000	0.4314	0.3279	-0.5343	0.4386	2.3155
H	$A \times B_4$	(-0.0748)	(-0.0134)	-0.0056	-0.0134	-0.0312	0.0112	-0.0313	-0.1585
I	$E \times FA$		(-0.5617)	-0.3260	-0.3441	-0.3199	0.2330	-0.3443	-1.6631
J	$G + H + I$			0.6684	0.0739	-0.0232	-0.2901	0.0630	0.4919
K	$J \div (-J_4)$			-1.0000	-0 1106	0.0347	0.4340	-0.0943	-0.7359
L	r_5K	(0.1795)	(0.6251)	(0.4314)	1 0000	0.3861	-0.4102	0.5327	2.7446
M	$A + B_5$	(-0.1795)	(-0.0332)	(-0.0134)	-0.0322	-0.0749	0.0270	-0.0752	-0.3804
N	$E \times F_5$		(-0.5929)	(-0.3441)	-0.3632	-0.3376	0.2459	-0.3634	-1.7553
O	$J \times K_5$			(-0.0739)	-0.0082	0.0026	0.0321	-0.0070	-0.0544
P	$L + M + N + O$				0.5964	-0.0238	-0.1052	0.0871	0.5545
Q	$P \div (-P_5)$				-1.0000	0.0399	0.1764	-0.1460	-0.9297
R	r_6K	(0.4170)	(0.6259)	(0.3279)	0.3861	1.0000	-0.3391	0.8485	3.2663
S	$A \times B_6$	(-0.4170)	(-0.0748)	(-0.0312)	(-0.0749)	-0.1739	0.0626	-0.1747	-0.8838
T	$E \times F_6$		(-0.5511)	(-0.3198)	(-0.3376)	-0.3138	0.2286	-0.3378	-1.6316
U	$J \times K_6$			(0.0232)	(0.0026)	-0.0008	-0.0101	0.0022	0.0171
V	$P \times Q_6$				0.0238	-0.0009	-0.0042	-0.0035	-0.0221
W	$R + S + T + U + V$					0.5106	-0.0622	0.3417	0.7901
X	$W \div (-W_6)$					-1.0000	0.1218	-0.6692	-1.5474
A'	r_7K	(-0.1502)	(-0.4283)	(-0.5343)	(-0.4102)	(-0.3391)	1.0000	-0.4363	-1.2984
B'	$A \times B_7$	(0.1502)	(0.0269)	(0.0112)	(0.0270)	(0.0626)	-0.0226	0.0629	0 3183
C'	$E \times F_7$		(0.4014)	(0.2330)	(0.2459)	(0.2286)	-0.1665	0.2461	1.1886
D'	$J \times K_7$			(0.2901)	(0.0321)	(-0.0101)	-0.1259	0.0273	0.2135
E'	$P \times Q_7$				(0.1052)	(-0.0042)	-0.0186	0.0154	0.0978
F'	$W \times X_7$					(0.0622)	-0.0076	0.0416	0.0962
G'	$A' + B' + C' + D' + E' + F'$						0.6588	-0.0430	0.6160
H'	$G \div (-G_7)$						-1.0000	0.0653	-0.9350

study the given factor was always the accident rate whether this be for non-intersectional, intersectional, or total accidents. The multiple correlation procedure provides two valuable forms of information. First, it provides a coefficient of multiple correlation (R) which expresses quantitatively the extent of association between the several factors and the accident rates. This multiple correlation coefficient varies from 0 to 1. The stronger the relationship between these various roadway factors and accident rates, the larger R becomes. If there is no relationship, R equals 0; if there is perfect relationship, R equals 1. The second important result of the multiple correlation technique is the development of a multiple regression equation. The multiple regression equation is a statement of the theoretical contribution of the various roadway elements to the accident rate. The effectiveness of the multiple regression coefficient and equation depends on the predictive factors selected for study.

In other words, if there are 20 factors, the regression equation's usefulness will depend upon the particular variables chosen for analysis. Thus, it would be impractical to evaluate the separate contributions of all the factors when it can be assumed that many of the 20 variables will be of little or no appreciable value. The decision as to which variables to include is made on the basis of the variable's relationship to the accidents recorded. A quantitative measure of these relationships is given by the Pearsonian correlation coefficient:

$$r = \frac{N\Sigma XY - (\Sigma X)(\Sigma Y)}{\sqrt{N\Sigma X^2 - (\Sigma X)^2}\sqrt{N\Sigma Y^2 - (\Sigma Y)^2}} \qquad (1)$$

TABLE 9

ILLUSTRATION OF MULTIPLE CORRELATION TECHNIQUE
Two-Lane Urban and Suburban, ADT 10,000 and Over

A Check of the Back Solution of the Normal Equation

The Back Solution

$\beta_{17.23456}$						-0.0653 =	-0.0653	(-HI)
$\beta_{16.23456}$					0 6612 =	-0.0080	+0.6692	(-XI)
$\beta_{15.23456}$				0 1609 =	+0 0264	-0.0115	+0 1460	(-QI)
$\beta_{14.23456}$			0 0711 =	-0.0178	+0 0229	-0.0283	+0.0943	(-KI)
$\beta_{13\ 23456}$		0 0694 =	-0.0413	-0.0986	-0.3765	-0 0271	+0 6129	(-FI)
$\beta_{12\ 23456}$	0.0868 =	-0 0125	-0 0053	-0.0289	-0.2757	-0.0098	+0 4190	(-BI)

Note: The Beta Coefficients found at the left of each row are the algebraic sums of the values to the right of the equal sign: i e $\beta_{15\ 23467}$ = 0.0264 - 0.0115 + 0 1460

A Check of the Back Solution

X_K		β_{1K}	rK_7	β_{1KrK_7}
X	ADT	0.0868	-0.1502	-0 0130
X	CU	0 0694	-0 4283	-0.0297
X	CDW	0 0711	-0.5343	-0 0380
X	INT	0.1609	-0.4102	-0.0660
X	SIG	0 6612	-0.3391	-0 2242
X	SP	-0 0653	1.0000	-0 0653
X	A		r_{17} = -0.4363 $\Sigma\beta_{1KrK_7}$	-0.4362

in which:

N = Number of sections;
ΣX = Sum of the X-factor values (e. g., sum of CDW);
ΣY = Sum of the Y-factor values (e. g., sum of accident rates);
ΣY^2 = Sum of the squared Y-factors;
ΣX^2 = Sum of the squared X-factors; and
ΣXY = Sum of the X and Y crossproducts.

As shown in Eq. 1, the two sums of squares, the sum of the crossproducts, the two individual sums, and the sample size, are the only data required for the calculation. The results of the IBM computations described in Appendix B provided this information. Insertion in Eq. 1 and the subsequent calculation was accomplished by a desk calculator. After a large portion of the analysis had been completed, an IBM 650 became available and a program was obtained which computed all the zero order correlations and the final regression equations. This reduced tremendously the amount of desk calculator work required in making the computations.

In a general way, the factors which will have the most value in a multiple regression technique or multiple regression equation, are those which are most highly correlated with the factor that is to be predicted, and at the same time related to the slightest extent to the other predicting factors. In the example that will be illustrated shortly these most valuable factors are: average daily traffic, commercial units, commercial driveways, intersections, signals, and speed. This example shows the development of the equations for prediction of accident rates on 2-lane urban roadways with an ADT of 10,000 and over. The zero order correlations between the various factors and the accident rates are shown in Table 7, along with the means and the standard deviations of each of the factors concerned (the mean is the same as the arithmetic average, and the standard deviation is the standard way of presenting the amount of variability within the group). All the data are available which are necessary to proceed with the development of the multiple correlation coefficient and the multiple regression equation.

Eq. 2 is used to develop the coefficient of multiple correlation.

$$R_{1.234567} = \sqrt{\beta_{12}r_{12} + \beta_{13}r_{13} + \beta_{14}r_{14} + \beta_{15}r_{15} + \beta_{16}r_{16} + \beta_{17}r_{17}} \quad (2)$$

All the information necessary to solve this equation is contained in Tables 7, 8, and 9. This illustrates the method of multiple correlation developed by Doolittle.[4] Substituting the appropriate values, the multiple correlation coefficient becomes 0.888.

Table 9 provides the check of the Back Solution of the Beta coefficients shown in the table. The next step in the multiple correlation technique is the development of a multiple regression equation. This is shown in Eq. 3.

$$Y = a + b_{12}M_2 + b_{13}M_3 + b_{14}M_4 + b_{15}M_5 + b_{16}M_6 + b_{17}M_7 \quad (3)$$

The coefficients shown in Eq. 3 are developed as shown in Eqs. 4, 5, 6, 7, 8, and 9:

$$b_{12} = (\sigma_1/\sigma_2)\,\beta_{12} \quad (4)$$

$$b_{13} = (\sigma_1/\sigma_3)\,\beta_{13} \quad (5)$$

$$b_{14} = (\sigma_1/\sigma_4)\,\beta_{14} \quad (6)$$

$$b_{15} = (\sigma_1/\sigma_5)\,\beta_{15} \quad (7)$$

$$b_{16} = (\sigma_1/\sigma_6)\,\beta_{16} \quad (8)$$

$$b_{17} = (\sigma_1/\sigma_7)\,\beta_{17} \quad (9)$$

where σ_1, σ_2, σ_3, σ_4, σ_5, σ_6, σ_7, are the standard deviations of accident rates, average daily traffic, commercial units, commercial driveways, intersections, signals, and speed, respectively. The Beta Coefficients are those shown in Table 9.

The a coefficient of Eq. 10 is developed by subtracting the products of the various b coefficients, times their means (M) from the mean of the predicted value, in this case the mean of the accidents.

$$a = M_1 - b_{12}M_2 - b_{13}M_3 - b_{14}M_4 - b_{15}M_5 - b_{16}M_6 - b_{17}M_7 \quad (10)$$

Substituting the appropriate values, the multiple regression equation becomes that shown in Table 5. The equation is repeated in Eq. 11.

$$A = -18.21 + 0.09\,\text{ADT} + 0.25\,\text{CU} + 0.07\,\text{CDW} + 0.41\,\text{INT} + 3.87\,\text{SIG} - 0.16\,\text{SP} \quad (11)$$

For example, if a particular 1-mile section in this volume range had a 35-mile indicated speed, 14 intersections, 59 commercial units, 49 commercial driveways, 2 signals, and an average daily traffic of 10,800, the predicted number of accidents would be derived as follows:

$$A = -18.21 + 0.09(108) + 0.25(59) + 0.07(49) + 0.41(14) + 3.87(2) -0.16(35)$$

$$A = -18.21 + 9.72 + 14.75 + 3.43 + 5.74 + 7.74 - 5.60 = 17.57 \text{ Accidents/mvm}$$

[4] Guilford, J. P., "Psychometric Methods." McGraw-Hill Book Company, p. 393-399, (1936).

Sampling Procedures for Determining Speed Characteristics at Rural Locations: A Progress Report

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The research activities covered by this progress report are concerned with two major factors regarding sampling procedures for determining speed characteristics at rural locations, as follows:

1. The best times for observing speeds in order to obtain the desired characteristics.
2. The size of sample required for acceptable accuracy.

Discussions of field study procedures, the sites used for observations, the analyses made, apparent trends in the data, and the development of a table of confidence interval limits are included.

Because the work in this area is still in progress, no definite conclusions are proposed. However, some trends suggested by the data are discussed.

● IN THE FALL of 1955 an investigation of vehicle speed regulation was initiated at the University of Illinois. This research project is being conducted by the Engineering Experiment Station of the University of Illinois in cooperation with the Illinois Division of Highways and the U. S. Bureau of Public Roads.

Speed and speed regulation are of primary interest to everyone concerned with the operation of vehicles on streets and highways. Due to the divergence of speed control criteria among different localities, many streets and highways are zoned with unnecessarily high or low speed regulations. Basically, all vehicles should be operated at speeds which are reasonably safe and proper for existing conditions. Where speed regulation appears desirable it should be based on a proper traffic engineering investigation and evaluation of conditions rather than on an arbitrarily selected speed restriction.

With these thoughts in mind, the following major objectives were established for this research project:

1. To determine the factors involved in regulating vehicular speeds.
2. To evaluate these factors and to establish warrants for the regulation of vehicular speeds.
3. To develop procedures for the application of these warrants.
4. To develop methods and devices for obtaining maximum compliance with the speed regulation.

A necessary instrument in achieving these objectives is the sampling of vehicle speeds in order to determine speed characteristics at various locations. Because so many varying opinions presently exist as to how this should be done, initial efforts are being directed towards developing scientific sampling procedures.

In general, there are three classifications of areas for which sampling procedures conceivably would vary considerably—rural areas, urban areas, and those in-between or fringe areas which are neither completely rural nor completely urban. A possible fourth classification is controlled access facilities. In the development of scientific sampling techniques for speed surveys, the procedure of beginning in the simplest area, the rural classification, and progressing towards the most difficult was followed.

The following discussion of progress in developing sampling procedures for determining speed characteristics at rural locations is based on the work accomplished thus far. It is not intended to present conclusive finalities on the subject, but only the possible implications of the findings. It is hoped that this information will be of use to those in the field of highway and traffic engineering in developing scientific sampling techniques for vehicle speed surveys. In addition, other agencies may be encouraged to perform research along similar veins which will supplement these data.

DEFINITIONS

An attempt has been made to use standard terminology in this report. However, to avoid confusion the following definitions are employed throughout this paper:

Speed Differential—The difference between the 85th percentile speed and the 15th percentile speed; a measure of the differential between higher speeds and lower speeds; gives the miles-per-hour range in which 70 percent of the drivers are traveling.

Mean Speed, Arithmetic Mean—The sum of the speeds of all the vehicles sampled divided by N, the number of vehicles sampled.

Percentile Speeds—The speed at or below which the designated percentage of vehicles was traveling.

Freeflowing Vehicles—Vehicles traveling at speeds which are not influenced by a leading vehicle.

Population—The specific set of speed values being estimated in the study; the population values presented in this paper are for a 24-hr period.

Standard Deviation—A measure of the dispersion of the individual speed values in the sample about the mean value of the sample.

Absolute Deviation—The numerical difference between the sample speed and the population speed without regard to sign.

STUDY PROCEDURE

The development of study procedures for determining sampling techniques for speed surveys was directed toward answering two major questions, as follows:

1. What minimum size of sample is necessary to provide acceptable accuracy for practical application?
2. During what time periods of the day should observations of speeds be made in order to obtain representative characteristics?

A third question of importance is the effect of including non-freeflowing vehicles in a sample. The primary concern in this problem is the difficulty of defining and identigying freeflowing vehicles and of being consistent in designating them when different observers are making the surveys. For the purposes of surveys conducted in this study, a non-freeflowing vehicle is considered to be a "tailgating" vehicle or a vehicle in the act of passing another. Tailgating vehicles are those vehicles following sufficiently close to a leading vehicle to have their speed appreciably influenced. The designation of such vehicles for this study was by the individual observers, based on the speed of the vehicle and the time between vehicles. The vehicles had to be traveling approximately the same speed and have less than 4-sec time intervals.

Two categories of studies appeared necessary to answer the major questions. Intuition suggests that speed values vary from hour to hour at a given site. Current literature indicates that studies have inferred this as truth. However, it is necessary to know how consistent these variations are from site to site and from day to day.

The first type of study needed was a continuous 24-hr survey at each of many sites. Tentative answers to the major questions could be formulated from the data acquired in such studies.

Supplementary studies were then needed to indicate the variability of speeds at a given site. The second category included two forms of studies—daily variation studies to indicate the day-to-day consistency of speed values and monthly variation studies to indicate the probable variation of speeds throughout the year.

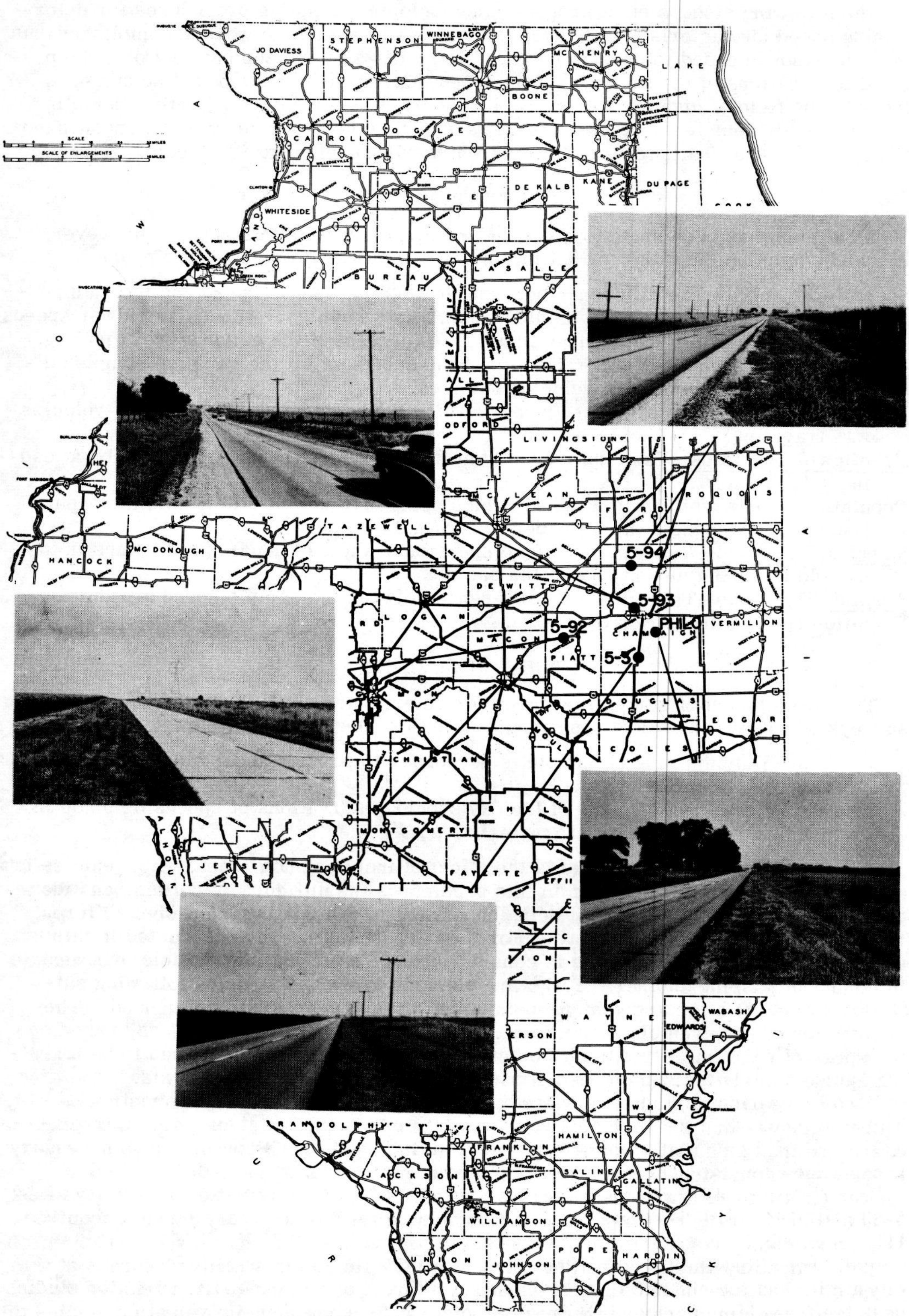

Figure 1. Map of Illinois, showing location of 24-hr survey sites.

Two pilot 24-hr surveys were performed during the summer of 1956 to develop study methodology. From these studies, procedures for conducting the different types of surveys were developed and refined. A schedule was then prepared for detailed studies during the summer of 1957.

ESTABLISHMENT OF SITES

All of the speed survey sites for studies of sampling techniques have been located in rural areas on generally level tangent sections of highway. No sites were selected within ½ mi of a minor highway intersection, or within 1 mi of a major intersection. Efforts were made to provide an adequate shoulder and ditch, or driveway, to permit parking the survey vehicle at least 8 ft from the edge of the pavement. To facilitate the initial collection of field data, all of these project sites were located in the vicinity of the University of Illinois.

In addition to the surveys conducted by project personnel, the Illinois Division of

TABLE 1

DATA FOR 24-HR SPEED SURVEY SITES

Site No.	Location	ADT 1957	Observed 24-Hour Volume	Trucks (%)	Date of Study	Pavement Surface	No. of Lanes	Lane Width (ft)	Approx. Shoulder Width (ft)
Philo	Philo Road	2,100[1]	2,010	11	June 1956	Bit. Conc.	2	10	6
5-3	US 45, Tolono	4,500[1]	3,690	18	October 1956[2]	Bit. Conc.	2	12	6
5-3	US 45, Tolono	4,700	4,240	18	July 1957	Bit. Conc.	2	12	6
5-92	Ill. 47, Monticello	2,100	2,240	16	June 1957	P. C. Conc.	2	11	10
5-93	US 150, Champaign	4,800	5,370	16	June 1957	Bit. Conc	2	9	8
5-94	US 136, Fisher	3,100	2,700	18	June 1957	P. C. Conc.	2	9	10

[1]1956 ADT.
[2]Not a continuous time period.

Highways conducted several 24-hr surveys during the summer of 1957. The sites for these surveys were located throughout the state. These data are to be used for checking tentative theories based on project surveys.

Five project sites (Fig. 1) were selected with various traffic volumes ranging from approximately 2,000 to 5,000 ADT. Traffic at only one site (5-93) reached near capacity volumes during the survey. All of the sites were located on two-lane pavements with various lane widths. A total of six 24-hr surveys was made at the five locations. One 24-hr survey was conducted at each site except site 5-3, where two 24-hr surveys were conducted. Site 5-3 was also used for the monthly and daily variation studies. Two of the 24-hr surveys were pilot studies. Table 1 gives pertinent data for these locations.

The first 24-hr survey was a pilot study performed at the Philo Road site in 1956. This is on a paved local road, southeast of Urbana. Data from this survey were used in establishing tentative procedures for performing the subsequent 24-hr surveys. These procedures were revised slightly after the second pilot study, which was made at site 5-3 in 1956. Although this survey was not made in one continuous time period, it provided adequate data for developing study procedures.

During June 1957 three 24-hr surveys were made at new locations, designated 5-92, 5-93 and 5-94. Site 5-93 was located 2 mi west of the outskirts of Champaign on US 150. Some highway construction work was in progress at the city limits, but this probably did not affect the speeds at the site. However, due to this close proximity to the city limits and the high ADT, near-capacity volumes were reached at site 5-93 during peak traffic volume periods. None of the other sites approached capacity.

On July 1, 1957, a new statewide speed limit became effective in Illinois. This law

Figure 2. Radar set-up and observer's car.

replaced the previous law, which called for reasonable and prudent speeds in areas not specifically zoned. The revised speed limits on rural highways without access control are as follows:

Passenger vehicles	65 mph
Buses	60 mph
Trucks less than 8,000 lb gross weight	55 mph
Trucks over 8,000 lb gross weight	50 mph

These limits are effective for both day and night periods. Although enforcement was not heavy immediately following the enactment of these limits, their effects on the speed surveys conducted after July 1 must be considered.

In July 1957 a second 24-hr survey was made at site 5-3. Also, a daily variation study was made at this site, later supplemented with another such study at the same site in August, the latter not being one continuous 7-day period as was the first.

In March 1957 a monthly variation study was started at site 5-3. It is anticipated that this will be a continuing study for the duration of the project.

FIELD STUDY PROCEDURES

All speed observations were made with an S-1 model radar speed meter operated from 6-volt batteries and periodically checked to assure its precision and sensitivity. During speed surveys its accuracy was also checked with three tuning forks with frequencies which represented 30, 50, and 70 mph.

Distances from the edge of the pavement to the speed meter antenna and from the antenna to the edge of the observer's vehicle were measured for each set-up. As nearly as possible these distances were held constant for all surveys, being specifically kept constant for surveys at the same site.

IHR-53 Vehicular Speed Regulation

24-HOUR SURVEYS

Time:____________________(DAY) (NIGHT) OBSERVATIONS Date:____________ Page_______ of___

Site ____________________ (ONE) (BOTH) DIRECTIONS Weather:_____________ Observer_________

N W	NO	PASSENGER ILL	PASSENGER FOR	TRUCKS L	TRUCKS M	TRUCKS H	BUS	TIME	N W	NO	PASSENGER ILL	PASSENGER FOR	TRUCKS L	TRUCKS M	TRUCKS H	BUS	TIME
	0									50							
	1									51							
	2									52							
	3									53							
	4									54							

Figure 3. Field data sheet for 24-hr surveys.

To disguise the radar antenna as much as possible and still keep it operative, a plastic container resembling a rural mail box was constructed. Observations are possible in either direction from this unit. Figure 2 shows the general location of the vehicle in relationship to the disquised radar antenna unit.

When speed surveys were made, the following general precautions were followed:

1. Speeds were observed during good weather conditions for a continuous time period.
2. Surveys were conducted on Tuesdays, Wednesdays, and Thursdays (except for daily variation studies).
3. Days providing unusual traffic characteristics were avoided (such as holidays, special events in nearby towns, etc.).
4. The speeds of vehicles which turned off or onto the highway, or slowed purposely within the immediate vicinity of the survey site, were not recorded.

A special field data sheet (Fig. 3) was developed to facilitate recording the required information, as well as for coding the data for analysis with electric business machines and a digital computer. All studies were conducted by two persons—an observer and a recorder. The time that each vehicle passed was recorded by 5-min increments, together with the speed, direction, and type of vehicle. Vehicles were classified as freeflowing, passing, or tailgating, and listed according to the following categories:

1. Illinois passenger.
2. Foreign (out-of-state) passenger.
3. Light trucks (pickups, panels, etc., all two-axle four-wheeled trucks).
4. Medium trucks (two axles, dual rear wheels).
5. Heavy trucks.
6. Buses (commercial differentiated from other types).

After completion of the pilot surveys, the preceding procedures were adapted to three types of speed surveys. Although in some instances deviations were necessitated because of inclement weather, the following surveys were conducted:

1. 24-hour surveys. These surveys were conducted for a 26-hour period extending from 1 hour prior to sunset to approximately 1 hr past sunset the next day. The additional time provided extra data for graphing purposes.
2. Daily variation studies. These surveys consisted of observations of speeds at times representing morning rush, morning non-rush, afternoon non-rush, afternoon rush, and evening periods. Starting times were designated after studying the 24-hr survey data. Observations were made of all vehicles (including non-freeflowing) until 150 passenger vehicle speeds were recorded. Surveys were made each day for a week; any periods which were rained out were then repeated.
3. Monthly variation studies. These surveys were conducted at site 5-3 starting at 9:30 a.m. and extending until 350 speed observations were recorded. An analysis

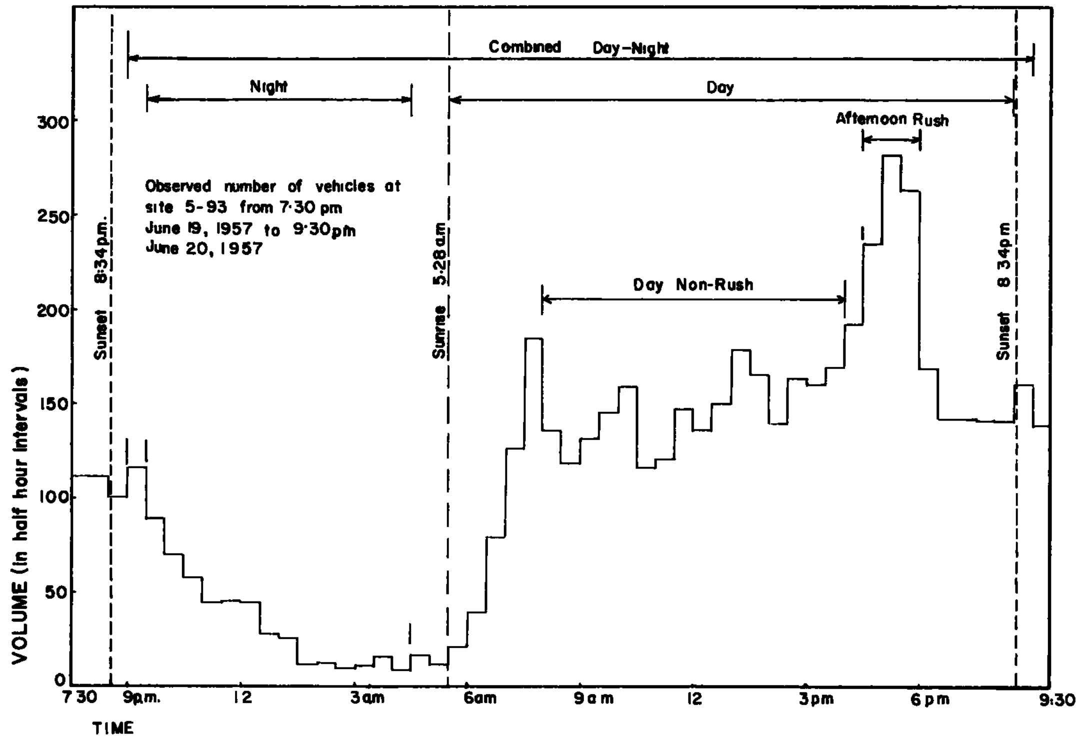

Figure 4. Designation of time periods at site 5-93.

of data from the first 24-hr survey at site 5-3 indicated that samples taken between approximately 9:00 a.m. and noon provided speed values most representative of the 24-hr speed values. Project personnel decided that a 350-vehicle sample would provide sufficient data to permit analyses of several vehicle classifications of sample sizes large enough to provide acceptable accuracy. These surveys were conducted every fourth week on Tuesday, Wednesday, or Thursday. Tuesday is the preferred day because most of the monthly variation studies to date have been made on that day of the week.

DISCUSSION OF 24-HOUR SURVEYS

The first tentative result of the 24-hr surveys was the minimum size of sample necessary to provide acceptable accuracy for practical application. In studying this question, it was felt that a major consideration was the type of traffic to be studied. This consideration not only includes different vehicle classifications but also different time periods during the entire study period of 24 hr.

The preliminary analysis presented in this paper consists of only one vehicle classification--all vehicles. Future analyses will be conducted to determine the characteristics of all freeflowing vehicles, passenger vehicles, freeflowing passenger vehicles, and commercial vehicles.

The entire 24-hr period was considered in the preliminary analysis, whose objective has been to determine the speed characteristics on rural highways for the entire 24-hr period and to develop tentative sampling procedures for the determination of these values with a shorter speed survey.

Future studies will consider using different time periods within the entire 24-hr period. To determine these various periods, two criteria will be used--the amount of daylight and the traffic volume. In differentiating between day and night travel, project personnel feel that it will be desirable to eliminate the periods of dusk and dawn because they are transition periods. (The sum of the day vehicles plus the night vehicles will not equal all the vehicles observed during the 24-hr period.) Thus, the following six time periods will be the major classifications for which analyses are desirable.

1. Day (time interval from the nearest half hour after sunrise to the nearest half hour before sunset).
2. Night (time interval from the nearest half hour after sunset to the nearest half hour before sunrise).
3. Combined (the entire 24-hr period).
4. Morning rush period.
5. Afternoon rush period.
6. Day non-rush period.

The last three time periods are chosen from within the day time interval and according to a graph of the half-hour volume variation observed at the site.

An example of the designation of time periods is indicated in Figure 4, which presents the volume data obtained in June 1957 for site 5-93. Unlike other sites, the morning rush period here was almost non-existent as far as volume differentiation is concerned. In such instances, this period cannot be analyzed separately unless additional criteria are established for designating time periods.

After designation of time periods, efforts will be made to determine the size of the sample for each period and for each vehicle classification which will provide speed values most nearly representative of those for the entire period. The analysis presented in this paper is for the combined period, which includes the entire 24-hr period. The same procedure as used for this analysis will be used for the individual time periods and all of the vehicle classifications.

Analyses of the pilot studies indicated that a constant sample size was apparently undesirable for sampling speeds on routes with different volumes. For all except rush

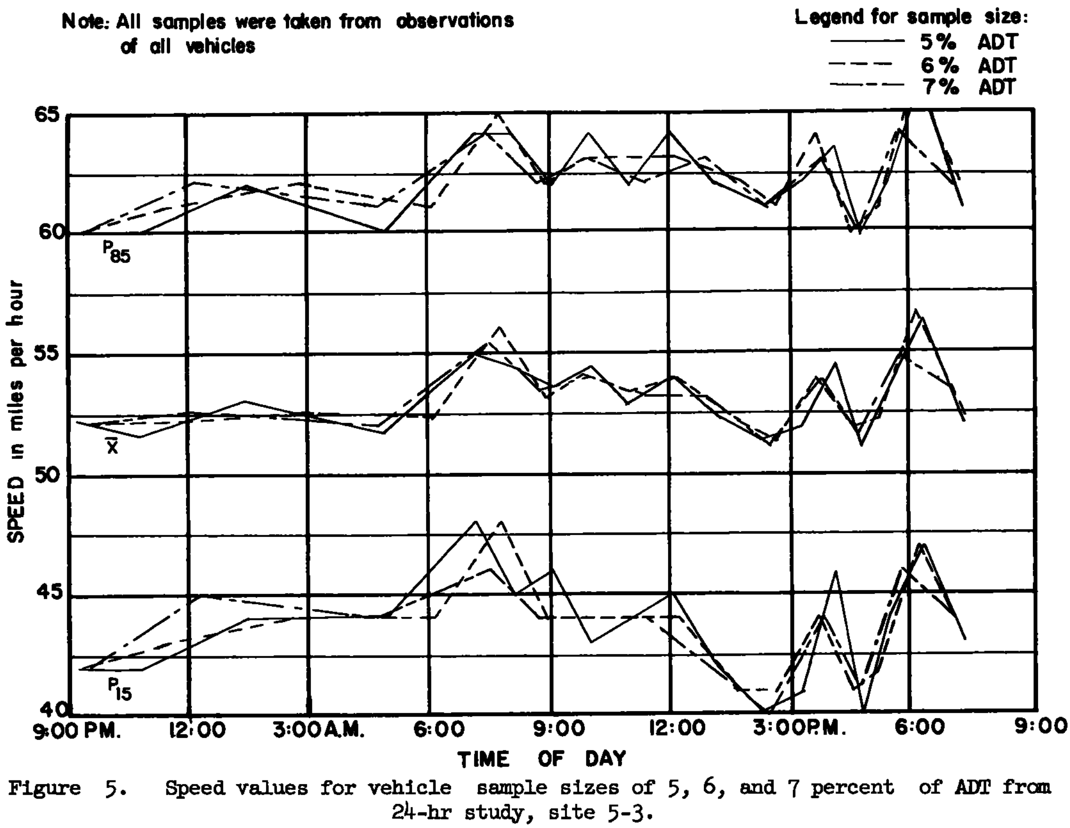

Figure 5. Speed values for vehicle sample sizes of 5, 6, and 7 percent of ADT from 24-hr study, site 5-3.

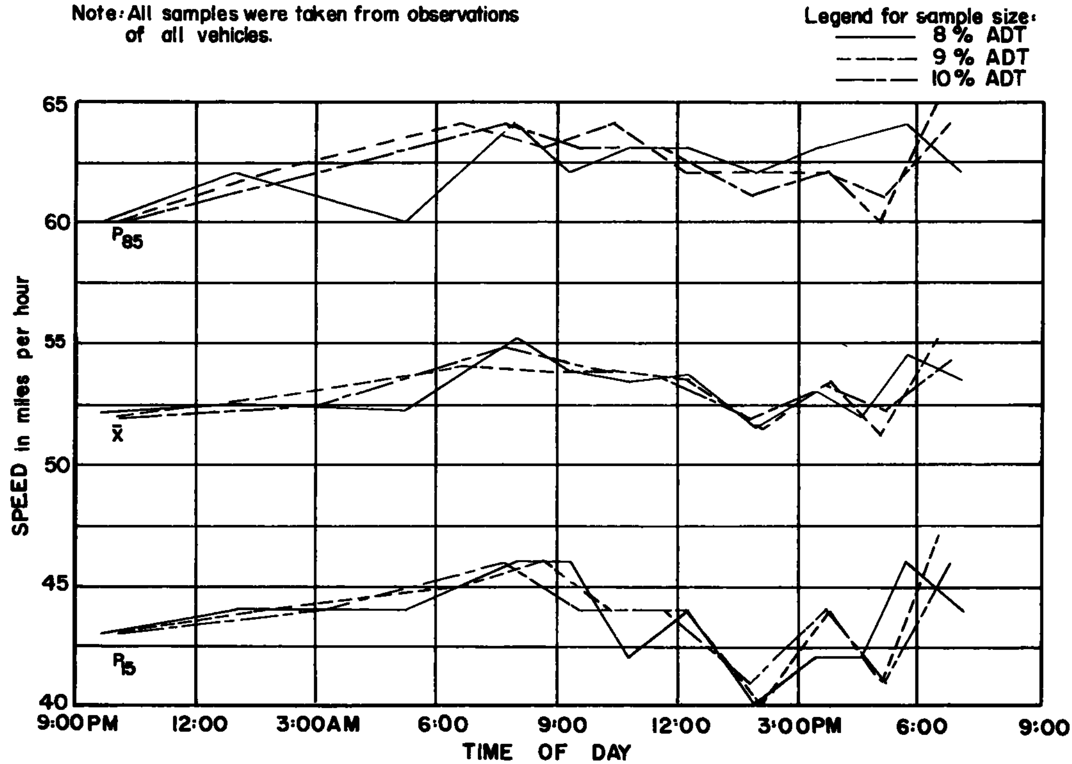

Figure 6. Speed values for vehicle sample sizes of 8, 9, and 10 percent of ADT from 24-hr study, site 5-3.

periods it appeared that the best sample size would probably be a function of the estimated ADT at the site for the year in which the study is made. Following this premise, sample sizes of 5, 6, 7, 8, 9, and 10 percent of the estimated ADT were investigated. For purposes of analysis, these values were rounded off to the nearest 25 vehicles. Thus for site 5-93, with an ADT estimate of 4,800 for 1957, the following size samples were used: 250, 300, 325, 375, 425, and 475.

A general procedure was followed for the vehicle classification studied in an effort to determine the best size of sample and time period for sampling. The first step was to determine the mean and percentile speed values which would be obtained from an analysis of all vehicle speeds for that particular time period (the entire 24-hr period, the morning rush period, etc.). These are designated "population speed values." The second step was to determine the mean and percentile speed values for each sample size to be studied. Then the absolute numerical difference (in miles per hour) of each sample speed value from its respective population speed value was determined. Graphs of these absolute numerical differences versus time were then used in selecting the most representative sample sizes and the best times for sampling.

One other important item was the method of selecting the number of samples of any constant size to be taken from a given time period for graphing purposes. For instance, from 1,000 speed observations during a given time period, ten samples of 100 vehicles each could be made by starting the first sample with the first vehicle, the second sample with the 101 vehicle, the third sample with the 201 vehicle, etc. A second method would be to start the first sample of 100 with the first observed speed, the second sample with the second observed speed, etc., until 901 samples of 100 vehicles each were formed.

The second method, of course, will give the exact indication of what speed values would have been obtained from a random sample of constant size started at any instant

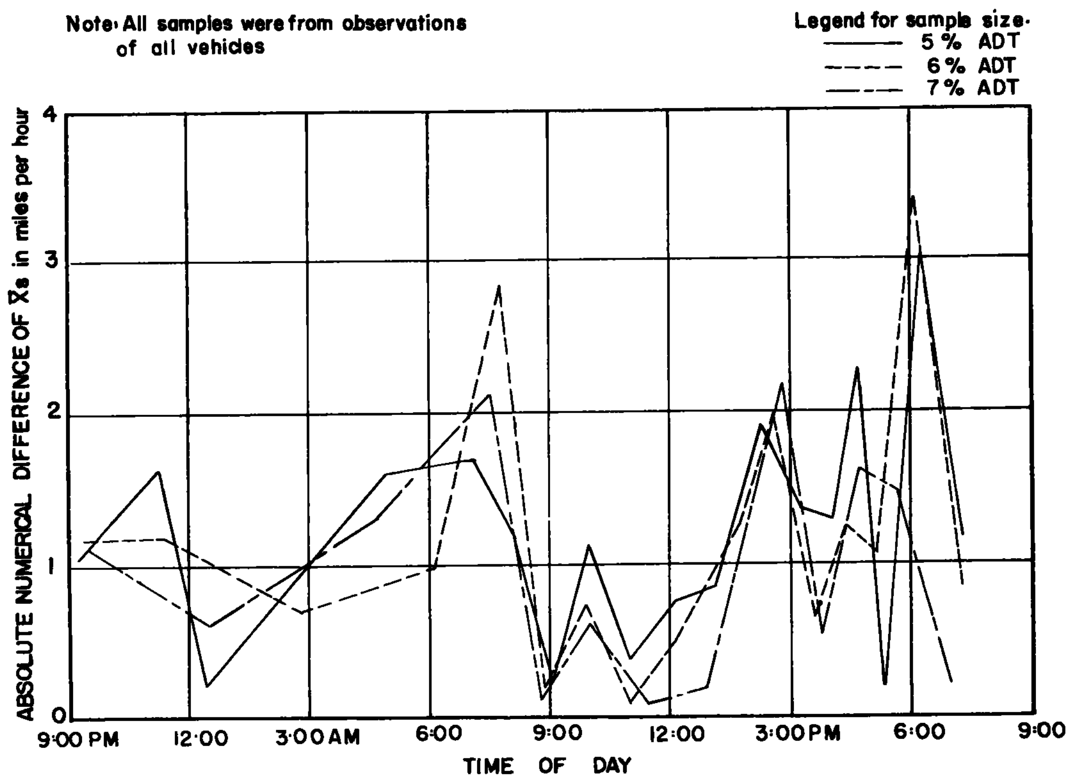

Figure 7. Speed values for the absolute numerical difference between the sample mean and the population mean speed for sample sizes of 5, 6, and 7 percent of ADT, site 5-3.

during the time period considered. However, this exactness is not only unnecessary but also very time consuming. A compromise between this and the previous method was therefore desirable.

Due to programing difficulties for the digital computer and the resulting lack of time, the first method of running samples consecutively instead of any overlapping of vehicles was the method used in the analysis presented in this paper.

For future analysis project personnel have decided to begin their consecutive samples at intervals of approximately 50 vehicles. For most routes this method has indicated that sample spacings of from 10 min to approximately 30 min will result, depending on the time of day or night and the traffic volume of the route.

The second question to be answered tentatively from the 24-hr surveys was: "When should samples be taken, within each time period, in order to best represent that time period's population speed values?"

The method of determining the best time period and sample size is determined by studying the absolute numerical differences (deviations) from the population speed values. This is accomplished by using graphs with the absolute difference plotted against time.

DISCUSSION OF 24-HOUR SPEED SURVEY ANALYSIS

In the analysis of all vehicles for the four 24-hr surveys, some basic trends seem to be apparent. Figures 5 and 6 show the speed variations observed at site 5-3. The speed fluctuations for the mean, 15th percentile, and 85th percentile speeds throughout the entire 24 hr may be observed in these figures.

Although the speed values observed at the four individual sites were not the same, the basic trends appear to be somewhat similar. The length of time required to observe

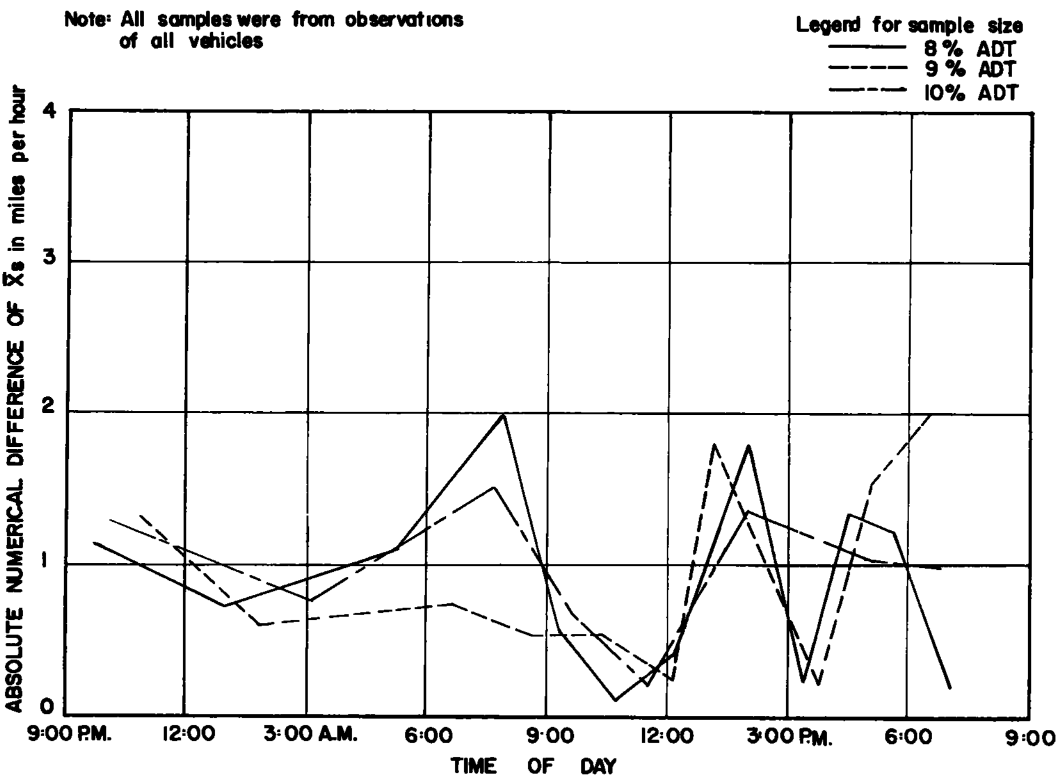

Figure 8. Speed values for the absolute numerical difference between the sample mean and the population mean speed for sample sizes of 8, 9, and 10 percent of ADT, site 5-3.

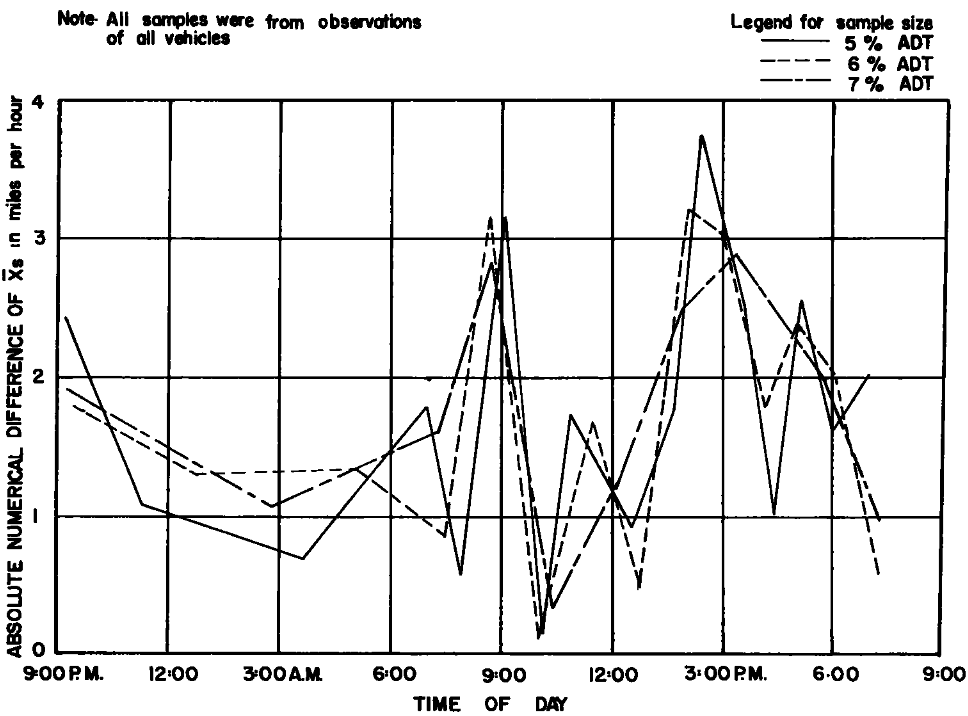

Figure 9. Speed values for the absolute numerical difference between the sample mean and the population mean speed for sample sizes of 5, 6, and 7 percent of ADT, site 5-94.

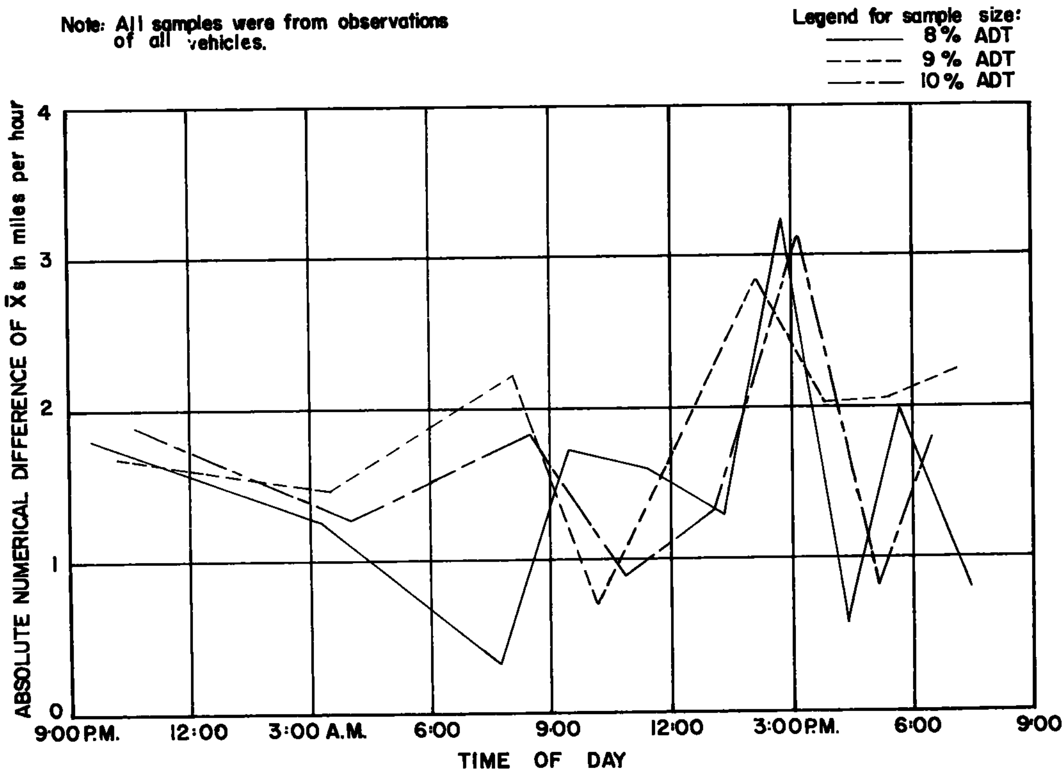

Figure 10. Speed values for the absolute numerical difference between the sample mean and the population mean speed for sample sizes of 8, 9 and 10 percent of ADT, site 5-94.

a designated sample size increases to a maximum during the morning hours immediately following 1 a. m. The values for the mean, 15th percentile, and 85th percentile speeds remained somewhat constant for all sites through the period from 8:30 p. m. to approximately 7 a. m. At this time, the speed values increased for all sites and then started to decrease shortly after 8:30 a. m. This decrease of speeds was apparent at all sites. Sites 5-3, 5-92, and 5-94 tended to have a general decrease in speeds until approximately 3 p. m., when the vehicle speeds again increased for approximately 45 min. They then decreased again for about 45 minutes and then increased.

Site 5-93 reached the minimum values shortly after 9 a. m. The speeds then gradually increased until approximately 3 p. m., when a more rapid increase in speeds was observed. The rapid increase in speeds continued until 6 p. m. After 6 p. m., a slight decrease in speeds was observed for all four sites.

An attempt to find an apparent trend in the speed differential during the various time periods was unsuccessful from this analysis. There did not seem to be any basic trend of the numerical difference between the 15th percentile and 85th percentile speeds observed. The range of speed differentials are as great within any given period of similar speed activity as between the different periods. Therefore, it appears that the 15th and 85th percentile speeds vary as the mean speed varies.

Figures 7, 8, 9, and 10 show the absolute numerical difference of the sample means from the population mean (24 hour). Figures 7 and 8 indicate the absolute difference of the sample means from the population mean for various sample sizes at site 5-3. Figures 9 and 10 show similar deviations for site 5-94. These deviations indicate the time periods with speed values most representative of the population speed values. For instance, at site 5-3 the period 11:00 a. m. to 12:00 noon seems to be best for sampling to represent the speed values for all vehicles for the entire 24-hr period. For the sample sizes shown, the absolute difference was less than 0.5 mph.

TABLE 2

CONFIDENCE INTERVAL LIMITS FOR VARIOUS PERCENTILES

Sample Size	Desir. Conf Level	Confidence Interval Limits											
		P_{85}		Actual Conf. Level	P_{75}		Actual Conf. Level	P_{50}		Actual Conf. Level	P_{15}		Actual Conf. Level
		i	j		i	j		i	j		i	j	
50	0.90	39	48	0.923	32	43	0.926	19	31	0.908	3	12	0.923
	0.95	38	48	0.956	32	42	0.952	18	32	0.951	3	13	0.956
	0.99	36	50	0.994	30	46	0.992	16	35	0.993	1	15	0.994
100	0.90	80	92	0.906	68	83	0.918	42	59	0.911	9	21	0.906
	0.95	79	94	0.956	67	84	0.951	40	60	0 954	7	22	0.956
	0.99	76	95	0.992	65	89	0.990	37	63	0.991	6	25	0.992
150	0.90	121	136	0.913	104	122	0.911	65	86	0.914	15	30	0.913
	0.95	120	138	0.955	103	124	0.952	63	88	0.959	13	31	0.955
	0.99	117	140	0 990	100	129	0 991	59	91	0.991	11	34	0.990
200	0.90	162	179	0.913	140	161	0.914	88	112	0.910	22	39	0.909
	0.95	161	181	0.952	139	164	0.956	86	114	0.952	20	40	0.952
	0.99	158	185	0.990	135	168	0.992	82	119	0.991	16	43	0.990
300	0.90	245	266	0.911	213	238	0.905	136	165	0.906	35	56	0 911
	0.95	243	268	0.957	210	240	0.950	133	167	0.950	33	58	0.957
	0.99	239	271	0.991	207	247	0.991	128	173	0.991	30	62	0.991
400	0.90	329	353	0.907	286	315	0.906	184	217	0.901	48	72	0.907
	0.95	326	355	0.958	283	317	0.950	180	220	0.954	46	75	0.958
	0.99	322	359	0.991	278	323	0.991	174	226	0.991	42	79	0.991

Thus, from the analysis of all vehicles for the entire 24-hr period, there has been some indication that there are certain periods of the day when a sample may be taken in order to closely approximate the speed values for the entire period. This period of time was not the same at all four sites. Therefore, a more detailed study must be made, which will include further analyses, as well as the selection and study of more sites.

DETERMINATION OF SAMPLE SIZE

The sample size required to meet acceptable accuracy cannot be determined from the limited analyses to date. However, a table has been developed to aid in the determination of necessary sample sizes.

Table 2 may be used to determine confidence interval limits for various sample sizes ranging from 50 to 400 vehicles. These confidence interval limits were developed through use of mathematical tables of the cumulative binomial probability distribution and show the range in which various percentile speeds will fall with a given confidence.

Table 2 also may be used to determine the range in which a speed value will occur at a given location for samples taken at the same time of day, on the same day of the week, and under the same conditions. The limits of this range in speed values (confidence limits) are given for 90, 95, and 99 percent certainty. The desired percentile speed will fall between the ith and jth observations given in the table after the speeds are arrayed in ascending order.

Use of Table 2 is illustrated by the following example: Assume a sample size of 200 vehicle speeds has been observed and the 85th percentile speed with 95 percent confidence is desired. The speeds of the 200 vehicles are then arrayed in ascending order. The true 85th percentile speed at that location (for all samples taken at the same time of day, on the same day of the week, and under the same conditions) will, with 95 percent confidence, lie between the 160th and 180th speed values.

It is hoped that the application of this table of confidence interval limits to field observations will result in another table which will enable one to determine the size sample necessary to prevent exceeding a specific range in miles per hour between the ith and jth values for a stipulated confidence. This range is called a "tolerance" in Table 3. This table must be developed from many field observations and for all types of highways. Future plans are to develop such a table for rural, intermediate, and urban areas.

An example of the table which might be developed is illustrated in Table 3, in which the values given are for illustrative purposes only and are not to be taken as actual or recommended values.

DISCUSSION OF DAILY VARIATION STUDIES

From the data obtained by daily variation studies, detailed statistical analyses were performed in an effort to determine the consistency of speed values at a given site over a week. Even after determining desirable sample sizes and times for speed observations from 24-hr surveys, one must know how consistent the speeds are from day to day at a given site before he can scientifically take samples to represent the various speed characteristics.

Tables 4 and 5 present some results of a daily variation study at site 5-3. The value for each time period for each day (a cell) is the arithmetic mean of 150 speed observations. These tables indicate considerable variation between the values. However, such variations are possibly due to several sources, as follows:

1. Variation due to random sampling. If one chooses from a very large group of objects (that is, a population) 35 samples of comparatively small size, any descriptive

TABLE 3

EXAMPLE OF TABLE THAT MAY BE DEVELOPED TO DETERMINE SAMPLE SIZE FOR REQUIRED CONFIDENCE

Desired Confidence	Tolerance (mph)	Size of Sample To Use[1]		
		Urban	Rural	Freeway
0.90	2			200
	3		200	100
	4		100	
	6			
	8			50
	10		50	
0 95	2			200
	3			
	5		200	100
	7		100	
	10			50
	14		50	
0.99	4			200
	5			
	6		200	100
	8		100	
	11			
	14			50
	17		50	

[1]Sample sizes shown are based on preliminary observations and are not to be taken as actual or recommended values. Urban areas and sample sizes in rural areas greater than 200 have not as yet been studied. There has not been any distinction made for the various percentiles

TABLE 4

MEAN SPEEDS OF ALL VEHICLES, SITE 5-3

Time Period	Mean Speed (mph)							
	Tues 7-9-57	Wed. 7-10-57	Thurs. 7-11-57	Fri. 7-12-57	Sat. 7-13-57	Sun. 7-14-57	Mon. 7-15-57	All Days
Morning rush	52.96	52.91	53 91	53.85	52.87	53.11	54.40	53.43
Morning non-rush	51.31	52.46	52.18	51.91	52.51[1]	52.37	51.59	52.05
Afternoon non-rush	50.79	52.85	53.04	49.72	52.05	52.29	51.96	51.81
Afternoon rush	52.29	52 63	53.32	53.62	52.99	51.67	54.22	52.96
Night	49 99	50.10	49.86	49.74	51.09	49.31	49.68	49.97
All	51.47	52.19	52.46	51.77	52.30	51.75	52.37	51.95

[1]Original sample period rained out; data from study on July 20, 1957.

TABLE 5

MEAN SPEEDS OF ALL PASSENGER VEHICLES, SITE 5-3

Time Period	Mean Speed (mph)							
	Tues. 7-9-57	Wed. 7-10-57	Thurs. 7-11-57	Fri. 7-12-57	Sat. 7-13-57	Sun. 7-14-57	Mon. 7-15-57	All Days
Morning rush	54.01	53.61	55.05	55.39	53 53	53.61	55.17	54.29
Morning non-rush	52 63	53.92	53.69	53.71	52.65[1]	52.79	52.51	53.13
Afternoon non-rush	52.11	54.11	54 63	50.79	52.13	52.60	52.90	52.75
Afternoon rush	53.87	53.31	54.10	54.25	53.44	51.90	54.87	53.68
Night	50.61	50.99	49.99	49.87	51.52	50.13	51.26	50.62
All	52.75	53.06	53.60	52.81	52.65	52.21	53.33	52.75

[1]Original sample period rained out; data from study on July 20, 1957.

data based on these samples will vary from sample to sample simply because each is based on a different set of observations. Each of the samples of 150 vehicles whose arithmetic mean speeds are presented in Tables 4 and 5 is a sample from a population of all possible speeds which might be observed. For this reason, some variation from cell to cell would be expected in each table. If sampling fluctuation is the only factor responsible for variation, one would expect this variation to be random in nature, the "true" speed values being underestimated and overestimated equally often. In assessing the effects of other variables on speed, one essentially asks if all of the observed variation is random or if certain systematic influences on the variation can be detected.

2. Variation due to day of the week. Each of the mean speeds in Tables 4 and 5 is based on a group of observations which is a sample not only of all possible observations of speed but also of observations representing all possible days on which observations might be made. Each table also contains the mean speeds observed on different days of the week. Again, these values are seen to vary. If day of the week has no effect on speed, this variation in column values would be random.

3. Variation due to time of day. The combinations of all seven samples whose means are represented in each row of Tables 4 and 5 may be thought of as a sample of a particular time period, taken across days. Again, the means of these samples vary. If time period does not affect speed this variation in row values would be random.

4. Variation due to interaction of time and day. It is possible that all of the non-random variation in the speeds observed in this study is attributable to the effects of hour of the day and day of the week. Another possibility, however, is that certain time periods and certain days interact to produce varied speeds. For example, it is possible that the morning rush period on a weekday will produce speeds different from

TABLE 6

STATISTICAL SIGNIFICANCE OF MEAN SPEED VARIATIONS FOR ALL VEHICLES, SITE 5-3, DAILY VARIATION STUDY

ENTIRE 7-DAY PERIOD Time Period	Tues.	Wed.	Thur.	Fri.	Sat.	Sun.	Mon.	All 7 Days
Morning rush	←			Not significant			→	HIGHLY SIGNIFICANT
Morning non-rush	←			Not significant			→	
Afternoon non-rush	←			HIGHLY SIGNIFICANT			→	
Afternoon rush	←			SIGNIFICANT			→	
Night	←			Not significant			→	
All	←			SIGNIFICANT			→	

NON-WEEKEND DAYS Time Period	Tues.	Wed.	Thur.	All 3 Days
Morning rush	←	Not significant	→	HIGHLY SIGNIFICANT
Morning non-rush	←	Not significant	→	
Afternoon non-rush	←	HIGHLY SIGNIFICANT	→	
Afternoon rush	←	Not significant	→	
Night	←	Not significant	→	
All	←	HIGHLY SIGNIFICANT	→	

those, say, of the afternoon non-rush period on a non-weekday, even though the variation between these two days as a whole or between these two time periods across days is not significant. For instance, although non-random variation across columns of Table 4 is attributed to the influence of days, and non-random variation across rows is attributed to the influence of time periods, further variation across individual cells is possible. This variation is attributed to the interactions of day and time period.

5. Variation within classes. The foregoing sources of variation all deal with variation of summary values across cells of Tables 4 and 5. Each of these summary values is based on 150 observations, which, of course, vary among themselves. This variation of individual observations within samples of 150 is referred to as variation within classes. The variation within a particular class is not affected by variation in time and day, because these variables are constant for a particular sample. This variation, therefore, can serve as a standard for evaluating the effects of day, time of day, and interaction. If the variation of individual speeds observed on different days, for example, is not greater than that of the speeds within a single class, this is evidence that day has no effect on speed.

The conventional statistical technique for determining the effect of several variables on a set of observations is known as analysis of variance. In general terms, it consists of sorting out the sources of variation already discussed and determining whether the variation attributable to a particular variable is great enough to indicate that that variable produced a non-chance effect on the observations. An index of variation is obtained for each hypothesized source of variation (that is, days, time periods, and interaction). A similar index is obtained for within-classes variation. The ratio of the variation index for the variable under consideration to that for within-classes variance affords a means of assessing the significance of the effect of the variable under

TABLE 7

STATISTICAL SIGNIFICANCE OF MEAN SPEED VARIATIONS FOR ALL PASSENGER VEHICLES, SITE 5-3, DAILY VARIATION STUDY

<table>
<tr><td colspan="8">ENTIRE 7-DAY PERIOD</td><td rowspan="2">All 7 Days</td></tr>
<tr><td>Time Period</td><td>Tues.</td><td>Wed.</td><td>Thur.</td><td>Fri.</td><td>Sat.</td><td>Sun.</td><td>Mon.</td></tr>
<tr><td>Morning rush</td><td colspan="7">← HIGHLY SIGNIFICANT →</td><td rowspan="5">↑ HIGHLY SIGNIFI-CANT ↓</td></tr>
<tr><td>Morning non-rush</td><td colspan="7">← Not significant →</td></tr>
<tr><td>Afternoon non-rush</td><td colspan="7">← HIGHLY SIGNIFICANT →</td></tr>
<tr><td>Afternoon rush</td><td colspan="7">← HIGHLY SIGNIFICANT →</td></tr>
<tr><td>Night</td><td colspan="7">← Not significant →</td></tr>
<tr><td>All</td><td colspan="7">← HIGHLY SIGNIFICANT →</td><td></td></tr>
<tr><td colspan="4">NON-WEEKEND DAYS</td><td rowspan="2">All 3 Days</td></tr>
<tr><td>Time Period</td><td>Tues.</td><td>Wed.</td><td>Thur.</td></tr>
<tr><td>Morning rush</td><td colspan="3">← Not significant →</td><td rowspan="5">↑ HIGHLY SIGNIFI-CANT ↓</td></tr>
<tr><td>Morning non-rush</td><td colspan="3">← Not significant →</td></tr>
<tr><td>Afternoon non-rush</td><td colspan="3">← HIGHLY SIGNIFICANT →</td></tr>
<tr><td>Afternoon rush</td><td colspan="3">← Not significant →</td></tr>
<tr><td>Night</td><td colspan="3">← Not significant →</td></tr>
<tr><td>All</td><td colspan="3">← SIGNIFICANT →</td><td></td></tr>
</table>

consideration. The significance of this ratio, known as the F-ratio, depends on the number of cases observed and the number of categories into which the variable under consideration is grouped.

The meaning of an observed F-ratio depends on its statistical significance. Suppose one is investigating the effect of day of week on speed. If such an effect indeed exists, one would expect the F-ratio to be larger than 1.00, because the variation due to days would be greater than the variation within classes. The question arises as to how much greater than 1.00 must the ratio be to enable one to conclude that day of week has an effect on speed. The answer is expressed in terms of the probability of obtaining by chance a value of the F-ratio as great as or greater than the one obtained.

If the chances are small, say less than 5 in 100, one may conclude that he cannot safely maintain that day of the week has no effect. Thus, the observed variation can be called "significant" in such an instance. As previously mentioned, the chances of obtaining such a value depend on the number of cases and the number of groups of observations.

TABLE 8

SUMMARY OF STATISTICAL ANALYSES OF DAILY VARIATION STUDIES, SITE 5-3

Comparison	Observed F-Ratio: All Vehicles	Observed F-Ratio: All Passenger Vehicles
(a) All 7 Days		
For all time periods:		
Between days	2.63[1]	3 84[1]
Between time periods	41.63[2]	52.70[2]
Interaction	2.85[2]	2.82[2]
Within individual time periods		
Morning rush	1.51	3.09[2]
Morning non-rush	1.94	1.29
Afternoon non-rush	5.60[2]	6.60[2]
Afternoon rush	2.50[1]	3.30[2]
Night	0.92	1 46
(b) Non-Weekend Days		
For all time periods:		
Between days	4.94[2]	3.91[2]
Between time periods	17.61[2]	22 99[2]
Interaction	1.13	1.93
Within individual time periods:		
Morning rush	1.50	2.87
Morning non-rush	1.26	1 03
Afternoon non-rush	8.95[2]	6.40[2]
Afternoon rush	0.91	0.58
Night	0.51	0.88
Weekend vs Non-Weekend Days	0.90	8.59[2]

[1]Significant at 0.05 level.
[2]Significant at 0.01 level.
Note: Direct comparison between F-Ratios for the "All 7 Days" grouping and the "Non-Weekend Days" grouping is not possible due to different sample sizes.

In this paper, if the chances are less than 1 in 100 that a result as extreme as or more extreme than the one observed would have occurred by chance, the result is said to be "significant at the 0.01 level," or "highly significant;" if the chances are less than 5 in 100, but greater than 1 in 100, the result is said to be "significant at the 0.05 level," or "significant;" if the chances are more than 5 in 100, the result is said to be "not significant." (The usage of 0.01 and 0.05 levels of significance is a conventional statistical tool and has no special statistical justification.)

DAILY VARIATION STUDY AT SITE 5-3

The results of the statistical analysis for all vehicles is give in Table 6; for all passenger vehicles, in Table 7. Table 8 summarizes the observed values of the F-ratios representing the various hypothesized sources of variation and the significance attributed to the result.

The most pronounced influence on speeds, as revealed in the daily variation studies to date, is that of time periods. This influence appears when observations cover all seven days of the week and when observations are restricted to non-weekend days (Tuesday, Wednesday and Thursday). It is apparent in the speeds of all vehicles and of passenger vehicles (Tables 6 and 7). The size of the observed F-ratios enables one to conclude that the time of day has an appreciable effect on speed. This is perhaps not a surprising result, because highway and traffic engineers have usually assumed that such an influence exists.

The effect of day of week requires closer examination. When observations cover all seven days, day of the week exerts a significant influence on the speeds of all vehicles and a highly significant influence on all passenger vehicles (Tables 6 and 7). Further analysis permits a more detailed study of this influence. It is generally believed that speed characteristics on weekend days are different from those of non-weekend days.

It may be asked whether day of the week affects speeds when only non-weekend days are considered. From observations on Tuesday, Wednesday, and Thursday, the days on which speed characteristics may be assumed to be most nearly constant at a single site, it is seen that a significant influence is again revealed by the data. However, this considers all time periods during each day. (It is not possible to compare the relative sizes of the F-ratios in Table 8 for weekdays with all days because of the different sample size on which they are based.)

The effect of day of week may be further examined by analysis of speeds observed in single time intervals. Table 7 shows that with observations covering all seven days of the week, day of the week exerts an effect on the speed of passenger vehicles during every time period except morning non-rush and night. Such an effect is apparent on the speeds of all vehicles in afternoon periods (Table 6). When speeds on non-weekend days are considered, the only time interval in which day of week has a significant effect is the afternoon non-rush period. It appears, then, that most of the day effect on weekday speeds is accounted for by the afternoon non-rush period.

It is likely, however, that there are effects during other periods which, although not great enough to be significant in a single time period, increase the cumulative effect over the entire day. The results also suggest that most of the effect of the day of week on time periods, other than afternoon non-rush when observations cover all days, is due to the different speed characteristics of weekend days and non-weekend days.

The results with regard to interaction of time and day are interesting. Within weekdays, no significant interaction was apparent (Table 8). Considering all days, on the other hand, a significant interaction appeared in the speeds of all vehicles and of passenger vehicles. This indicates that over the entire week, time period has effects during some days which are different from the effects during other days; that is, day of the week and time of day interact to produce effects not produced by either of the variables considered separately. The fact that this effect does not appear during non-weekend days suggests that the major source of interaction is between time periods and weekdays vs weekend days.

Finally, the difference between speeds observed on the three weekdays and those observed on the four weekend days were analyzed. As shown in Table 8, these speeds were significantly different in the case of passenger vehicles; they were not significantly different in the case of all vehicles. This result suggests that the varying speed characteristics of weekend and non-weekend days has a greater influence on the speed of passenger vehicles than that of non-passenger vehicles.

The results of this phase of the investigation suggest that day of week and time period during the day both have a significant influence on observed speed. Statistically, one may say that speeds observed on different days, and speeds observed during different time periods, are samples from different populations. Translated into engineering terms, this would imply that speed characteristics at a given site vary according to day of week and according to time of day.

It should be borne in mind that these results apply to the present set of observations only, representing only one observation site, and made during a limited time period. Thus, variation in speed characteristics due to site conditions restricts generalization of these results. Furthermore, it is possible that the choice of week and of time periods has introduced "invisible error" into the results. It is desirable that further investigations of other sites, other weeks, etc., be performed. Several such investigations are now being conducted by this project. The present results, however, can yield interesting insights into traffic problems, and permit fruitful speculation as a guide to future research.

A further interesting speculation arises from a comparison of the results for all passenger vehicles and those for all vehicles. It may be noted in Tables 6, 7, and 8 that in most cases the effects of the variables under consideration are more pronounced on the speeds of passenger vehicles than on those of all vehicles. Although insufficient data were available for a definite conclusion, these results suggest that the greatest effects of the variables considered here are on passenger vehicles. It is possible that these variables have little or no effects on the speeds of non-passenger vehicles.

As previously mentioned, the conclusion suggested by the data obtained in this daily variation study was that day of the week and time of day both have a statistically sig-

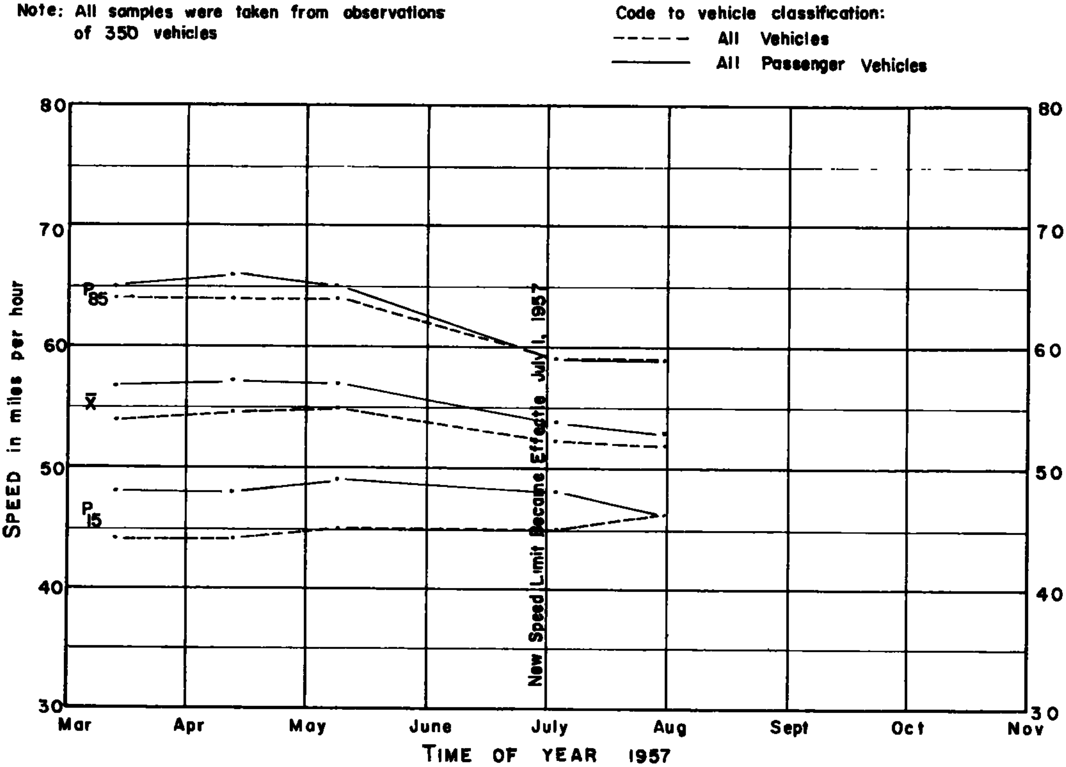

Figure 11. Speed values from monthly variation study, site 5-3.

nificant influence on vehicular speeds. The question remains as to whether these influences are large enough to greatly complicate the sampling of speeds. Tables 4 and 5 reveal that the range in the mean speed across time periods, the variable whose effect on speeds is the greatest of those considered here, is about 3½ mph. It is suggested that for practical purposes these differences are small, considering the error introduced by the very nature of sampling and the accuracy of recording equipment. From site 5-3 data it may be inferred that the effect of day and time of day on speeds, although such effects do exist, is not so great as to preclude sampling without a great deal of concern for them. However, many more studies similar to this one are needed before even tentative conclusions may be drawn.

DISCUSSION OF MONTHLY VARIATION STUDIES

One monthly variation study is in progress (at site 5-3). This study began in March 1957 and has continued with observations every fourth week whenever possible. (Weather conditions and equipment failure have sometimes necessitated postponement of these observations.)

An analysis of variance was made for all vehicles of this study. The time periods included in the analysis were March through July. No significant difference was detected between the speed values of all vehicles for the months of March through June 1957.

The analysis could not be compared with the months starting in July, due to the new Illinois numerical speed limits, which went into effect on July 1, 1957. The study is being continued and other studies will be conducted to determine if any significant variations in vehicular speeds occur from month to month.

Figure 11 shows the speed values for the mean, 15th percentile, and 85th percentile speeds at this location for the months of March through July 1957.

SUMMARY

This paper reports on research on vehicular speed regulation performed at the University of Illinois. The major activities have been development of a confidence interval limits table, establishment of survey sites, determination of the type of studies to be made, and selection of the procedures for conducting the surveys, collecting field data, and analyzing the results.

The development of the confidence interval limits was based on mathematical tables. If studies are made under the stipulated conditions, the results are mathematically reliable. The development of the table of sample sizes for required confidence must result from many field observations on various types of highways. Future plans include development of this type of table for rural, intermediate, and urban areas.

In the discussion of the analysis of the four 24-hr surveys, some apparent trends are noted. These studies are being continued.

A daily variation and monthly variation study is discussed. The method of analysis and the trends that seem apparent from these studies are included in the report. These studies are being continued, and the establishment and study of additional locations will continue.

This report is intended to be only a progress report and the findings and apparent trends are not to be taken as final. Further studies and analyses must be made in order to determine common speed characteristics that may be helpful in sampling to determine representative speed information on rural highways. In the future, similar studies will also be directed toward intermediate and urban areas.

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An Advisory Committee also aids in directing project activities. This Committee comprises Charles H. Bowman, Professor of Law, University of Illinois; D. A. Branigan, District Engineer of Traffic, Illinois Division of Highways; W. E. Chastain, Sr., Engineer of Physical Research, Illinois Division of Highways; William E. Kappauf, Professor of Psychology, University of Illinois; Warren L. Kessler, Assistant Traffic Operations Engineer, Illinois Division of Highways; and David Solomon, Highway Transport Research Engineer, U. S. Bureau of Public Roads.

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

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

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

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