HIGHWAY RESEARCH RECORD

Number 162

Geometric Aspects of Highways

5 Reports

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51	Highway Safety
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Foreword

A highway can be properly designed only after the establishment of appropriate geometric design standards that take the complexities of the driver and vehicle into account. In recent years, there has been a considerable amount of nationwide research dedicated to the improvement of geometric designs. This RECORD presents five papers that are concerned with such research.

The first paper attempts to provide a framework of knowledge so that highway designers can obtain a better perspective of the factors involved in arriving at decisions for median designs. The study deals with aspects of median width and headlight glare, effect of median appurtenances on safety, and a variety of factors involved in eliminating cross-median accidents.

The second paper describes freeway levels of service using acceleration noise as a parameter. (Acceleration "noise" is the standard deviation of vehicle acceleration, not noise in the usual sense.) Using logical concepts and analyses of Texas freeway data, the researchers indicate that the level of service concept as presented in the new HRB Highway Capacity Manual is akin to the assumption in their acceleration noise model and the results obtained therefrom.

Although freeways have existed for many years, not enough is known yet about their volumes and traffic characteristics, especially during peak traffic periods. Peak periods within the peak periods have been studied in the research reported on in the next paper, as has composition of traffic and distribution by lanes. Linear regression models for estimated peak traffic are found to be applicable. By using appropriate regression models, total freeway volumes can be estimated by use of a detector in one lane only.

The fourth paper presents a novel concept that indicates a fatality and accident rate reduction can be achieved by converting highways to freeways. The author indicates that a system of freeways, in addition to the state highway system, will not reduce accidents because the accident rate increases on the state system and the overall totals will not be less than they were before the construction of freeways.

The last paper is concerned with the use of Interstate Highway shoulders. Frequency, purpose, and duration of stop were researched. Some relationships are explored between use of shoulders and accident experience.

This RECORD should be of prime interest to highway designers and those concerned with traffic operations on highways, as well as to those concerned with freeway surveillance projects.

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Safety Considerations in Median Design

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The frequency, nature and causes of vehicle encroachments on medians of divided highways were investigated to obtain information needed in establishing traffic safety criteria for median width and cross section design. The effects of median width and cross section, traffic volume, roadway alignment, weather, roadside signs, grade separation structures and other features of the highway and driving environment were considered. Relationships between traffic volume and the frequency and nature of vehicle encroachments on medians are presented. The recommendations include minimum design requirements for safe stopping or control of vehicles in the median.

•THIS IS the final report on a six-year study of the frequency, nature and causes of vehicle encroachments on medians of divided highways. It covers one phase of a research project conducted by the University of Illinois Engineering Experiment Station as a part of the Illinois Cooperative Highway Research Program. Other phases of this work have been reported previously (1-4, 7).

The objective of this study was to determine the frequency and nature of vehicle encroachments on certain types of medians under selected field conditions, in order to evaluate the median's potential as a stopping or recovery space for erratically moving vehicles. Because the importance of designing the median as a stopping or recovery space varies with the extent to which the frequency and nature of encroachments can be controlled, particular attention was also devoted to factors causing or influencing encroachments.

STUDY METHOD

In planning the study it was assumed that conventional accident records would be of little value in determining either the frequency or nature of vehicle encroachments on medians. A subsequent comparison of accident records and project data supported this assumption (2).

The argument against the use of accident records as the primary source of data was that they gave little indication of the extent to which medians are successful as a stopping or recovery space for vehicles.

Accident records could provide measurements of the failure of the median to serve as a vehicle stopping or recovery space. However, the limited amount of police surveillance allows some undetected cross-median vehicle movements and many unreported minor collisions of vehicles with fixed objects in the median.

The study consisted of analyzing the evidence at the site of each encroachment on highway medians. This required carefully planned, frequent coverage of the entire length of the selected highway segments to locate and evaluate evidence of encroachments.

Surveillance was provided by two-man teams who patrolled the highway in specially marked slow-moving vehicles. Each encroachment record included a sketch of the

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vehicle path with dimensions, highway cross-section dimensions, type of median cover, approximate time of occurrence, and other pertinent data. A visual record of each encroachment was compiled through the use of colored and black-and-white pictures. To avoid duplicate reporting during the 3½-year study on I74, each encroachment was assigned a number which was painted on the pavement at the site.

Encroachment frequency was determined from records of every incident involving vehicle travel in the median during a chosen time interval. Intentional turns across the median and encroachments resulting from maintenance activities were not used in these calculations.

The nature of encroachments was determined from data collected at the sites, and from additional information in available accident reports. Because of the necessary accuracy and detail of the needed data, some observed locations had to be excluded from this part of the study for lack of sufficient evidence for analysis.

Factors causing or influencing encroachments were given considerable attention throughout the study. An analysis of conditions at several points of high encroachment frequency on US 66 was reported previously (3). A similar analysis of 24.6 mi of I74 is included herein.

ENCROACHMENT FREQUENCY

Because of limitations of time, money and personnel, only four-lane highways were considered in this study. Of primary concern was the development of relationships between the frequency and/or rate of encroachment and traffic volume. No attempt was made to obtain the great quantity of data necessary for development of a critical comparison of these relationships for six- and eight-lane highways.

Data for this phase of the study were obtained from I74 (from US 150 Spur, Urbana, to the US 150 Junction, Danville) and a portion of Kingery Expressway (from Calumet Expressway to the Illinois-Indiana state line, Chicago). Both of these highway segments have dual 24-ft roadways, complete control of access and essentially tangent alignment. The widths and cross sections of the medians are different; I74 has a forty-ft median, depressed about 3 ft, whereas the Kingery Expressway median is 18 ft in width and is depressed about 6 in. However, from the standpoint of factors affecting frequency, the most significant difference in the design features of these two highways is the extent to which roadway delineation has been provided (3). I74 has reflective delineators; Kingery Expressway does not. Some roadway delineation is provided by wooden cable-barrier posts in the Kingery Expressway median, but the general level of delineation is undoubtedly lower than on I74.

Traffic volumes increased from about 1,700 to 6,000 veh/day on I74 and from 18,000 to 31,000 veh/day on Kingery Expressway during the periods for which data were compiled.

Encroachment and traffic volume data for the 3-mi Kingery Expressway study section were obtained from the Illinois Division of Highways, and were for four-month periods extending from December 1 through March 31 of 1957-58, 1958-59 and 1959-60. Data for the 24.6-mi I74 study section were recorded by project personnel from October 4, 1960 through April 6, 1964. Traffic volume and encroachment data are presented in Table 1 and Figure 1.

Figures 2 and 3 show the relationship between traffic volume and the recorded encroachments. The curves in these figures have been constructed through the weighted average of I74 data points associated with traffic volumes of approximately 4200 and 5800 veh/day. This was done to decrease the effects of seasonal variations in number of encroachments for periods with approximately equal traffic volumes. The basic character of the relationships (Figs. 5 and 6) is not changed by averaging the data points (Figs. 2 and 3). Observed seasonal variations are shown in Figure 4.

Figure 2 indicates that encroachment frequency on I74 increased with traffic volume until an ADT volume of about 4,000 veh/day occurred. At higher traffic volumes the frequency decreased until a minimum was reached at about 6,000 veh/day.

The dashed line (Fig. 2) connecting the data points for I74 and Kingery Expressway shows the general nature of the change in encroachment frequency expected on I74 at

TABLE 1

ENCROACHMENT FREQUENCY AND RATE DATA

Period of Observation	Days of Observation	Traffic Volume (ADT), Veh/Day	Vehicle-Miles of Travel	Observed Encroachment	Encroachment Frequency, Enc/Mi/Yr	Encroachment Rate, Enc/100 × 10^6 Veh Mi
(a) I74 (24.6 mi)						
Oct. 4, 1960-Dec. 22, 1960	79	1,900	3,693,000	16	3.0	433
Dec. 22, 1960-March 29, 1961	97	3,000	7,159,000	31	4.7	433
March 29, 1961-July 12, 1961	105	4,000	10,332,000	58	8.2	561
July 12, 1961-Dec. 2, 1961	143	4,150	14,599,000	30	3.1	205
Dec. 2, 1961-March 31, 1962	119	4,350	12,734,000	32	4.0	251
March 31, 1962-June 26, 1962	87	5,250	11,236,000	17	2.9	151
June 26, 1962-Oct. 13, 1962	109	5,750	15,418,000	16	2.2	104
Oct. 13, 1962-April 16, 1963	185	5,950	27,078,000	50	4.0	185
April 16, 1963-June 27, 1963	72	5,950	10,539,000	5	1.0	47
June 27, 1963-April 6, 1964	284	5,700	39,822,000	47	2.4	118
(b) Kingery Expressway (3.0 mi)						
Dec. 1, 1957-March 31, 1958	120	18,195	6,550,000	7[a]	7.1	107
Dec. 1, 1958-March 31, 1959	120	20,490	7,376,000	9	9.1	122
Dec. 1, 1959-March 31, 1960	121	31,253	11,345,000	14	14.1	123

[a]Incomplete record.

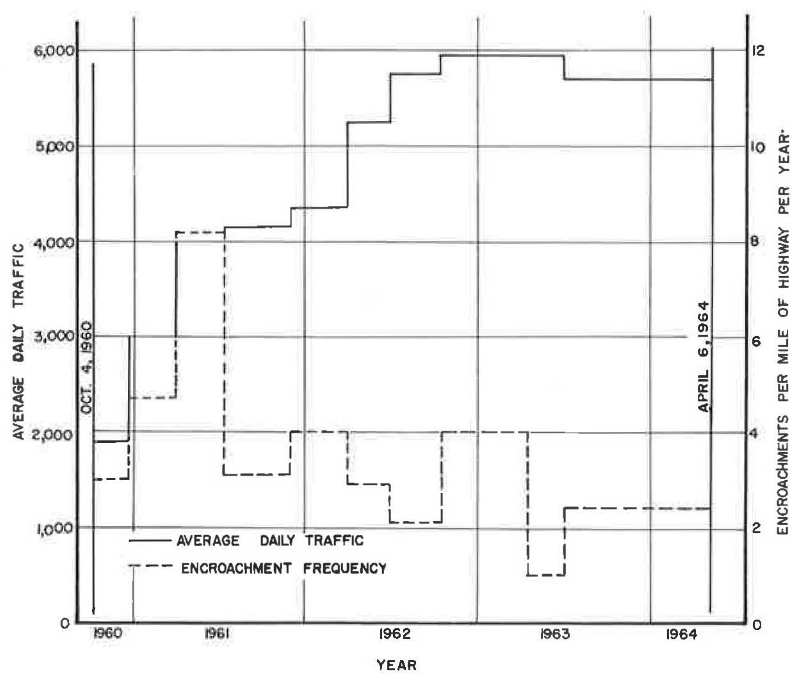


Figure 1. Comparison of variations in traffic volume and encroachment frequency for I74.

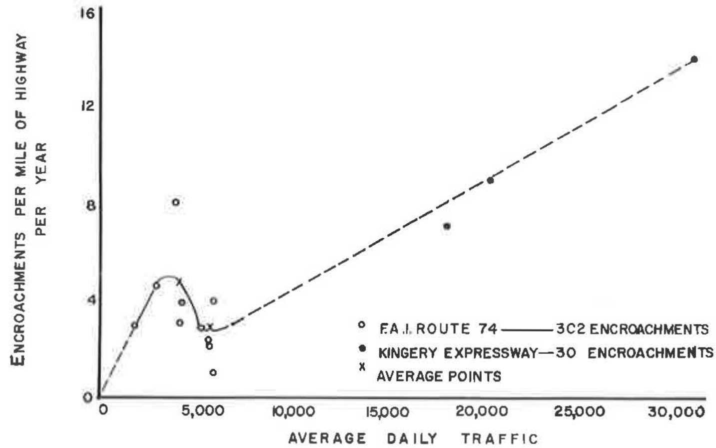


Figure 2. Encroachment frequency for I74 and Kingery Expressway.

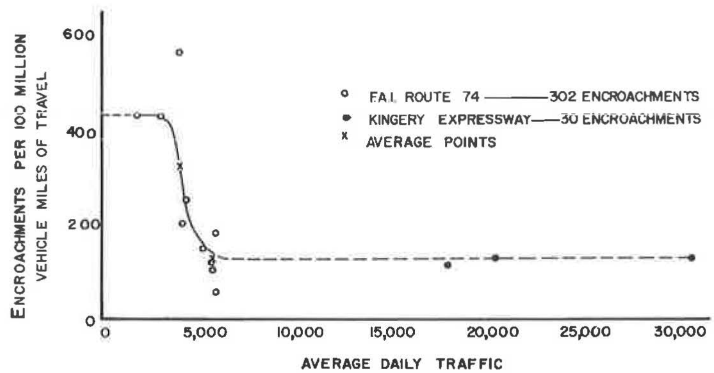


Figure 3. Encroachment rate for I74 and Kingery Expressway.

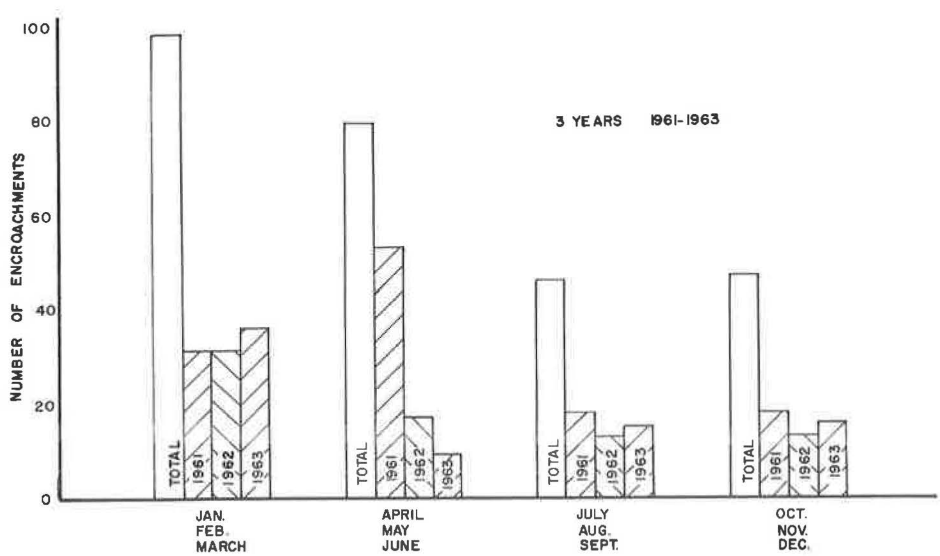


Figure 4. Seasonal variation in number of vehicle encroachments on median of I74.

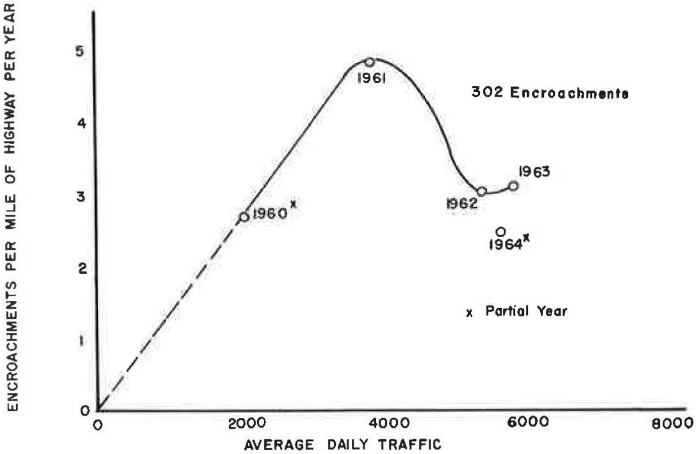


Figure 5. Yearly encroachment frequency for I74.

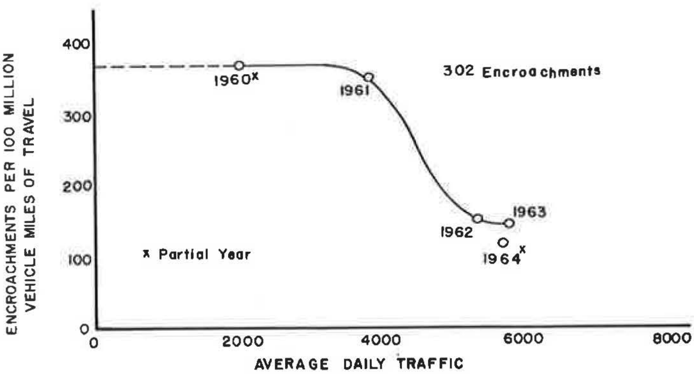


Figure 6. Yearly encroachment rate for I74.

volumes greater than 6,000 veh/day. It does not represent an extrapolation of data under the assumption that the relationship between frequency and traffic volume is not affected by differences in the design features of these highways. Previously reported findings indicate that such an assumption is not valid (4). The better roadway delineation on I74 should result in somewhat lower encroachment frequencies than are represented by the dashed line. Furthermore, the observed frequencies of encroachment on Kingery Expressway are probably high due to seasonal influences; all the data from Kingery Expressway were collected during winter months. Therefore, the dashed line in the interval from 6,000 to 20,000 veh/day is only an indication of the general shape and direction of the future volume-frequency relationship predicted for I74.

The point representing Kingery Expressway data for the winter of 1957-58 (approximately 18,000 veh/day) does not fall on the curve. This is because an incomplete record was obtained during the first winter of study on Kingery Expressway. Failure to properly coordinate field activities with the occurrence of snow storms resulted in the loss of some evidence. The data point for this period falls below the constructed curve, thus indicating a proper relative position of the portion of the curve representing Kingery Expressway data.

Encroachment rate (Fig. 3) is a function of the slope of the encroachment frequency curve. At a traffic volume of 3,000 to 4,000 veh/day the rate on I74 began to decrease rapidly with increasing traffic volume. As the traffic volume approached 6,000 veh/day the rate became relatively constant at a value less than one third of the original value. The trend of the curve at about 6,000 veh/day suggests a more or less constant rate, equal to or less than the rate for Kingery Expressway, at traffic volumes above 6,000 veh/day.

The proposed explanation of these changes in rate and frequency is based on the differences in driving environment associated with changes in traffic volume.

Drivers act more or less independently at low traffic volumes; there is extensive freedom of movement with restrictions imposed only by the physical features of the roadway. Furthermore, modern high-speed highways are designed to relieve the driver of many operational judgments and decisions associated with older two-lane highways. This environment leads to inattentiveness and reduced alertness which increase the probability of an inadvertent median encroachment. Since drivers operating at low traffic volumes can be considered as isolated units, it appears logical to assume that the probability of encroachment is constant for each vehicle and independent of the behavior of other vehicles.

Another important consideration is the reduced roadway delineation which occurs at low traffic volumes. In the absence of other vehicles the driver must orient his vehicle with respect to the physical features of the roadway which may or may not provide adequate delineation. If, however, other vehicles are present there is a tendency to caravan, with each driver consciously or subconciously following the vehicle ahead. Vehicles do, in a sense, delineate roadway alignment and provide a reference point for lateral positioning of vehicles further back in the traffic stream.

As traffic volume increases, driver-vehicle behavior is influenced to a greater and greater extent by the presence of other vehicles. The decreased spacing between vehicles should increase driver alertness and roadway delineation with a resulting reduced probability of an inadvertent encroachment. At the same time the probability of encroachment due to evasive action or actual physical contact between vehicles should increase.

Considering Figure 2, it is reasoned that, in the interval between zero and

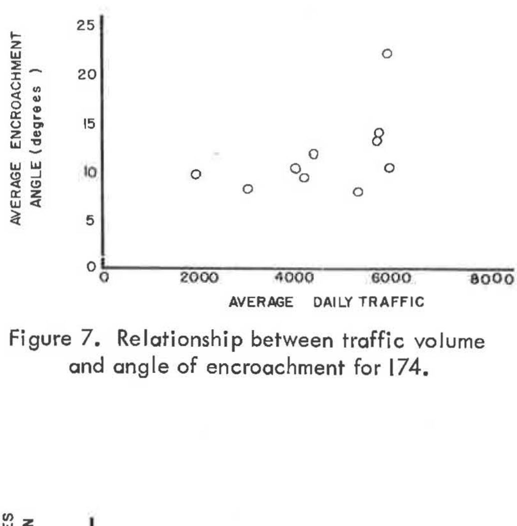


Figure 7. Relationship between traffic volume and angle of encroachment for I74.

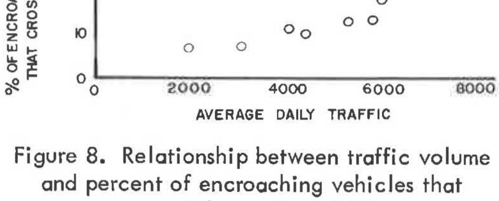


Figure 8. Relationship between traffic volume and percent of encroaching vehicles that crossed the median of I74.

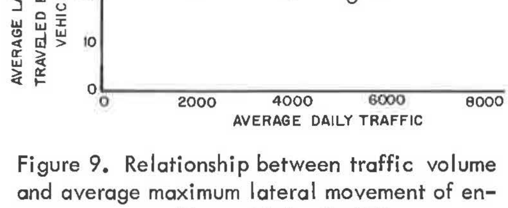


Figure 9. Relationship between traffic volume and average maximum lateral movement of encroaching vehicles for I74.

2,000 veh/day (extrapolated), the driver is operating independently of other vehicles and the probability of median encroachment is constant for each unit. Thus, the number of encroachments is primarily dependent on the number of vehicles subject to the chance of encroachment and a linear relationship should exist between frequency and traffic volume. As traffic volume increases (above 2,000 veh/day), the effect of improved roadway delineation and driver alertness is illustrated by the decreasing slope of the volume-frequency relationship; the probability of an inadvertent median encroachment is reduced and the frequency increases at a decreasing rate.

At about 4,000 veh/day the frequency of encroachment begins to decrease with increasing traffic volume; the effect of greater numbers of vehicles subject to the chance of encroachment has been more than offset by the gradual improvement in roadway delineation and the rising level of driver alertness.

However, as volume increases, there is also increasing friction and conflict between vehicles. The driver must devote an increasing share of attention to the maneuvers and positioning of other vehicles. Friction and conflict between vehicles is reflected in the more severe encroachments resulting from evasive action and/or physical contact between vehicles. The average angle of encroachment increases in the interval from 4,000 to 6,000 veh/day (Fig. 7). There is also an increase in the percentage of encroaching vehicles that cross the median (Fig. 8).

Concurrently with this increase in severity of the average encroachment, there is greater variety in the types of encroachments occurring. This is illustrated by an increasing scatter of data points in the interval from 4,000 to 6,000 veh/day (Figs. 7 and 9). The greater variety in types is a result of changes in the relative importance of many volume-related factors affecting the frequency and nature of encroachments. Most important among these factors in the increasing contrast between the types of driving environment found at different periods of the day. Inadvertent encroachments, associated with low traffic volumes, are still represented because the conditions that produce them are still prevalent during low volume periods. However, new reasons for inadvertent encroachments become evident during high volume periods of the day; the driver must often devote more attention to the relative position of nearby vehicles than to the alignment of the roadway and the lateral placement of his own vehicle. The driving task steadily becomes more complex as the traffic volume increases.

The trend of the volume-frequency curve (Fig. 2) in the interval from 5,000 to 6,000 veh/day reflects the growing complexity of the driving task and the resulting increase in encroachments that can be expected at higher traffic volumes. The dashed portion of the curve, in the interval from about 6,000 to 20,000 veh/day, shows a somewhat more rapid frequency increase than will probably be experienced on I74, but it is indicative of the anticipated linear increase.

Evidence points toward a linear increase in frequency with higher traffic volumes. This can best be illustrated in connection with encroachment rate. All the rates in Table 2 are of the same general magnitude even though a wide range of traffic volumes is represented. This suggests a more or less constant encroachment rate and a linear increase in encroachment frequency at higher traffic volumes.

The encroachment rates for Calumet and Edens Expressways were calculated from data collected in 1960 with evergreen trees installed in the medians of both expressways (4). The evergreen trees are assumed to have provided slightly better roadway delineation than is provided by the reflective delineators on I74. Therefore, the given rates for Calumet and Edens Expressways are assumed to be

TABLE 2

ENCROACHMENT RATES[a]

Highway	Rate
Calumet Expressway ADT = 12,000	58
Edens Expressway ADT = 26,000	61
Santa Ana Freeway Approximate ADT = 95,000	54
Nimitz Freeway Approximate ADT = 95,000	68

[a]Encroachments/100 x 10^6 vehicle miles.

slightly lower than can be expected on I74 at equivalent traffic volumes. The rate for higher traffic volumes on I74 is expected to be somewhere between the Kingery Expressway rate (Fig. 3) and the given rates for Calumet and Edens Expressways, i.e., between 60 and 120 encroachments per 100 million vehicle miles of travel.

Rates for the Santa Ana and Nimitz Freeways were calculated from the number of cable-chain-link fence barrier repairs reported by Moskowitz and Shaefer in connection with the study of median barrier performance on California freeways (5). It is doubtful that many vehicles were able to recover within the 6-ft half-width of the curbed medians on these freeways without damaging the barrier.

Edens Expressway and the Santa Ana and Nimitz Freeways are six-lane facilities and therefore cannot be expected to have the same traffic stream characteristics as four-lane facilities like I74 and Kingery Expressway. However, the differences in traffic stream characteristics do not appear to affect significantly the rate under the conditions of reduced vehicle headway associated with high traffic volumes. The average vehicle headway below which the encroachment rate is apparently no longer affected by increases in traffic volume is about 15 sec for I74 (8). The gap equivalent to this average headway can serve as the basis for a rough estimate of the volume above which the rate may be expected to remain relatively constant.

This gap, based on the observed 50th percentile speed of 62 mph at an ADT volume of 6,000 veh/day on I74, is about ¼ mi. The ADT volume producing an average headway equivalent to a ¼-mi gap is therefore the volume above which the rate appears to become relatively constant. In the absence of more complete data, this gap can serve as the basis for a rough estimate of the minimum ADT volume at which the rate may be expected to become relatively constant on a multilane divided highway with complete control of access. A comparison of the relatively constant rates for highways carrying equal or greater ADT volumes should provide a measure of the relative safety of the driving environment provided by different design features. The previously reported experiment with evergreen trees in the medians of Calumet and Edens Expressways was based on such a comparison of rates (4).

Data necessary for such comparisons will become more readily available in the form of median barrier repair records as the growing traffic volumes on many of our present multilane highways exceed the minimum volumes suggested as warrants for the installation of median barriers (6).

NATURE OF MEDIAN ENCROACHMENTS

One of the primary functions of the median (1) is to serve as a suitable stopping or recovery space for encroaching vehicles and yet, except for the study of median barrier performance, there has been little previous effort to determine the effects of median width and cross section on the behavior of encroaching vehicles.

Information on the lateral extent of encroachments can be used as an indication of the median width required to provide an appropriate stopping and recovery space. It can also be used as a measure of the effectiveness of the median cross section in controlling the lateral extent of vehicle movement. The length of vehicle travel during encroachment is an indication of the extent to which the median should be free of obstacles which cannot be traversed safely at normal highway operating speeds.

This analysis of the nature of encroachments concerns rural I74 and I57 and is limited to data from unintentional encroachments with lateral movements in excess of 3 ft. During the periods of study, 302 and 26 encroachments of this type were detected and recorded for I74 and I57 respectively.

In the analysis of physical parameters associated with encroachment patterns, data from less than the total number of encroachments are generally used. This procedure results because certain parameters could not be measured in all encroachments. In addition, only selected portions of the data are considered in certain portions of the analysis. The reasons for this selectiveness are explained in the discussion.

The physical characteristics of the encroachments which generally could be determined from an analysis of field evidence were the basic movement patterns (8), the angles of encroachment, and the lateral and longitudinal distances of vehicle travel.

All measurements were made with reference to the path of the left front wheel of the vehicle. The encroachment angle is defined as the angle between the pavement edge and the path of the left front tire of the vehicle as it departed from the pavement. The lateral extent of movement is defined as the perpendicular distance from the pavement edge to the path of the left front tire of the vehicle at some specified point in the encroachment pattern. The length of travel is the distance from the point at which the vehicle departed from the roadway to some specified point in the pattern as measured along the left front tire path. Table 3 summarizes the more significant vehicle movements and events associated with the encroachments.

TABLE 3

SUMMARY OF SIGNIFICANT MOVEMENTS AND EVENTS ASSOCIATED WITH VEHICLE ENCROACHMENTS

Category	Highway	
	I74	I57
1. Total number of detected vehicle encroachments with sufficient evidence for analysis (lateral movements more than 3 ft into median)	293	25
2. Crossed median (lateral extent of movement greater than 40 ft)	52(17.7%)	0
a. Hit obstacle in median prior to crossing	3	0
b. Recovery-to-right[a] prior to crossing	5	0
c. Would have hit obstacle if recovery-to-right had been attempted	7	0
d. Ran off pavement to the right prior to crossing	2	0
e. Entered roadway from interchange entrance ramp just prior to crossing	4	0
3. Hit major obstacle in median[b]	35(11.9%)	4(16%)
4. Recovery-to-right during encroachment	185(63.1%)	18(72%)
a. Crossed median prior to recovery [a]	3	0
b. Crossed median after striking obstacle subsequent to recovery	2	0
c. Crossed original lanes of travel after recovery	13	1
d. Crossed original lanes of travel after striking obstacle subsequent to recovery	1	0
5. Ran off pavement to the right prior to median encroachment	9(3.1%)	3(12%)
6. Entered roadway from interchange entrance ramp just prior to encroachment	9(3.1%)	0(0%)

[a]"Recovery-to-right" (recovery) is defined as the point in an encroachment pattern at which the lateral velocity component of vehicle movement changes sign due to driver steering through a horizontal recovery curve to the right.

[b]Crossover embankment, culvert headwall, drop inlet structure or earth berm (ditch check).

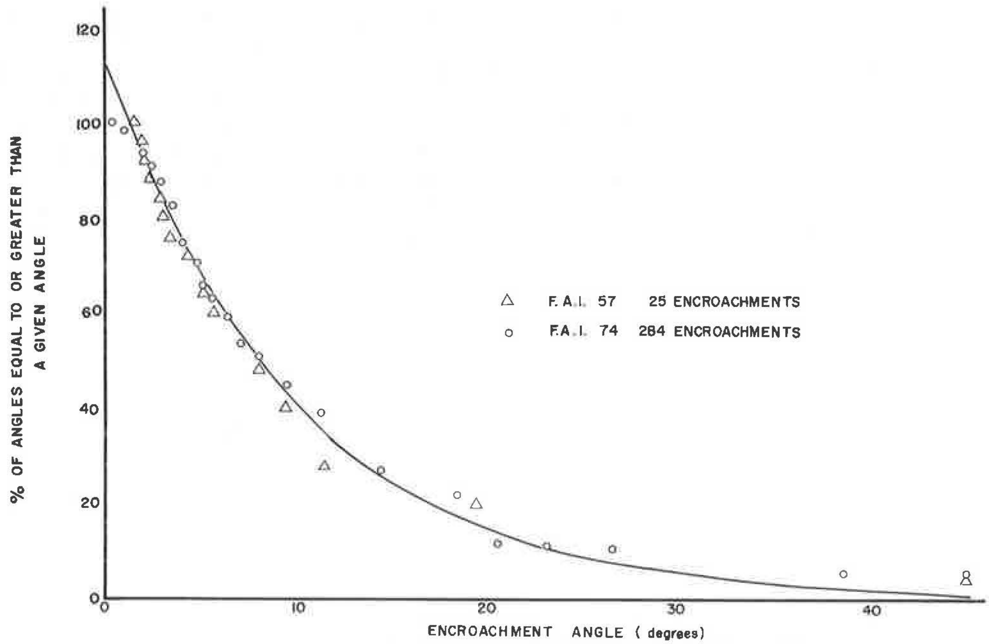


Figure 10. Distribution of encroachment angles for I57 and I74.

The distribution of angles is shown in Figure 10, which includes the combined data from I74 and 57. This figure represents the angles for all unintentional encroachments occurring during the periods of study and having lateral movements greater than 3 ft. The equation approximating the distribution is

$$\beta = 10^{(-0.044\theta + 2.057)}$$

where

β = percentage of θ's greater than or equal to a given θ, and
θ = encroachment angle, degrees.

The observed data deviate from the above expression at low angles (less than about 2 deg) and at high angles (greater then about 25 deg). The deviation at low values of θ is attributed to the failure to record shallow encroachments. The deviation at high values of θ is probably due to the circumstances normally associated with encroachment at large angles. It is doubtful that a vehicle could encroach upon the median at an angle greater than about 25 deg unless it was traveling at low velocity, was involved in a relatively severe collision, or was involved in initial movements resulting in running off the pavement to the right. Available data did not allow an appropriate consideration of the first two possibilities. In Table 3, however, it may be noted that 9 encroachments involved vehicles running off the pavement to the right prior to the actual median encroachment and that 9 others involved vehicles entering the roadway from an interchange ramp. The elimination of these encroachment types greatly reduces the deviation from the distribution relationship at high angles (Fig. 11).

Figure 11 was used to estimate the number of shallow encroachments which occurred on I74. The adjusted distribution curve in Figure 11 suggests that 45 shallow encroachments occurred on I74 during the period of study (17 percent of 266 encroachments).

A summary of the basic statistical parameters associated with encroachment angles in given in Table 4.

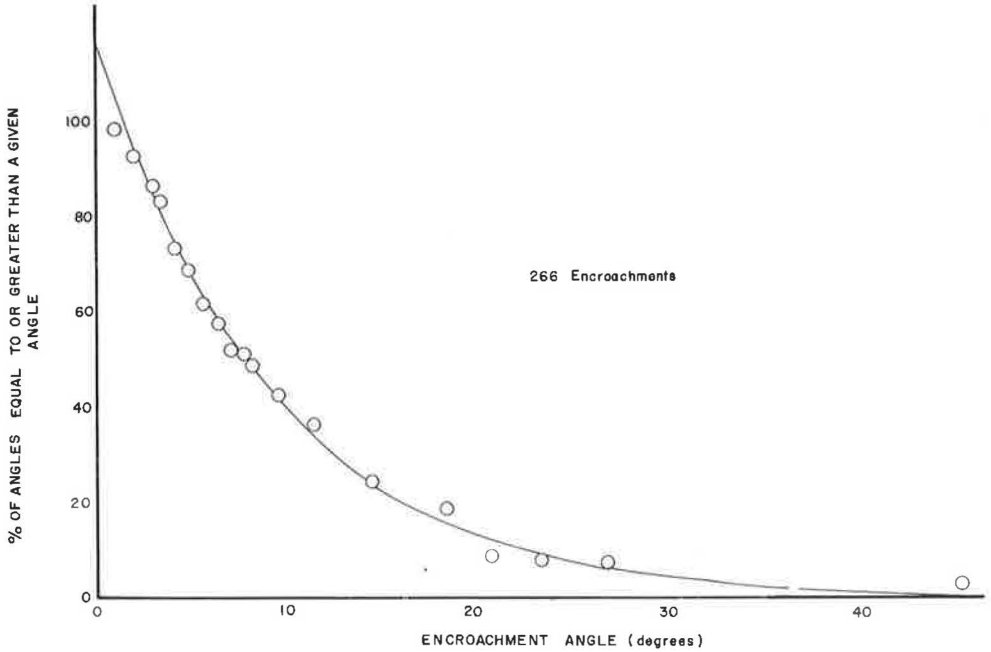


Figure 11. Distribution of encroachment angles for I74 (adjusted).

TABLE 4
BASIC STATISTICAL PARAMETERS ASSOCIATED WITH THE NATURE OF ENCROACHMENTS

Parameters	Number[a]			Average			Standard Deviation			Confidency Interval Coef of Confidence = 0.96		
	I57	I74	I57 and I74	I57	I74	I57 and I74	I57	I74	I57 and I74	I57	I74	I57 and I74
Encroachment angle (deg)	25	289	314	10.2	11.0	10.9	9.6	11.1	11.0	6.2 to 14.1	9.7 to 12.2	9.7 to 12.1
Length of travel (ft)	25	290	315	292	291	291	202	217	216	209 to 376	266 to 316	267 to 315
Lateral extent of travel (ft)	25	296	321	23	23	23	10	11	11	19 to 27	22 to 24	22 to 24

[a]The 3 basic parameters could not be measured for all detected encroachments.

The distribution of travel lengths during encroachment for I74 and I57 are shown in Figure 12. Since there appears to be no significant difference between the mean lengths of travel for the two highways (Table 4), data for both highways have been combined into a single distribution curve in Figure 13.

The distribution in Figure 13 closely approximates a normal distribution except at very low values (L less than about 60 ft) and very high (L greater than about 550). This deviation at high values may be due to the fact that it was impossible in many instances to determine whether a vehicle was not under control or whether the driver was merely operating his vehicle in the median area.

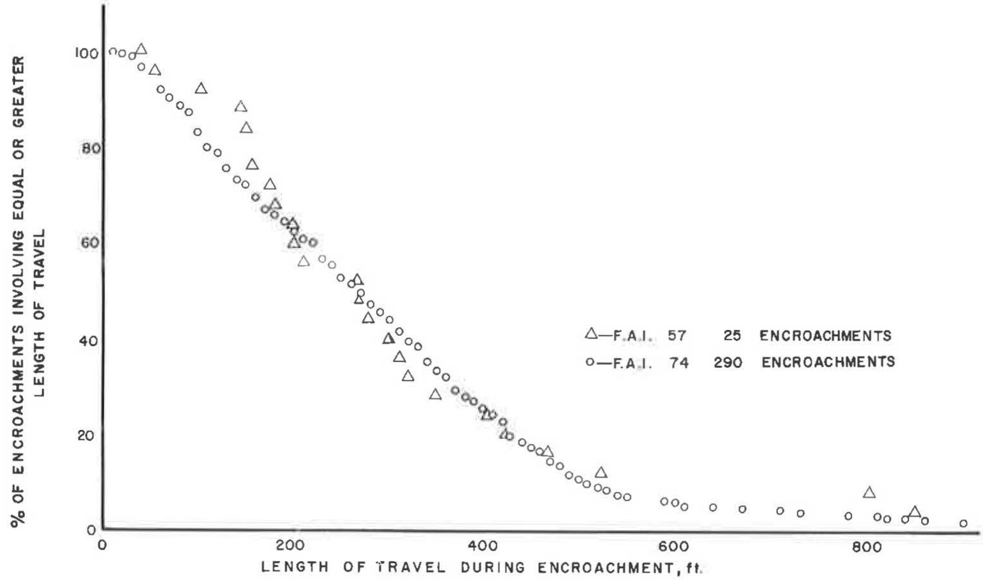


Figure 12. Distribution of lengths of vehicle travel during encroachment for I57 and I74.

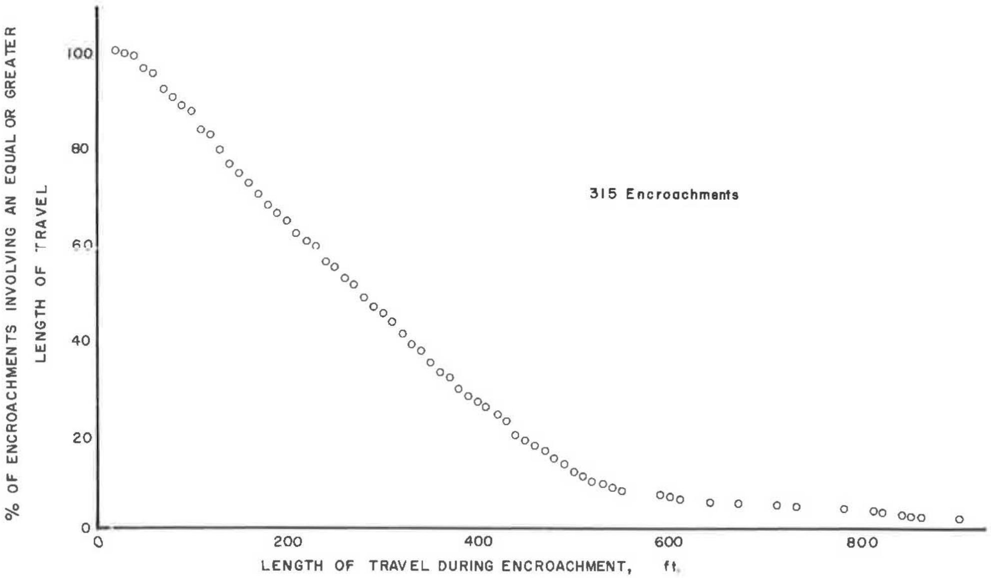


Figure 13. Distribution of lengths of vehicle travel during encroachment for I57 and I74, combined.

The mean values of L for I74 and I57 are 293 ft and 292 ft respectively. Other statistical parameters are summarized in Table 4.

The length of travel during encroachment is an indication of the reasonable extent to which the median should be free of obstacles which cannot be traversed safely at normal highway operating speeds. The greater the length of travel the greater the prob-

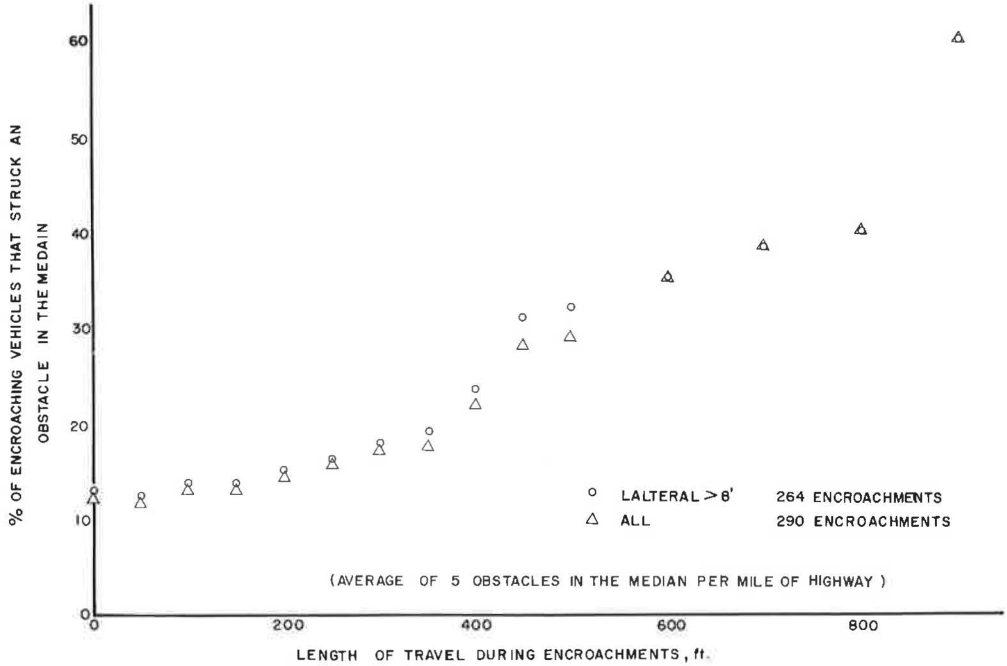


Figure 14. Frequency of vehicle collision with obstacles as related to length of travel during encroachment for I74.

ability that the vehicle will strike a median obstacle such as a culvert headwall, a drop inlet, or a ditch check. On I74 there is an average of five such obstacles per mile of median. Figure 14 indicates the percentage of vehicles that struck obstacles in the median. Figure 14 includes only those encroachments with lateral movements greater than 8 ft since there are no obstacles on the shoulder.

During the 3½-year study on I74, 11.9 percent of all encroaching vehicles struck obstacles in the median. Many other vehicles either straddled or passed between culvert headwalls or passed smoothly over headwalls with only the tires making actual contact with the obstacle. With possibly as many as 14 encroachments per mile per year at higher traffic volumes (Fig. 2), the above findings indicate that an average of more than 1.6 veh/mi per year could eventually be expected to strike obstacles in the median of I74.

The significance of obstacles in the median is even greater than would be indicated by an estimate of the accident cost and injury resulting directly from collisions with these obstacles. Many vehicles that crossed the median on I74 apparently did so as a result of the driver's attempt to dodge obstacles. Such encroachments are usually characterized by the lack of evidence of any driver attempt to recover-to-the-right; very few of the cross-median encroachments involved an actual collision with an obstacle in the median. Only three (5.8 percent) of the 52 cross-median encroachments involved vehicles striking obstacles (Table 3). All but 5 of the cross-median encroachments occurred without a recover-to-the-right. The dodging of obstacles in the median is an important factor in cross-median movements.

The distribution of maximum lateral movements during encroachment is shown in Figures 15 and 16. Figure 15 contains the separate distribution curves for I74 and I57 and Figure 16 contains the distribution curve for I74 and I57 combined. Since the median of I74 has essentially the same cross section as the first 40 ft of the 80-ft I57 median and since none of the encroachments on I57 had lateral movements greater than 40 ft, this combination of data appears to be justified. Basic statistical parameters associated with the lateral extents of movement for both highways are contained in Table 4.

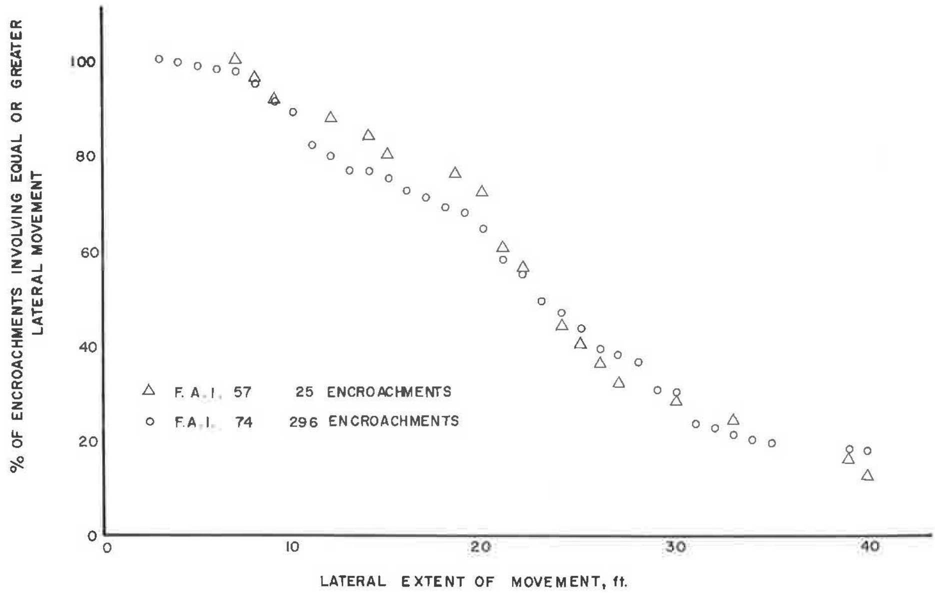


Figure 15. Distribution of lateral movements of vehicles during encroachments for I57 and I74.

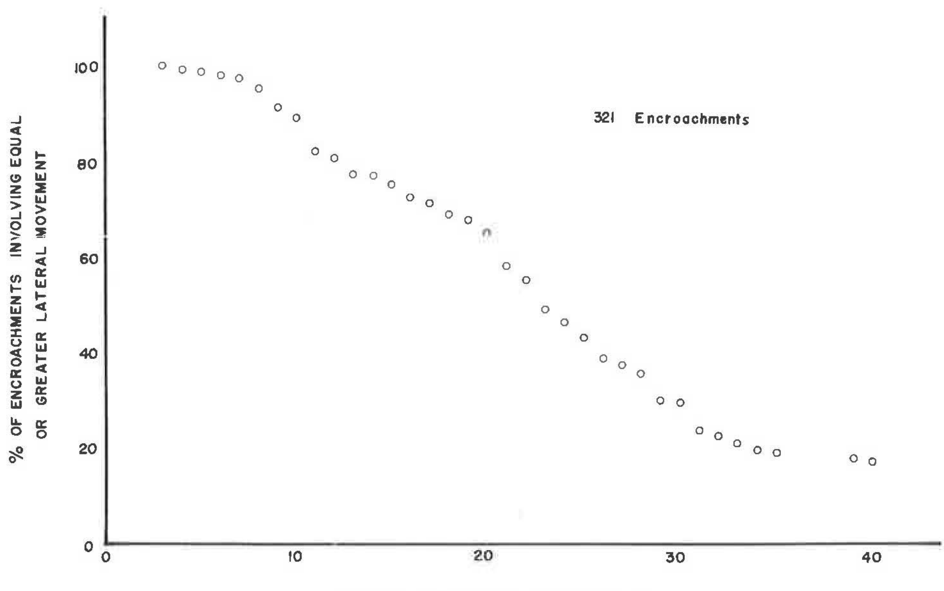


Figure 16. Distribution of lateral movements of vehicles during encroachments for I57 and I74, combined.

The important aspect of these relationships is the apparent influence of median cross section slopes; changes in the slope of the distribution curves occur at approximately the same lateral distances as do the changes in median cross section slope.

When a vehicle is moving at an angle to the left of its intended path of travel it is brought under control by steering it through a horizontal curve to the right. The slope of the median for the first 20 ft of lateral movement is negative superelevation (24:1 on the shoulder, 4:1 on the side slope) on the recovery curve traversed by the vehicle.

Figure 16 shows that, in the interval from 3 to 8 ft of lateral movement (shoulder area), the distribution curve is practically horizontal, indicating that very few vehicles are brought under control in this area. Few drivers are able to regain control of their vehicle on the shoulder once the left front wheel of the vehicle leaves the 3-ft stabilized portion of the shoulder. In the interval from 8 to 11 ft (rounded transition from shoulder to back-slope) a much greater percentage of vehicles is brought under control despite the change from a mild negative slope (24:1) to a severe negative slope (4:1). The relatively large number of recoveries occurring near the shoulder edge is attributed to the reaction time of the driver and vehicle. The vehicle could traverse the shoulder width in the time that it takes for perception by the driver and response by the vehicle to the driver's natural reaction to correct to the right. Assuming a vehicle speed of 60 mph, an average encroachment angle of 7.5 deg[1], and a 1-sec interval between driver perception and vehicle response, the vehicle would travel approximately 11.4 ft laterally.

In the interval from 11 to 20 ft the slope of the distribution curve is relatively flat, indicating that few vehicles are brought under control on the greater negative superelevation (4:1) of the median ditch side-slope. At about 20 ft of lateral movement the distribution curve steepens as a result of the positive superelevation provided by the back-slope of the median ditch. Beyond this, at a lateral distance greater than about 32 ft, the slope of the distribution curve becomes flatter again as the slope of the shoulder on the opposing roadway provides less positive superelevation on the vehicle recovery curve.

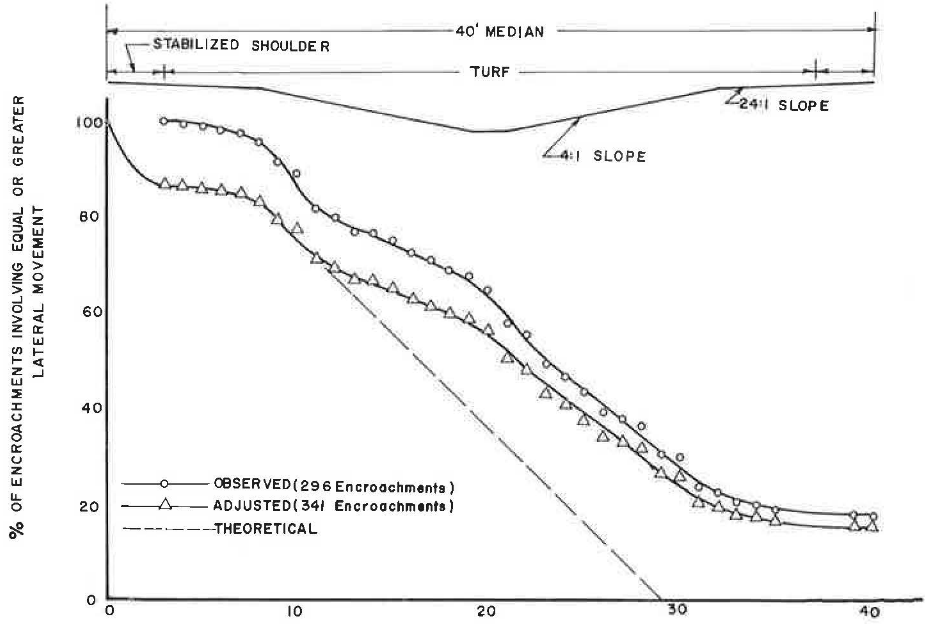


Figure 17. Distribution of lateral distances for I74.

[1]The average encroachment angle for vehicles recovering in the interval from 8 to 11 ft of lateral movement.

The interesting implication is the possibility of maintaining a steep slope of the distribution curve across the entire width of the median by providing a small negative superelevation such as the 24:1 slope used on the shoulders of I74.

Figure 17 shows the actual, adjusted, and theoretical distributions of maximum lateral movements for I74. The adjusted curve includes the estimated number (45) of shallow encroachments. This estimate indicates a relatively large number of encroachments occurring within the 3-ft width of stabilized shoulder. This portion of the distribution curve is substantiated to a certain extent by previous findings on US 66 (3). The theoretical curve is an estimate of the distribution of maximum lateral movements that would be obtained if the 24:1 shoulder slope were extended. It was constructed by extending the slope of the adjusted distribution relationship that occurs in the interval from 9 to 11 ft of lateral distance. This estimate is conservative because an even larger percentage of the drivers would probably have regained control of their vehicles in the interval from 8 to 11 ft of lateral movement if the vehicle had not been in an area of high negative superelevation (4:1) on the recovery curve. The one major limitation of this estimate is that, although the percentage of vehicles reaching the centerline of the median would be reduced, the effect of the decreased positive superelevation provided by the median ditch back-slope cannot be evaluated.

Considering this theoretical curve, it appears that practically all of the encroaching vehicles would have been brought under control within a lateral distance of about 29 or 30 ft. In view of the present trend toward wider medians to reduce headlight glare, the possibility of reducing median cross section slopes should be considered and investigated.

Another major benefit resulting from the use of flatter median cross slopes could be an increased driver calmness resulting in a higher probability of safe recovery. On the presently used 4:1 side-slope the driver's natural reaction is to turn sharply to the right while the median cross slope is causing the vehicle to be pulled violently to the left. This probably causes many drivers to overcorrect to the right. Table 3 shows that 13 vehicles (7.0 percent of those recovering to the right) were overcorrected to the right, across their former lanes of travel.

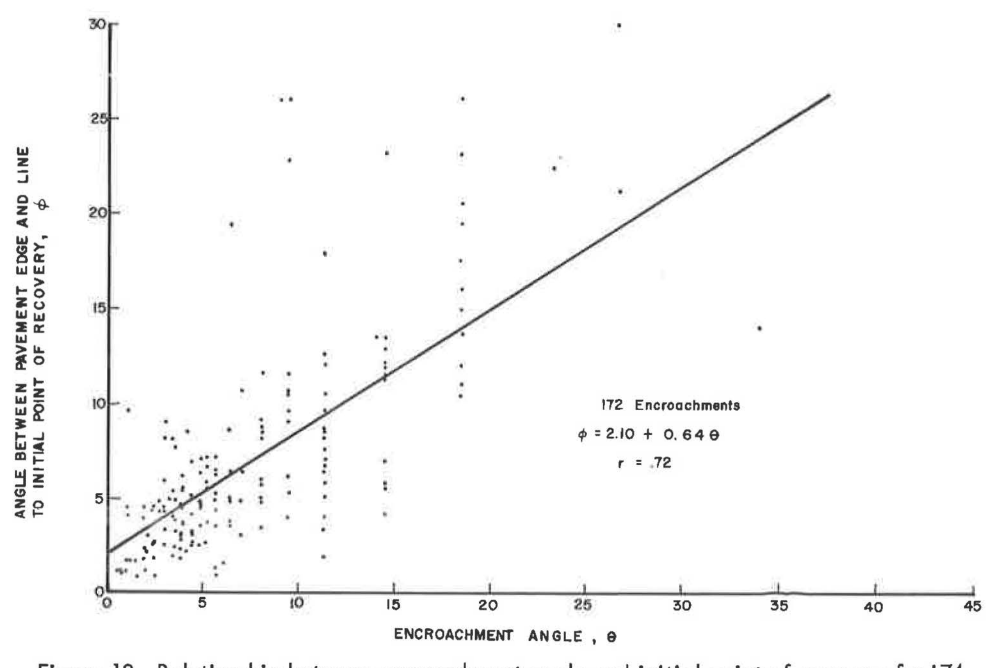


Figure 18. Relationship between encroachment angle and initial point of recovery for I74.

Attempts to determine significant relationships among the three basic parameters (angle of encroachment and lateral and longitudinal encroachment travel distances) were not successful; this was probably due to the large number of variables (vehicle speed, extent of braking, etc.) which could not be measured. Of particular interest, however, was the observation that in the case of 185 (63.1 percent of the 293 encroachments on I74), the drivers made a recovery-to-the-right.

Because of the relatively large number of drivers who made a recovery-to-the-right, an attempt was made to determine the relationship between the three basic parameters at the point of initial recovery-to-the-right. A significant relationship of this type would provide valuable guidance in the design and placement of highway guardrails.

The lateral extent of movement at the point of initial recovery-to-the-right is termed the initial lateral extent of movement, X′, and the length of travel to this point is termed the initial length of encroachment travel, L′. Figure 18 shows the relationship between θ and Φ. The equation approximating this relationship is

$$\Phi = 2.10 + 0.64\,\theta$$

where

Φ = angle between pavement edge and the line connecting the initial point of recovery with the point of beginning of the encroachment, and
θ = encroachment angle.

A second form of the equation is

$$\frac{X'}{L'} = \sin\,(2.10 + 0.64\,\theta)$$

where

X' = initial lateral extent of movement, and
L' = initial length of encroachment travel.

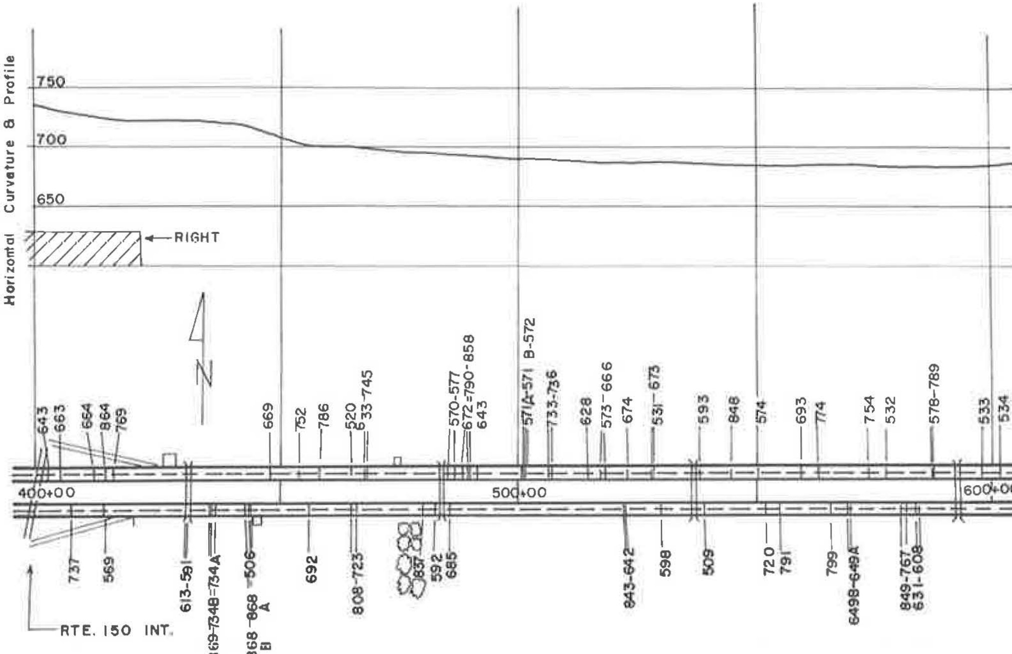
Although the above regression line is highly significant, the correlation coefficient indicates a large amount of deviation from the regression line. Thus, a definite relationship between θ and Φ appears to exist but it cannot be used to obtain a highly reliable prediction of the area of vehicle recovery. For any given encroachment angle, a wide range of lateral and longitudinal travel distances may be expected. This range appears to increase with traffic volume as discussed in connection with Figures 7, 8 and 9.

CAUSES OF ENCROACHMENTS

Over six million vehicle trips were made on this segment of I74 during the 3½-year study; less than five thousandths of one percent of these trips involved a vehicle encroachment upon the median. The individual driver, vehicle and highway factors involved in the encroachments are normally so subtle that they may seriously affect only one trip in ten thousand.

The driver is the ultimate variable because he is the conscious and subconscious sensor of landscape, vehicles, roadway alignment, weather, fatigue, light conditions, pavement surface conditions and most other factors involved in erratic vehicle movements. Because of the number of encroachments observed in this study, it was possible to recognize certain factors that had rather consistent overriding effects. Among these are light conditions, fatigue, roadway alignment, weather, roadside signs, grade separation structures and terrain features.

Unfortunately, the average number of encroachments observed per unit of length of highway is quite small when only 500 or 1000 ft of highway are being considered in connection with a curve, roadside sign or interchange ramp. If all observed encroachments during the 3½ years of study are considered, there is an average of only 2.325 encroachments per 1000 ft of highway. With such a small average there is no appropriate



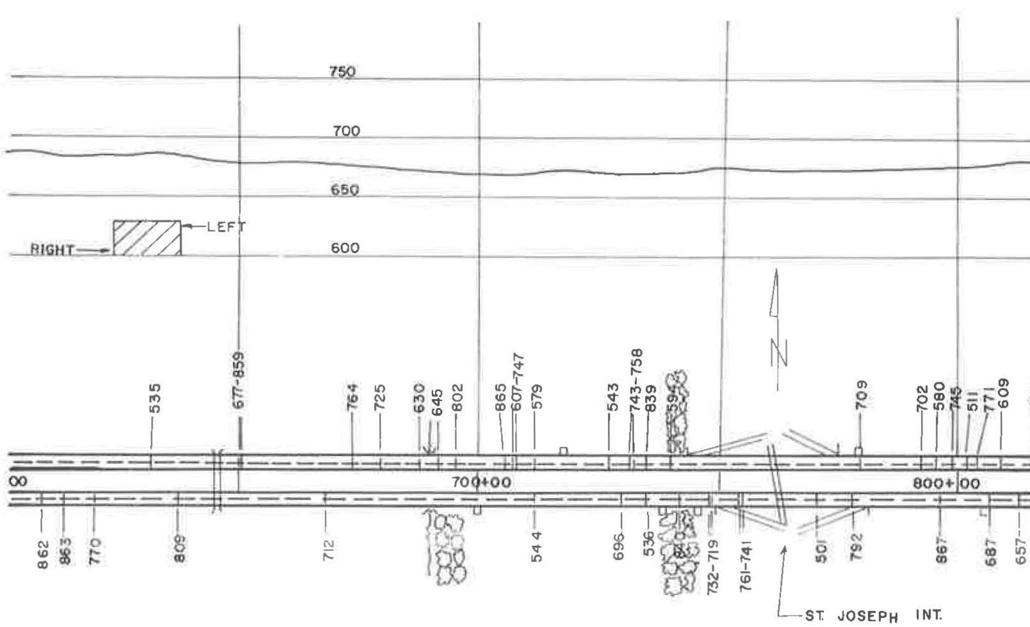
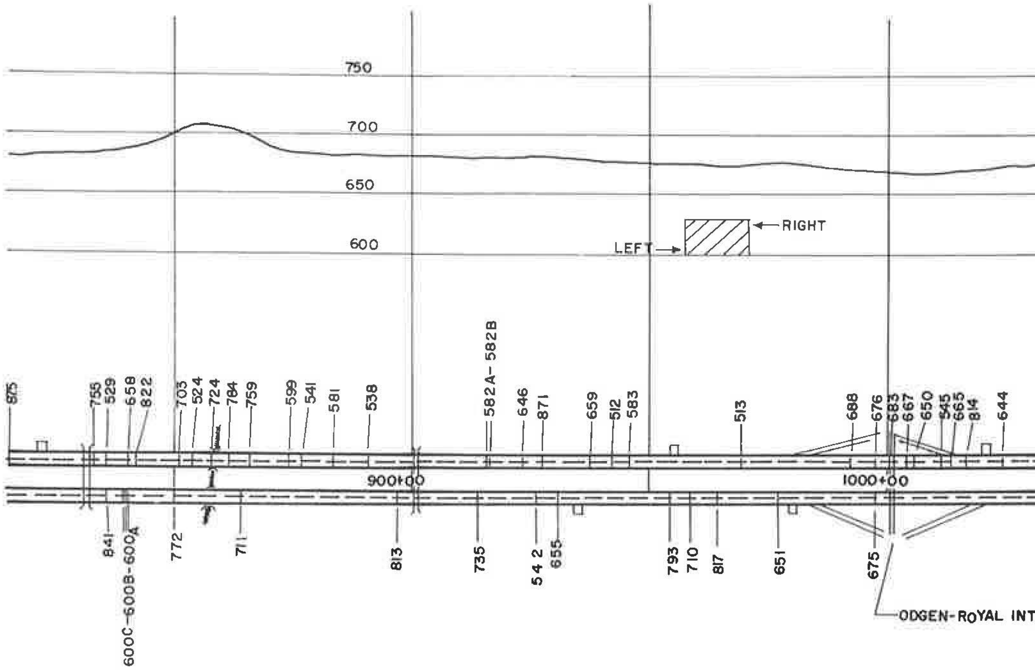


Figure 19. Strip map of 174.



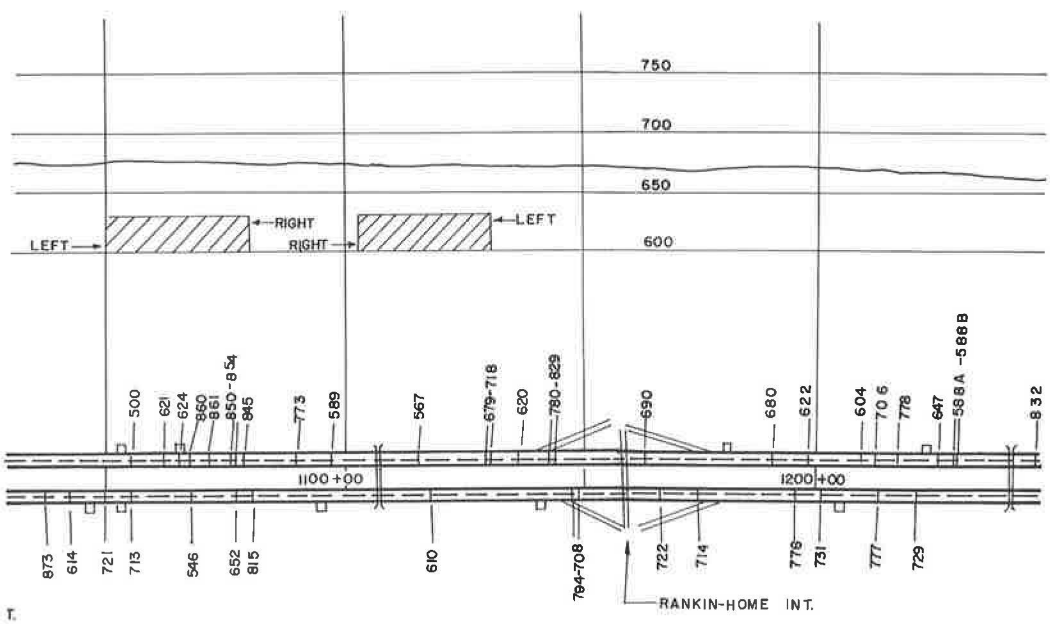
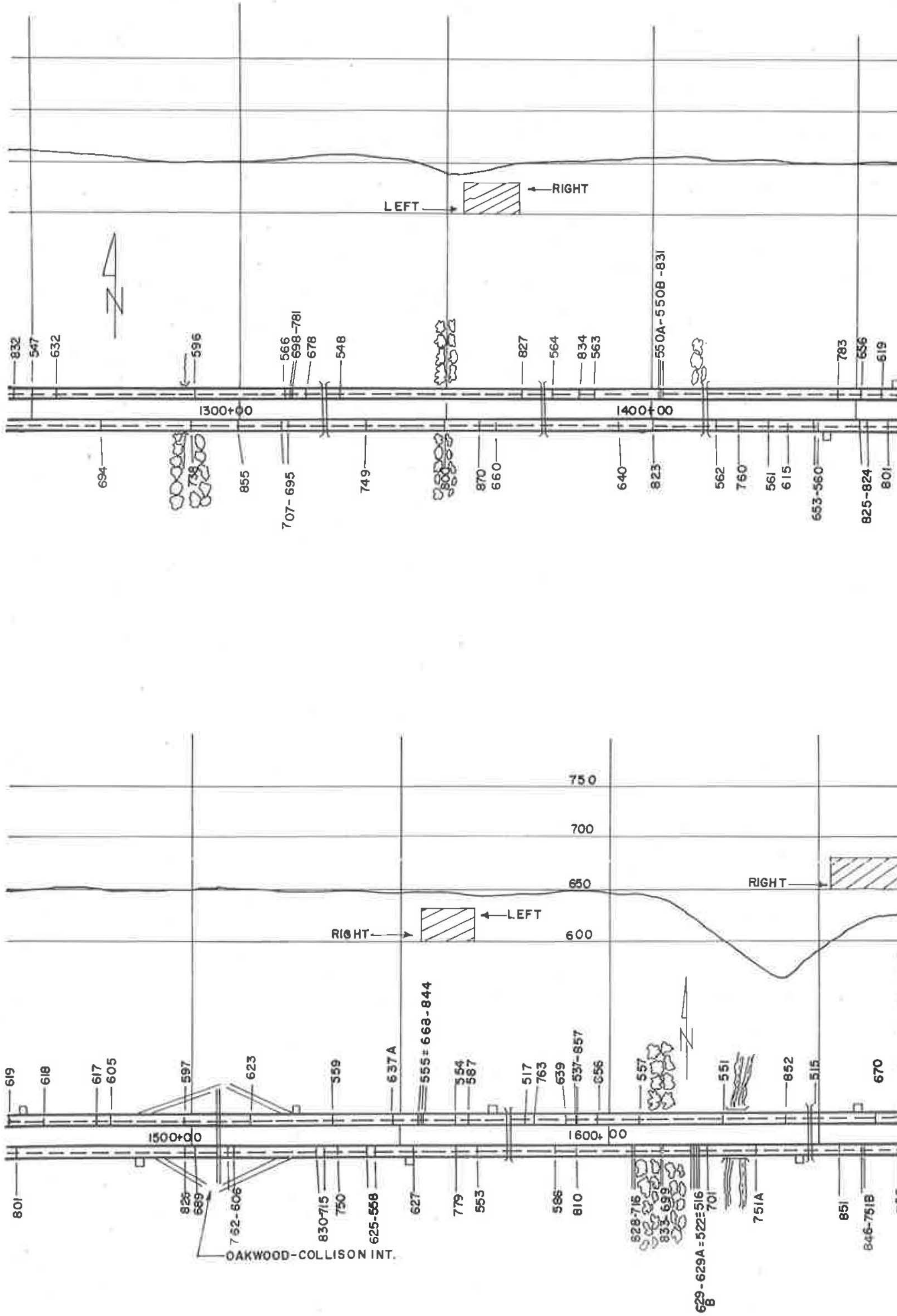


Figure 19. (Continued)



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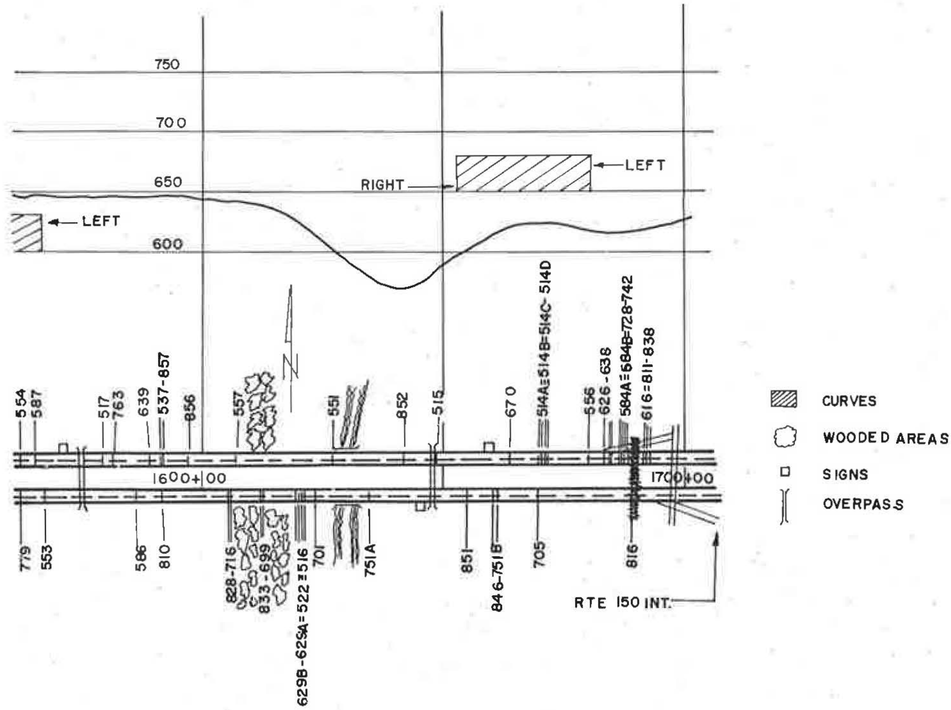

Figure 19. (Continued)

statistical test of the significance of 3, 4 or even 10 encroachments within any given 1,000-ft length of highway. Therefore, statistical tests were applied only in the case of factors affecting the entire length of study section.

Light Conditions and Fatigue

Figure 19 is a strip map of the 24.6-mi study section showing roadway features, pavement station numbers and the exact location of each of the 302 observed encroachments.

The identification number for each encroachment is located next to the roadway from which the encroachment originated. Cursory examination of the strip map reveals that a majority of the encroachments originated from the westbound traffic stream. Actually, 58 percent of the vehicles were westbound. Chi-square testing of this deviation from the expected even distribution by directions (151 eastbound and 151 westbound) shows that this difference is significant (99 percent level).

Drivers in the westbound traffic stream are facing the sun in the afternoon when they are most likely to be fatigued from the day's activities, whereas eastbound drivers face the morning sun at the beginning of the day when they are most likely to be refreshed. If these circumstances are an explanation of the observed difference in number of encroachments originating from the two directions of travel, then the previous conclusions with regard to vehicle caravaning would suggest that these differences in number should decrease as the volume increases. The gradually increasing number of vehicles in the westbound traffic stream should provide progressively better roadway delineation to help offset the effects of driving into the afternoon sun.

This reasoning is supported by the encroachment data. Figure 20 shows how the separate encroachment rates for the two directions of travel varied with traffic volume. The westbound rate, originally much higher than the eastbound rate, decreased rapidly in the volume range from 4,000 to 5,000 veh/day and finally became nearly equal to the

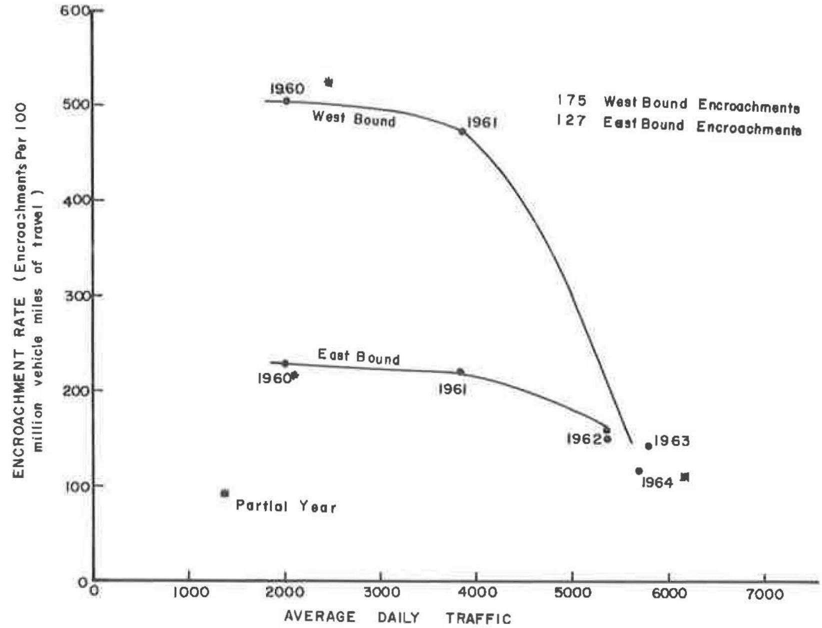

Figure 20. Yearly encroachment rates by direction of travel for I74.

eastbound rate. Apparently, the increasing number of vehicles in the traffic stream eventually provided enough roadway delineation to offset the effect of driving into the afternoon sun.

Another possible effect of fatigue is indicated by the distribution of encroachments along the study section. A relatively high percentage of encroachments occurred near the west end of the study section for the westbound traffic stream and near the east end for the eastbound traffic stream. Slightly over 22.5 percent of all encroachments originating from each traffic stream occurred on the last 4 mi (16.25 percent of mileage) at the downstream end. This includes consideration of those encroachments (10 eastbound and 10 westbound) resulting from entrance ramp friction at the upstream end for each direction of travel. If those encroachments are not considered, the percentage occurring on the last 4 mi in each direction of travel becomes 24 percent. Many different fatigue-related factors such as decreased alertness, velocitation and lowered visual acuity are undoubtedly involved in this downstream distortion of encroachment distribution.

Weather

Weather is partially responsible for the differences in observed numbers of encroachments in the two directions of travel. The prevailing wind and source of storms is from the westerly direction in central Illinois. Westbound vehicles are normally subjected to much higher wind velocities (travel speed plus wind speed) than are eastbound vehicles (travel speed minus wind speed). This difference in wind velocities magnifies the effect of windbreaks (overpass abutments, roadside groves of trees, etc.) on westbound vehicles. The hardest driven rain and snow storms are also from the west. Westbound vehicles are, therefore, subjected to more rapid accumulations of snow and rain on the windshield with resulting poorer driver visibility.

Some of the variation shown in Figure 4 is due to weather changes. As indicated, the number of encroachments during April, May and June varied greatly during 1961, 1962, and 1963. In 1961, this three-month period was marked by temperatures averaging 4.4 deg below normal and precipitation averaging 1.81 in. above normal. During this cold, wet weather in the spring of 1961 there were nearly three times more encroachments than were observed during the same three-month period in 1962 when the temperature averaged 2.1 deg above normal and the precipitation averaged 0.78 in. below normal. The same three-month period in 1963 brought even less precipitation (2.31 in. below normal) accompanied by a further decrease in encroachments.

Weather differences and traffic volume increases are both partially responsible for the rapid decrease in encroachment rate during 1962. The relative effects of each cannot be accurately judged. Nevertheless, the continued low rate after the spring of 1963, in the absence of any other significantly abnormal weather conditions, suggests that increasing traffic volumes would have eventually brought about the same reduction in rate even without improved spring weather. Therefore, traffic volume increases are assumed to have produced most of the rate reduction which occurred in 1962.

This assumption has no important effect on Figures 2 and 3. An adjustment of the volume-frequency relationship to account for the effects of spring weather would merely broaden the apex of the curve in Figure 2 and steepen the sharp downward slope of the curve in Figure 3.

Interchanges

There were 6 interchanges involved in the study section: 4 diamond interchanges within the study section and a trumpet interchange at each end. The number of encroachments at each interchange and in the vicinity of each interchange ramp was compared with the average number of encroachments per equivalent unit length of highway.

There was no significant variation that could be attributed to any of the interchanges as a whole, regardless of the length of highway considered to be within the area of influence. For an assumed area of influence extending 500 ft beyond the entrance and exit ramps, the interchanges represent 16 percent of the length of the highway within which only 13.8 percent of the median encroachments occurred.

If each interchange ramp is considered separately, there appear to be three locations with significant concentrations of encroachments: (a) the entrance ramp for eastbound vehicles at the west end of the study section (station 420 + 00), with 10 encroachments; (b) the exit ramp for westbound vehicles at Ogden, Illinois (station 1017 + 50), with nine encroachments; and (c) the entrance ramp for westbound vehicles at the east end of the study section (station 1685 + 00) with ten encroachments.

The reasons for the concentration of encroachments near the Ogden exit ramp were never apparent. Eight of these encroachments occurred during the first 1½ yr of surveillance and only one occurred during the remaining 2 yr.

The concentration of encroachments near the entrance ramp at the east end of the study section (station 1685 + 00) was a result of excessive vehicle entry speeds. Only two encroachments have occurred there since an advisory speed sign was installed on the ramp in November 1962 (Fig. 21).

Excessive vehicle entry speeds were only partially responsible for the ten encroachments near the entrance ramp at the west end of the study section (station 420 + 00). The overpass structure (station 431 + 00) and roadside sign (station 445 + 00) immediately downstream from the ramp are believed to be equally important, if not controlling factors at this location. Consequently, no special effort has been made to control vehicle speeds on this entrance ramp.

Grade Separation Structures

Within the 24.6-mi study section there are 18 grade separation structures which carry state highways and local roads over I74. Four of these are at the diamond interchanges. These 18 structures serve as windbreaks, the effects of which can be seen in the encroachment distribution (Fig. 19).

For an assumed area of influence extending 800 ft upstream from each structure, the combined length of influence area for all 18 grade separation structures is 11.1 percent of the length of the study section. This area of influence contains 16.2 percent of the encroachments originating from the westbound traffic stream and only 8.7 percent of the encroachments originating from the eastbound traffic stream. This is evidence of the effect of prevailing west wind as discussed in connection with weather.

Figure 21. Westbound entrance ramp to I74 near Danville, Illinois.

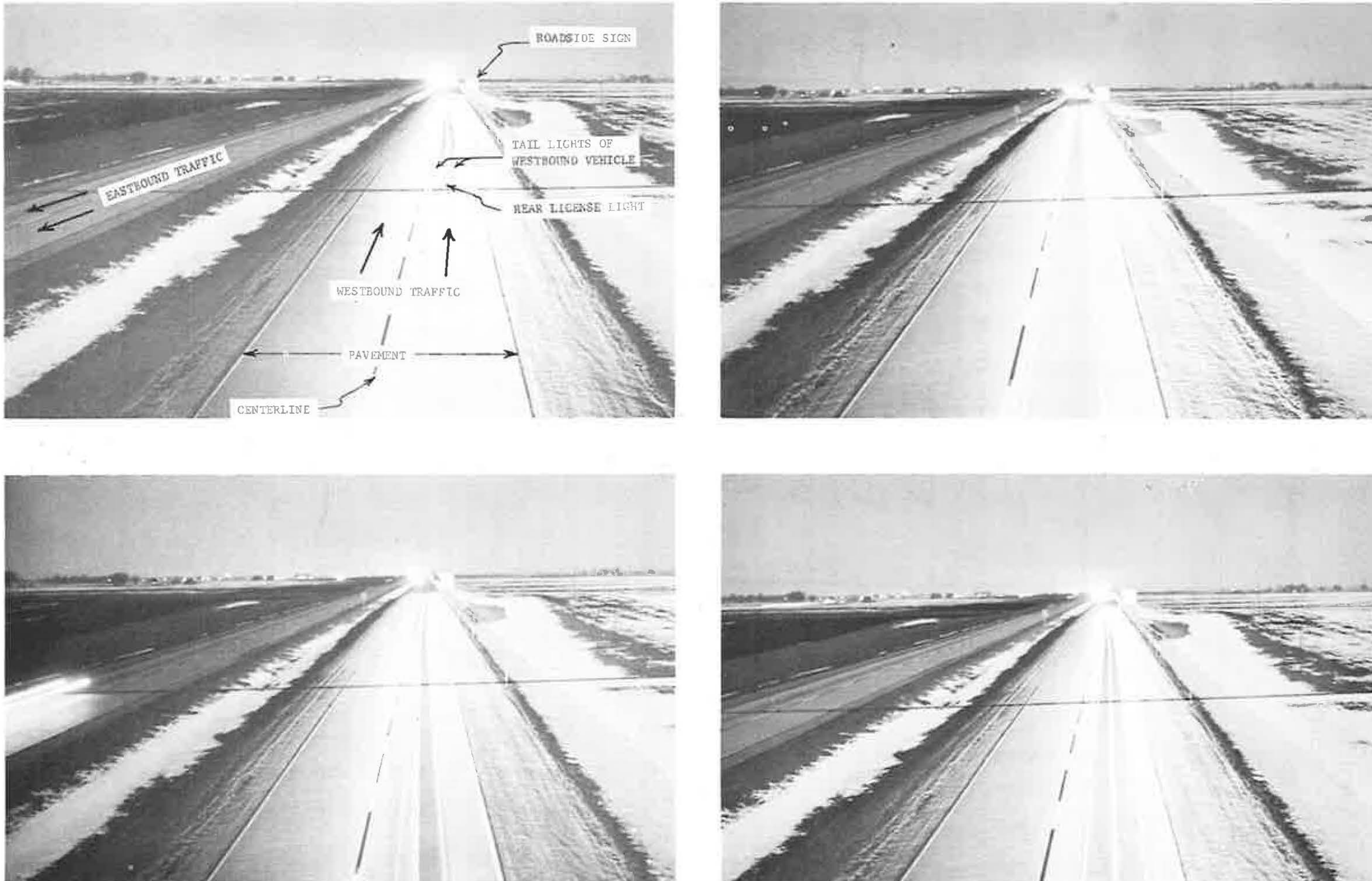

Figure 22. Time exposure of tall lights of individual westbound cars immediately west of grade separation structure at station 1241 + 00, showing how drivers veer away from large roadside sign in background.

TABLE 5

OBSERVATIONS OF VEHICLE BEHAVIOR AT ROADSIDE SIGN

Date	Method	Number of Vehicles Observed	Number Shifting to Left	Percent
11/9/63	Manual observer	50	13	26
2/7/64	Photographic	10	5	50
2/22/64	Photographic	32	10	31
	Total	92	28	30

If this same analysis is made for 800 ft downwind (from the prevailing west wind) areas of influence at each grade separation structure, the distribution becomes more nearly the same for both directions of travel. The combined influence areas contain 16.2 percent of encroachments originating from the westbound traffic stream and 13.4 percent of encroachments originating from the eastbound traffic stream.

Grade separation structures serve as windbreaks which cause erratic vehicle movements on the pavement downwind from the structures. Encroachments result only when these erratic vehicle movements occur in combination with other conditions such as driver inattentiveness, poor visibility and wet, icy or snow-covered pavement surface. The combinations of conditions necessary for encroachments rarely occur, but erratic vehicle movements can be observed downwind from the structures on any windy day.

Roadside Signs

Erratic vehicle movements on the pavement were also observed in the vicinity of large roadside signs. Vehicles apparently veer away from the signs as illustrated in Figure 22. The sign in Figure 22 faces westbound traffic and serves as a small windbreak for vehicles heading into the prevailing west wind. The absence of a heavy accumulation of snow on the roadside immediately downwind from the sign indicates the area of greatest air turbulence. The beginning of most erratic vehicle movements coincides with the downwind limits of this area of high air turbulence.

Three series of observations were made at the location shown in Figure 22. The results (Table 5) show that slightly over 30 percent of the vehicles shifted to the left in the vicinity of the sign. Lateral movements normally ranged from 2 to 4 ft, with the maximum lateral movement usually occurring about 500 ft upstream from the sign. However, some vehicles changed lanes, from right to left, in the vicinity of the sign and subsequently returned to the right-hand lane. Others shifted to the left at a considerably greater distance upstream from the sign, beyond the limits of the area affected by high air turbulence.

Six encroachments in the vicinity of this sign (station 1225 + 00) were probably influenced by such erratic vehicle movements on the pavement. Many other small concentrations of encroachments are associated with roadside signs throughout the length of the study section for both directions of travel. However, the greatest effect occurs where a roadside sign is located about 1,500 ft downstream from a grade separation structure.

One such location, immediately downstream from the entrance ramp at the west end of the study section, was discussed in connection with the effects of interchange ramps. Six of the first nine encroachments downstream from this entrance ramp were probably influenced by the roadside sign at station 445 + 00. A grade separation structure occurs just upstream from this sign, but it is far enough upstream that its effect as a wind-break could not have been responsible for encroachments near the sign. The abutment of the structure hides the sign from the driver's view until he is within about

2,500 ft of the sign. Then, as the sign begins to appear within the landscape viewed by the driver, it gives a subtle impression of having moved out toward the roadway from behind the abutment. This does not seem to make a conscious impression on the driver; all test drivers who were rightfully accused of having shifted to the left either emphatically denied it or claimed that they were not aware of having done so in the vicinity of the sign. Some were unsuspecting members of the research project staff who were aware of this phenomenon, but who were driving over the study section for other reasons.

Current practice with regard to placement of roadside signs should be reviewed in the light of these observations. Large roadside signs should be located as far downstream from overpass structures as is practical. Smaller signs, such as the ones giving distances to various cities, appear to have considerably less effect on driver-vehicle behavior.

Curves

Of the seven horizontal curves wholly within the study section, so many are accompanied by large roadside signs that the effects of the curves and signs cannot be separated.

Nevertheless, roadside signs appear to have some effect on the frequency of encroachment in the vicinity of horizontal curves. If the two long curves near station 1100 + 00 are compared, it can be seen that about 2½ times more encroachments occurred at the curve accompanied by large roadside signs.

A similar difference in numbers of encroachments is noted when the relatively short curve, with a large sign, at station 1560 + 00 is compared with the other three curves of similar length that are not accompanied by large roadside signs, stations 630 + 00, 960 + 00 and 1360 + 00. Curves and the approaches to curves should be avoided as locations for large roadside signs.

Curves, per se, do not appear to have been important factors affecting encroachment frequency. For an assumed area of influence extending from the beginning of each curve to a point 500 ft downstream from the end of each curve, the combined length of influence area for all curves represents 15.1 percent of the mileage of the test section, within which 15.1 percent of the encroachments occurred. Without the effects of roadside signs, the curves might possibly have been the safest sections of roadway.

Curves without roadside signs appear to be in areas where encroachments are thinly scattered, especially the areas from station 600 + 00 through 685 + 00 and from station 1330 + 00 through 1375 + 00.

Terrain Features

Several of the other areas with thinly scattered encroachments are upstream from groves of trees that serve as dominant terrain features and delineate roadway alignment. Two examples, in connection with the eastbound traffic stream, are the areas from station 650 + 00 through 775 + 00 and from station 1250 + 00 through 1350 + 00. The areas from station 1450 + 00 through 1410 + 00 and from station 790 + 00 through 740 + 00 are similar examples for the westbound traffic stream.

Roadside clumps of trees seem to be beneficial as roadway delineators. However, the concentrations of encroachments downstream from some of these locations must not be overlooked.

When particularly thick clumps of tall trees are encountered, such as at stations 740 + 00 and 1610 + 00, the downstream windbreak effect largely offsets the upstream delineation benefits and there is not net reduction in encroachments. Consideration should be given to the need for thinning and/or pruning trees outside the limits of the highway right-of-way. Particular attention should also be given to landscaping grade separation structures with types of plantings that will not accentuate the windbreak effects of the structures.

In view of the many different highway, driver and vehicle variables that may be assumed to cause or inhibit encroachments, it is not possible to assign a specific cause of any given concentration of encroachments. The foregoing discussion merely emphasizes those features of the driving environment which were most often associated with encroachments. In some cases it appears as though an adjustment of features would be both desirable and practicable.

CONCLUSIONS AND RECOMMENDATIONS

Curves, the approaches to curves and the areas immediately downstream from grade separation structures should be avoided as locations for large roadside signs; the safety benefits that might accrue from the use of overhead signs in lieu of roadside signs should be thoroughly investigated.

More emphasis should be placed on landscape planting to improve delineation of roadway alignment; attention should be given to control of the windbreak effects of all roadside plantings, both inside and outside the right-of-way limits.

The windbreak effects of grade separation structures should be considered in connection with the other factors controlling the number and location of such structures; landscape planting, structure modifications and other possible means of controlling these windbreak effects should be thoroughly investigated.

Because of the wide range of lateral and longitudinal travel distances components of vehicles must be taken into account in the design of median barriers to achieve safe vehicle deceleration rates. Angles do not provide a reliable basis for prediction of lateral velocity components of vehicles at any appreciable distance from the pavement edge.

Consideration should be given to the design of flatter median cross slopes as a means of (a) decreasing the maximum lateral extent of movement of encroaching vehicles, and (b) decreasing the frequency of erratic vehicle reentry to the traffic stream.

Obstacles commonly built into the median, in the form of culvert headwalls, drainage inlet structures, earthen ditch checks and crossover embankments, place a serious limitation on the value of the median as a safe vehicle stopping or recovery space. The median appurtenances represented by these obstacles should be decreased to the smallest practical number and designed so as to present the least possible hazard to the passage of vehicles encroaching upon the median at normal highway operating speeds.

A 30-ft width of obstacle-free median with mild cross slopes (24:1 for a 30-ft median width and steeper allowable slopes for greater median widths) appears to be the absolute minimum requirement for safe stopping or control of vehicles encroaching on medians at rural highway operating speeds. This provides no more than a minimum reasonable chance for escape from the median, under control, without crossing into the opposing traffic stream.

This minimum width should always be exceeded in the design of new rural highway facilities because, as indicated in Figures 7 and 8, the average severity of vehicle encroachment increases with traffic volume. Consideration must also be given to the additional median width needed for economical highway drainage design and for the plantings, glare screens or horizontal separation of roadways that will reduce headlight glare to tolerable levels. A desirable median width of more than twice the 30-ft minimum is therefore not unreasonable.

When, for reasons of economy, the above absolute minimum median width cannot be provided, the installation of suitable median barriers, on the basis of suggested traffic volume warrants (6), should be considered.

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Freeway Level of Service as Described by An Energy-Acceleration Noise Model

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•AN EVALUATION of any change in operation, design, or system analysis requires a thorough study of those traffic parameters which reflect changes in traffic flow not only quantitatively but also qualitatively. Drew (1) categorized the traffic parameters of volume, density, speed, and queuing of vehicles on the Gulf Freeway to present a complete description of the quantity of flow on the facility. Wattleworth (2) used a systems approach in his description of the quantity of flow in terms of capacity. The measurement of other similar parameters such as travel time and delay, to complete the quantitative analysis, presents no immediate problem. The need exists, however, for a means of qualitatively evaluating controls, geometric design, and system operation which can be expressed in quantitative terms and which is correlated to known measurable traffic parameters. Geometric design, for instance, is largely dependent on pure judgment, since there are no positive methods of comparing the effects of various designs on the quality of flow or level of service.

LEVEL OF SERVICE

Level of service, as applied to the traffic operation on a particular roadway, refers to the quality of driving conditions afforded a motorist by a particular facility. Factors which are involved in the level of service are: (a) speed and travel time, (b) traffic interruption, (c) freedom to maneuver, (d) safety, (e) driving comfort and convenience, and (f) vehicular operating costs.

Many approaches to the problem of measuring level of service have been introduced in an attempt to obtain a qualitative measurement of traffic flow. A universal standard has not yet been adopted, however, perhaps due to the uncertainty of what traffic parameters should be measured that truly indicate the quality of flow.

The need for an objective evaluation of the level of service has long been recognized. Travel time on a roadway has often been used as a criterion for this evaluation (3, 4, 5). However, travel time may not always reflect the true conditions on parts of a facility and therefore may not always indicate the congestion and discomfort experienced by the motorist.

For example, Figure 1 represents a travel time and speed pattern between successive 500-ft sections of the inbound Gulf Freeway for one trip made near the end of the peak period. The figure shows the delay in one section of the freeway which is somewhat concealed by the value of the average travel time. The average speed for the entire trip was 41.5 mph which indicates good flow. However, a speed differential of more than 30 mph existed between some sections. The speed in one section of the freeway averaged only 24 mph and dropped below 20 mph. Therefore, the congestion in this section is dampened by the smooth flow in the remaining portions of the freeway when the flow is evaluated on the basis of travel time.

Figure 2 shows that the discomfort experienced by the motorist is not necessarily reflected in travel time. This figure represents a comparison of a section of the Gulf

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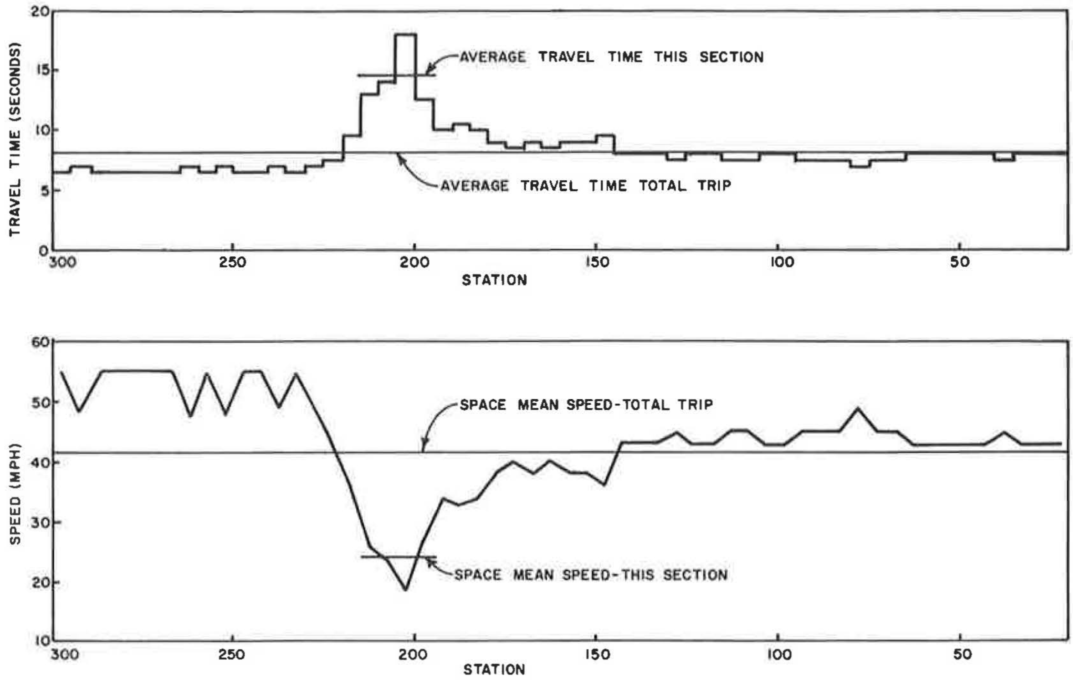


Figure 1. Comparison of travel time and speeds within sections to total trip.

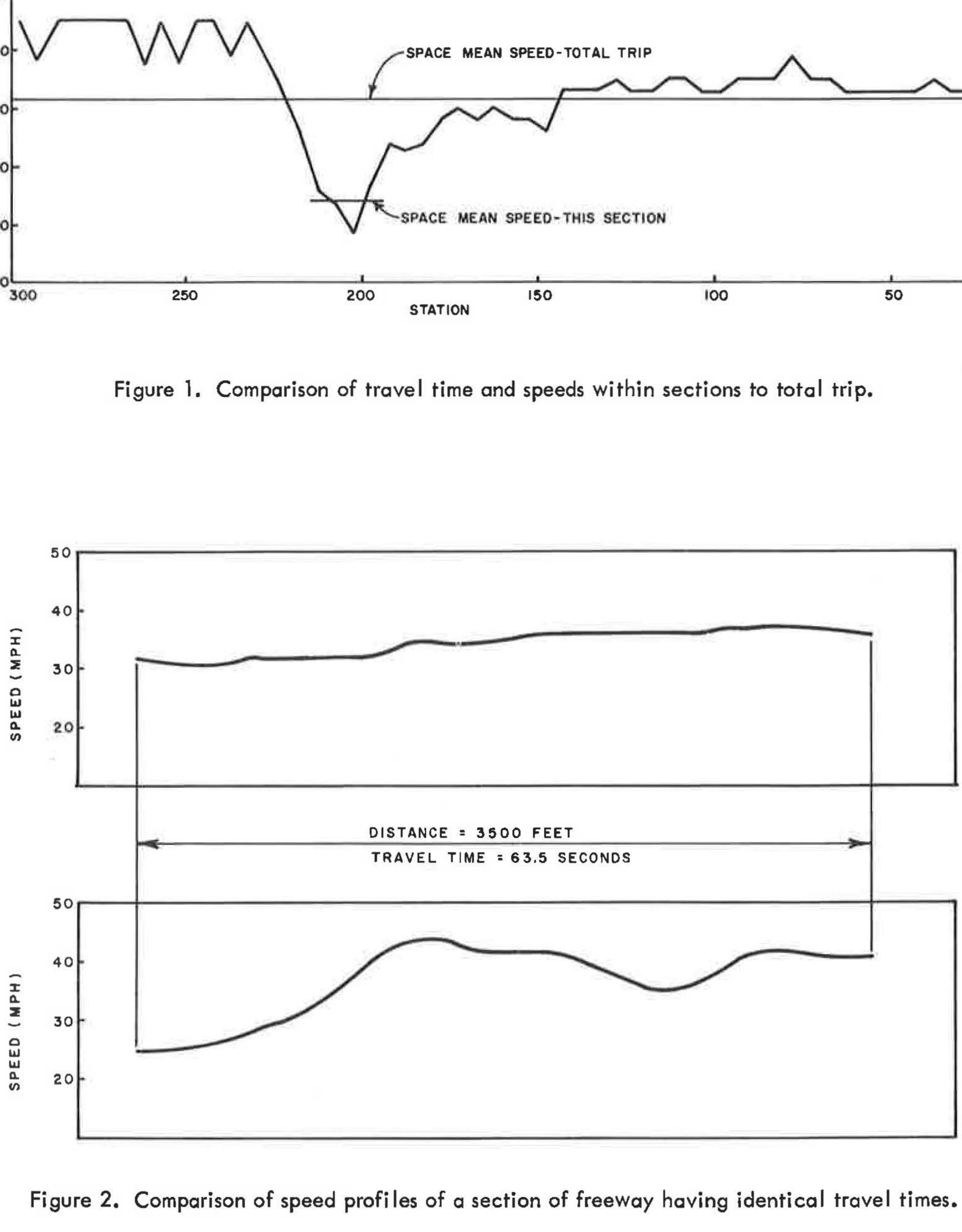


Figure 2. Comparison of speed profiles of a section of freeway having identical travel times.

Freeway 3500 ft in length, with identical travel times but with two distinct patterns of speed. The upper chart shows relatively smooth flow; however, the lower chart illustrates rapid decelerations which are indicative of motorist discomfort. Violent braking operations indicate dangerous conditions on the facility and add to the annoyance of the motorist, yet these maneuvers are not necessarily reflected by travel time.

The criterion of a systems approach (6) to increase the efficiency of traffic flow on a facility (or facilities) is based primarily on maximizing the system output. Since maximizing the volume output rate is equivalent to minimizing the travel time within the system, travel time is also used as the measurement of service provided to the motorist.

With a relatively large system and high volumes a small saving in travel time per vehicle, after geometric improvements have been made or during some control procedure, may result in a substantial reduction in travel time within a system. But will the reduction in travel time be appraised equally by both the motorist and the traffic engineer? For example, a saving of 2 min per vehicle within a large closed system may reflect considerable improvement to the traffic engineer. However, this saving may not be noticed by the average motorist because it may only represent a small percentage of his overall travel time. Unless a noticeable reduction of his travel time is provided, the motorist may conclude that the expenditures for the improvements or control system were not justified. However, if he were also provided with a smoother operation in addition to a saving in travel time, it most likely would be apparent to him.

There is evidence that the motorist personally evaluates a facility by the speed at which he can operate his vehicle and also by the uniformity of speed. Greenshields (7, 8) proposed a quality index with these factors in mind. His contentions in the development of the quality of flow index are that the overall speed determines travel time and is therefore proportional to the quality of flow. Also, the amount and frequency of speed changes are undesirable factors which irriate the motorist, increase the cost of operation, and are therefore inversely proportional to the quality of flow. The quality of flow index as formulated by Greenshields is

$$Q = KS/\Delta s\ \sqrt{f}$$

where

S = average speed (mph),
Δs = absolute sums of speed changes per mile,
f = number of speed changes per mile, and
K = constant of 1000.

Platt (9) observed that traffic delays and control devices are more annoying to the driver than slow moving traffic because they cause the motorist to stop. Driver satisfaction and driver effort do not vary linearly with speed but vary as complex functions. For these reasons two additional terms were added by Platt to Greenshields' quality of flow index. Some of the factors measured in the additional terms were speed change rate, steering reversal rate, accelerator reversal rate and brake application rate.

The Highway Capacity Committee of the Highway Research Board has proposed six levels of service as a basis of design. The basic speed-volume curve is divided arbitrarily into six levels of service as related to volumes and freedom to maneuver.

The indexes developed by Greenshields and Platt seem to possess excellent attributes for evaluating the overall efficiency of a long stretch of highway. However, it has been established that the varying nature of the geometrics is accompanied by a varied degree of optimum speed, volume, and density for short successive sections of freeway (1). If the parameter measuring the quality of flow is to be related to these known quantitative parameters, it must reflect the qualitative efficiency of small roadway sections. This parameter must not only reflect the engineer's evaluation of level of service but, more important, that measure of service which is considered by the individual motorist.

The Parameter

Because of recent research results, a traffic parameter referred to as acceleration noise has received attention as a possible measurement of traffic flow quality for two basic reasons. First, it is dependent on the three basic elements of the traffic stream: (a) the driver, (b) the road, and (c) the traffic condition. Second, it is, in effect, a measurement of the smoothness of flow in a traffic stream. The history, definition and manifestations of this parameter are discussed in the following paragraphs.

Definition and History. —On a highway with traffic volumes so light as to not restrict maneuverability, a motorist normally attempts to drive at a uniform and comfortable speed. Unconsciously, however, he accelerates and decelerates occasionally and deviates from a uniform speed. When traffic volumes increase to a level which restricts his desired speed, the motorist is forced to perhaps change lanes and increase speed to overcome slower moving vehicles. This results in higher and more frequent deviations from a uniform speed. The accelerations during his trip can be considered random components of time and the acceleration distributions essentially follow a normal distribution (10). The smoothness of his journey can be described by the amount that the individual random accelerations disperse about the mean acceleration. This deviation is measured by determining the standard deviation (also referred to as the root-mean-square deviation) of the accelerations.

The standard deviation of the accelerations is called acceleration noise. This parameter can be considered as the disturbance of the vehicle's speed from a uniform speed or can be identified as a measurement of the smoothness of traffic flow. The term noise is used to indicate that disturbance of the flow comparable to the coined phrase "video noise," which is used to described the fluttering of the video signal on a television set.

Vehicle accelerations can either be measured directly by an accelerometer or approximated from a speed-time graph of the vehicle's trip. This latter method is fully explained later in this report.

The distribution of accelerations of a vehicle which has been driven at almost a uniform speed throughout its trip will be similar to that shown in Figure 3. A vehicle experiencing higher deviations from a uniform speed might result in an acceleration distribution similar to that in Figure 4. Acceleration noise then varies with the amount and frequency of acceleration and deceleration. The more violent and more frequent these maneuvers are, the higher is the noise. Since violent deceleration seriously affects the acceleration noise, this parameter also reflects potentially dangerous traffic conditions.

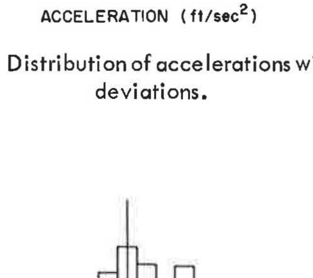


Figure 3. Distribution of accelerations with minor deviations.

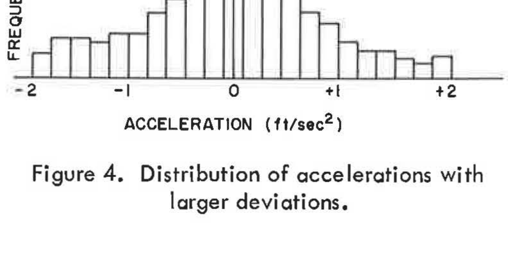


Figure 4. Distribution of accelerations with larger deviations.

The concept of acceleration noise was first introduced in conjunction with the car-following equations (11, 12). Additional investigations by Jones and Potts (13) showed its possible usefulness in helping to evaluate the quality of traffic flow in quantitative terms. The mathematical equation for approximating acceleration noise is derived in the Appendix.

Driver. —One of the most critical factors which affects the characteristics of traffic flow is the driver. However, because of the many external and internal elements which affect his decisions (14), evaluation of his behavior in a traffic stream presents a complex problem.

Acceleration noise seems to be a useful parameter in helping to evaluate the behavior of various drivers in a traffic stream in terms of danger potential. Since this parameter is affected by acceleration and deceleration, a reckless driver who attempts to drive faster than the stream of traffic will accelerate and decelerate violently and perhaps frequently, and will experience a much larger acceleration noise than the motorist who is content (or patient) with present flow conditions.

Road. —It might be conjectured that if a motorist was able to operate a vehicle on a perfect roadbed without the influence of traffic, the acceleration noise would be zero. However, tests (11) conducted using four drivers attempting to maintain constant speeds between 20 and 60 mph on the GM proving grounds resulted in acceleration noise of 0.32 ft/sec^2. Although a motorist desires to maintain a constant speed with ideal conditions, he unconsciously is unsuccessful.

Other experiments (11) using the same drivers and without the influence of traffic but on roads with varied geometric design produced an increased acceleration noise. The noise determined from runs in New York's Holland Tunnel was 0.73 ft/sec^2 and preliminary runs on poorly surfaced winding country roads was even twice this amount. The increase of noise in the tunnel was ascribed to the narrow lanes, artificial lighting and confined conditions. Investigations on winding country roads by Jones and Potts (13) showed values between 0.79 and 1.41 ft/sec^2. The acceleration noise decreased with improved road design features in the latter studies.

Acceleration noise which is present on a road in the absence of traffic is called the natural noise of the driver on the road (11). Based on the above findings it might be rationalized that this noise is some factor times the natural noise which would be experienced on a perfect roadbed. The factor can be ascribed basically to geometrics of the facility.

Comparison of acceleration noise on similar roadways might establish whether or not the intrinsic design features of the facilities have equal effects on the flow of traffic. In other words, a comparison of acceleration noise could perhaps relate the effects of the smoothness of flow due to the difference in geometric design of the similarly classed roads to the quality of flow. One might expect a higher level of service from the facility with the lowest acceleration noise.

Of equal significance is the attribute which this parameter seems to possess in comparing the smoothness of flow before and after changes in the geometric configuration of a facility. It has been established that each roadway exhibits natural noise in an amount greater than that of a near-perfect roadbed in relation to its intrinsic design features. Any improvement in design could perhaps be evaluated qualitatively by determining the amount that the acceleration noise approaches the noise of the ideal roadway. An attempt might be made to minimize acceleration noise in design procedures if this parameter shows signs of merit.

Traffic Condition. —Results of preliminary studies (13) on suburban highways showed that noise increased when congestion increased due to higher volumes and influence of parked cars. The amount of acceleration noise in excess of the natural noise of the facility was basically due to the existing traffic condition.

Although stopped time or delay is a popular parameter used in congestion evaluation, it is not necessarily a satisfactory means of evaluating congestion. Results of studies (13) on a suburban road passing through a shopping center which incidentally experienced a high accident rate (80 accidents per million vehicle miles) showed a stopped time of only 7 sec in an overall travel time of 128 sec. But the acceleration noise during the peak period was 1.43 compared to 0.77 ft/sec^2 during the off-peak period. The latter comparison no doubt gives a better indication of the degree of congestion.

The Model

One of the most difficult tasks facing the traffic researcher is the translation of a real traffic problem situation, comprising drivers, vehicles, control devices and the highway, into a set of mathematical symbols and relationships that reproduces their behavior. The fundamental conceptual device which enables one to regard this interaction as a whole is a model.

A model is an idealized representation of reality. It must be constructed in such a way as to reproduce the behavior of the real world with acceptable accuracy, recognizing that no abstraction can be identical to reality. This attempt to establish a correspondence between the problem and rational thought may be realized by forming a physical model or a theoretical model. Physical models may be either scalar or analogs of the prototype, such as a wind tunnel for testing aircraft, or an hourglass for measuring time. Kirchoff's dynamical analogy illustrates that the critical load on an axially loaded structural column may be determined by studying the oscillations of a pendulum of equal length. Numerically oriented analog models may be even more subtle. The slide rule and nomograph are numerical analogs; the odometer of a vehicle gives us the area under a vehicular speed vs time curve and as such is an integrator.

A theoretical model is essentially a hypothesis. For example, Newton's three laws of motion provide a theoretical model of our physical world. Most theoretical models are mathematically oriented for obvious reasons. Unless the hypothesis and the situation it describes are very simple, the only practical method of studying the many manifestations of a complex system is with the aid of mathematics. The trick, in representing a traffic situation, is to define the relevant parts and arrive at a set of relationships between them which, while simple, will result in meaningful predictions.

The traffic engineer has only a limited influence on the traffic variables. True, he can add traffic lanes so as to reduce the rate of flow per lane; he can set speed limits to discourage high speeds; and he can erect traffic signals to alternate the right-of-way between conflicting streams. However, traffic demand, vehicular capabilities and driver performance are a few of the variables over which he has little influence.

As in other disciplines, many problems can be reduced to finding the maximum or minimum of some function. Thus, the determination of the number of traffic lanes is contingent on finding the maximum rate of flow (capacity) whereas traffic signal cycle apportionment may be based on minimizing delay. If the mathematical model chosen enables one to compute precisely what will happen to one variable if a specified value is chosen for another variable, the model is said to be deterministic. This may be contrasted with stochastic models in which allowance is made for the probability of a variable assuming various values.

Generally, any sequence of experiments that can be subjected to a probability analysis is called a stochastic, or random, process. Such a process can be either independent or Markov. An independent process varies when experiments are performed in such a way that the outcome of any one experiment does not influence the outcome of any other experiment. If the outcome of any particular trial or experiment depends on the outcome of the immediately preceding trial, it is called a Markov process.

The deterministic aspects of vehicular traffic may be explored by studying a single unit of traffic or the traffic stream as a whole. The effect of the motion or headway of one vehicle on another vehicle is referred to as the local or microscopic properties of traffic. On the other hand, the relation between traffic flow, over an extended period of time, to traffic concentration, over a portion of the roadway, defines the overall or macroscopic properties of traffic. In the energy model discussed in this report, the term microscopic applies to aggregate conditions or behavior in the system.

Most situations in nature are so complicated that they cannot be dealt with exactly by mathematics. The regular procedure is to apply mathematics to an ideal situation having only important features of the actual one. The results are approximations having a practical importance which depends on the closeness of the approximation as verified by reasoning and experiment. The deterministic approach to traffic theory involves analyzing the pertinent traffic characteristics, devising a theory, and then applying methods in the development of which differential equations usually play a prominent role. In this treatment the relationships obtained by observation, experimentation and reasoning are given; the researcher is required to express them in mathematical symbols, solve the resulting differential equations, and interpret the solutions.

Heat Flow Analogy. —Consider the problem of one-dimensional heat flow in a long slender insulated rod satisfying the differential equation

$$\frac{\partial \theta}{\partial t} = a \frac{\partial^2 \theta}{\partial x^2} \tag{1}$$

where θ is the temperature, x is the distance, t is time and $a = C/sk$ where C is the conductivity of the material, s the specific heat and k the density. The boundary conditions are statements of the initial conditions at the ends of the rod at any time, and the initial conditions throughout the rod at time zero. What is usually desired is a description of the temperature at any time and place along the rod, or, in other words, a solution of the differential equation expressed in the form

$$\theta(x, t) = f(L, a, x, t) \tag{2}$$

On the other hand, if a continuous record in time were kept of the temperature at various points along the rod, one could solve for a and thus determine some property of the material such as its conductivity, C.

In many traffic situations, a single traffic lane acts as a long, slender, insulated rod (controlled access and no opportunity for lane change). If θ in Eq. 1 is allowed to assume the role of some parameter related to such conventional traffic variables as speed, concentration and flow, the solution, Eq. 2, provides the means for evaluating some property of the highway—call it the "trafficability." All that need be done would be to measure the traffic characteristics along the stretch of highway defined by L.

Harr and Leonards (14) postulate, for example, that the movement of traffic results from a motivating "pressure potential" analogous to θ in Eq. 1. In their solution the parameter θ was eliminated, leaving vehicular speed as a function of "trafficability," thus affording a means of rating such various geometric features as curve, grade, entrance, ramp, etc.

The difficulty in utilizing the deterministic approach is not in solving a differential equation, but in finding one that expresses the physical condition realistically. For example, Harr and Leonards (14) define θ so that

$$\frac{\partial \theta}{\partial x} = -c_1 u \tag{3}$$

and

$$\frac{\partial k}{\partial \theta} = c_2 k \tag{4}$$

where u and k are speed and concentration. Solving Eqs. 3 and 4 simultaneously would suggest the following speed-concentration relationship:

$$u = -(c_1\ c_2\ k)^{-1}\ \frac{\partial k}{\partial x} \tag{5}$$

This implies that speed might be negative if the change in vehicular density with respect to distance is increasing. This not logical.

Fluid Flow Analogy.—It seems more realistic to express an equation of motion (Eq. 5) in terms of acceleration rather than velocity or speed, since the sign (positive or negative) would not specify forward or backward movement, but merely speeding up or slowing down. Consider the following equation of motion (15) which expresses the acceleration of the traffic stream at a given place and time as

$$\frac{du}{dt} = \frac{-c^2}{k}\ \frac{\partial k}{\partial x} \tag{6}$$

This states that a driver adjusts his velocity at any instance in accordance with the traffic conditions about him as expressed by $k^{-1}\ \partial k/\partial x$. If traffic is thinning out (negative $\partial k/\partial x$), the driver accelerates (positive du/dt) and vice versa.

Equation 6 is of the form of the equation of motion of a one-dimensional compressible fluid with a concentration k and a fluid velocity u (10). This is illustrative of several new theories of traffic flow described in terms of fluid or hydrodynamic flows. Their analyses are based on a partial differential equation expressing the conservation of matter and an assumed relation between flow and concentration such as Eqs. 5 and 6.

In fluid mechanics, fluids are commonly divided into liquids and gases. Their chief differences are that liquids are practically incompressible whereas gases are compressible and must be so treated. A single stream of traffic offers a striking analogue to the flow of a compressible gas in a constant-area duct. Both consist of discrete particles—individual molecules in the case of a fluid, and individual vehicles in the case of the traffic stream.

Lighthill and Whitham (16) applied fluid dynamic principles to various highway occurrences and concluded that discontinuities in traffic flow are propagated in a manner similar to shock waves in the theory of compressible fluids. Greenberg (15) developed the fluid dynamic analogy still further, resulting in functional relations for the basic interactions between vehicles. Herman and Potts (17) emphasized the macroscopic nature of the traffic quantities, flow and concentration, by referring to the somewhat analogous properties of a gas, such as pressure, volume and temperature. Thus, pressure can be computed from the average number of molecular collisions on the containing wall over a long interval of time. Since the relation between the pressure, volume and temperature is called the gas equation of state, the relation between traffic flow and traffic concentration has come to be known as the traffic equation of state.

Because of the widespread interest in recent years in high-speed gas flow, the dimensionless parameter called the "Mach number" has become very significant in fluid dynamics. The Mach number is the ratio of the actual fluid velocity to the acoustic velocity or velocity of sound propagation, under the conditions of the flow where the velocity in question is measured. Following the logic of compressible flows wherein local sonic speed represents the condition of maximum flow per unit area (18), the critical velocity for traffic flow corresponds to the optimum velocity, u_m, or the velocity of traffic at capacity, q_m.

The analogy between fluid dynamics and traffic dynamics is seemingly endless. For example, just as the gas equation of state can be derived from the microscopic law of molecular interaction for two molecules, it has been shown that the traffic equation of state can be derived from the microscopic car-following law governing the motion of two cars (19).

In a previous publication (20), utilizing the hydrodynamic analogy, an energy-momentum concept of traffic flow was formulated in which momentum ku was likened to traffic flow q, and the kinetic energy of the traffic stream was defined as ku^2. Some interesting comparisons based on optimizing both momentum and kinetic energy were made. It was further suggested that acceleration noise might represent the internal energy or lost energy of the traffic stream. This research deals with the theoretical and practical implications of the acceleration noise parameter and the energy model, as related to the level-of-service concept.

Objectives

The objectives of this research fall into three phases: (a) theory formulation, (b) measurement of appropriate traffic characteristics for theory verification, and (c) recommendations for application. They include:

1. The formulation of a complete "energy" model of traffic flow which includes both "kinetic energy" and "internal energy."
2. Measurement of the acceleration noise on the Gulf Freeway to test the hypothesis that acceleration noise represents the "internal energy" of a traffic stream.
3. Determination of the effects of such geometrics as grades of the facility on acceleration noise.
4. Determination of the effects of operational control procedures such as ramp metering on acceleration noise.
5. Recommendations for application of energy parameters in freeway design and operations.

6. The application of the energy concept to the quantitative description of freeway level of service.

DEVELOPMENT OF ENERGY MODEL

Flow Oriented Parameters

Fluid mechanics is based on, among other things, the principle of conservation of mass. Imagine a volume in space: if the outflow of mass is greater than the inflow, the principle of conservation of mass requires that there is an equal decrease of mass stored in the volume. When this principle is stated mathematically as an equation, it is called the equation of continuity.

Considering traffic flow as a conserved system, the change in the number of vehicles on a length of road dx in an interval of time dt (Fig. 5) must equal the difference between the number of vehicles entering the section at x and the number of vehicles leaving the section at x + dx. If the number of vehicles on the length dx at time t is kdx and the number of vehicles entering in time dt at x is expressed as qdt, the conservation of vehicles can be expressed symbolically as

$$kdx - \left(k - \frac{\partial k}{\partial t}\,dt\right) dx = qdt - \left(q + \frac{\partial q}{\partial x}\,dx\right) dt \tag{7}$$

Making use of q = ku yields the equation of continuity for traffic flow

$$\frac{\partial k}{\partial t} + \frac{\partial (ku)}{\partial x} = 0 \tag{8}$$

If it is assumed that drivers adjust their speed in accordance with the traffic conditions about them as expressed by the general expression $k^n\,\partial k/\partial x$, the equation of motion of the traffic stream in terms of acceleration becomes

$$\frac{du}{dt} = -c^2 k^n \frac{\partial k}{\partial x} \tag{9}$$

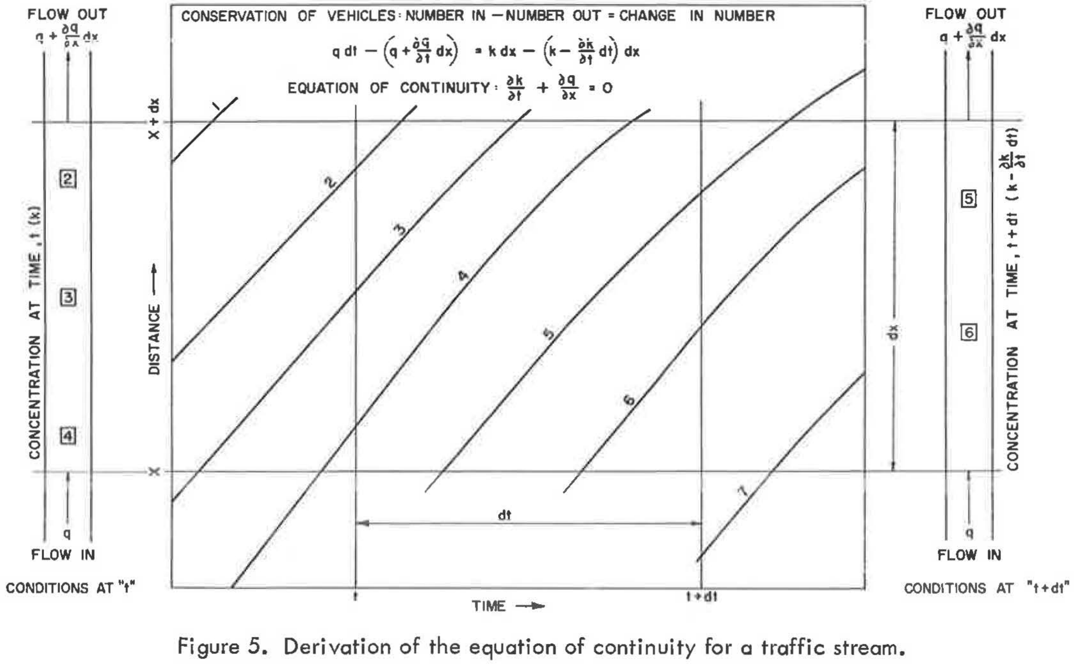


Figure 5. Derivation of the equation of continuity for a traffic stream.

Solving Eqs. 8 and 9 for $u = f(k)$ and utilizing $q = ku$ yields the generalized equation of state for a traffic stream

$$q = ku_f \left[1 - \frac{k^{(n+1)/2}}{k_j}\right] \quad n > -1 \tag{10}$$

where u_f is the free speed and k_j is the jam concentration. The exponent of proportionality n has been shown to provide flexibility in fitting a theoretical flow-concentration curve to a particular highway (1).

The concentration values and speed for which flow is a maximum are obtained by setting the derivation of $q = f(k)$ and $q = f(u)$ equal to zero:

$$k_m = [(n + 3)/2]^{-2/(n + -1)} \; k_j, \quad n > -1, \tag{11}$$

$$u_m = n + 1/(n + 3) \; u_f, \quad n > -1. \tag{12}$$

TABLE 1

CORRESPONDENCE BETWEEN PHYSICAL SYSTEMS

Characteristic	Hydrodynamic System	Traffic System
Continuum	1-dimensional compressible fluid	Single-lane traffic stream
Discrete Unit	Molecule	Vehicle
Variables	Mass density, ρ Velocity, v	Concentration, k Speed, u
Equations		
Continuity	$\frac{\partial\rho}{\partial t} + \frac{\partial(\rho v)}{\partial x} = 0$	$\frac{\partial k}{\partial t} + \frac{\partial(ku)}{\partial x} = 0$
Motion	$\frac{dv}{dt} + \frac{c^2}{\rho}\frac{\partial\rho}{\partial x} = 0$	$\frac{du}{dt} + c^2 k^n \frac{\partial k}{\partial x} = 0$
Momentum	$\frac{\partial(\rho v)}{\partial t} + \frac{\partial(\rho v^2 + \rho c^2)}{\partial x} = 0$	$\frac{\partial(ku)}{\partial t} + \frac{\partial(ku^2 + k^{n+2} c^2/n + 2)}{\partial x} = 0$
State	$P = c\rho T$	$q = ku$
Parameters	Critical velocity, V_c	Critical speed, u_m
	———	Critical concentration, k_m
	Critical flow, Q_c	Capacity flow, q_m
	Momentum, ρv	Flow, ku
	Kinetic energy, $\rho v^2/2$	Kinetic energy, αku^2
	———	Optimum speed, u'_m
	———	Optimum concentration, k'_m
	———	Optimum flow, q'_m
	Internal energy, ϵ	Internal energy, σ
	Friction factor, f	Natural noise, σ_n

Using the compressible fluid analogy wherein the conditions at maximum flow are termed "critical," we shall refer to k_m and u_m as the critical concentration and critical speed of the traffic stream. Their product q_m is the maximum flow of which the highway lane is capable, the critical flow, or capacity. These critical values are the experimentally determined maximums, and should not be confused with a statistical "extreme value."

Momentum-Kinetic Energy

The speed of waves carrying continuous changes of flow through the stream of vehicles is given by the derivative q' of the q-k equation defined in Eq. 10. Discontinuities in traffic flow are propagated in a manner similar to shock waves in the theory of compressible fluids. The speed of a shock wave U is given by the slope of the chord joining the two points on the flow-concentration curve which represent the conditions ahead of and behind the shock waves. Application of the mean value theorem suggests that the speed of the shock wave is approximately the mean of the speeds of the waves running into it from either side:

$$U = 1/2(q'_1 + q'_2) \tag{13}$$

The very strong analogy between traffic flow and fluid flow suggests that the conditions of continuity of momentum and energy should be fulfilled at the surface of a traffic shock wave, just as the equations of dynamic compatibility must be fulfilled in fluid dynamics. Multiplying Eq. 8 by u and Eq. 9 by k, then adding the two equations gives

$$\frac{\partial(ku)}{\partial t} = \frac{-\partial[ku^2 + k^{n+2}\, c^2/(n+2)]}{\partial x} \tag{14}$$

Equation 14 is the law of conservation of momentum in the differential form as applied to traffic flow. Comparing Eqs. 8 and 14 with the classical forms in hydrodynamics, we can add to the analogy between the fluid and traffic quantities. This correspondence is summarized in Table 1.

The equations of continuity, motion and momentum are identical in the two systems when the exponent of proportionality n is set equal to -1. Because, in classical systems, the conservation of momentum equation serves to establish the form for momentum, the quantity ku in Eq. 14 is defined here as the momentum of the traffic stream. If in fact, traffic momentum is equal to ku, it is apparent that it is also equivalent to traffic flow and that the flow-oriented parameters (u_m, k_m, and q_m) discussed in the previous section are based on optimizing this momentum.

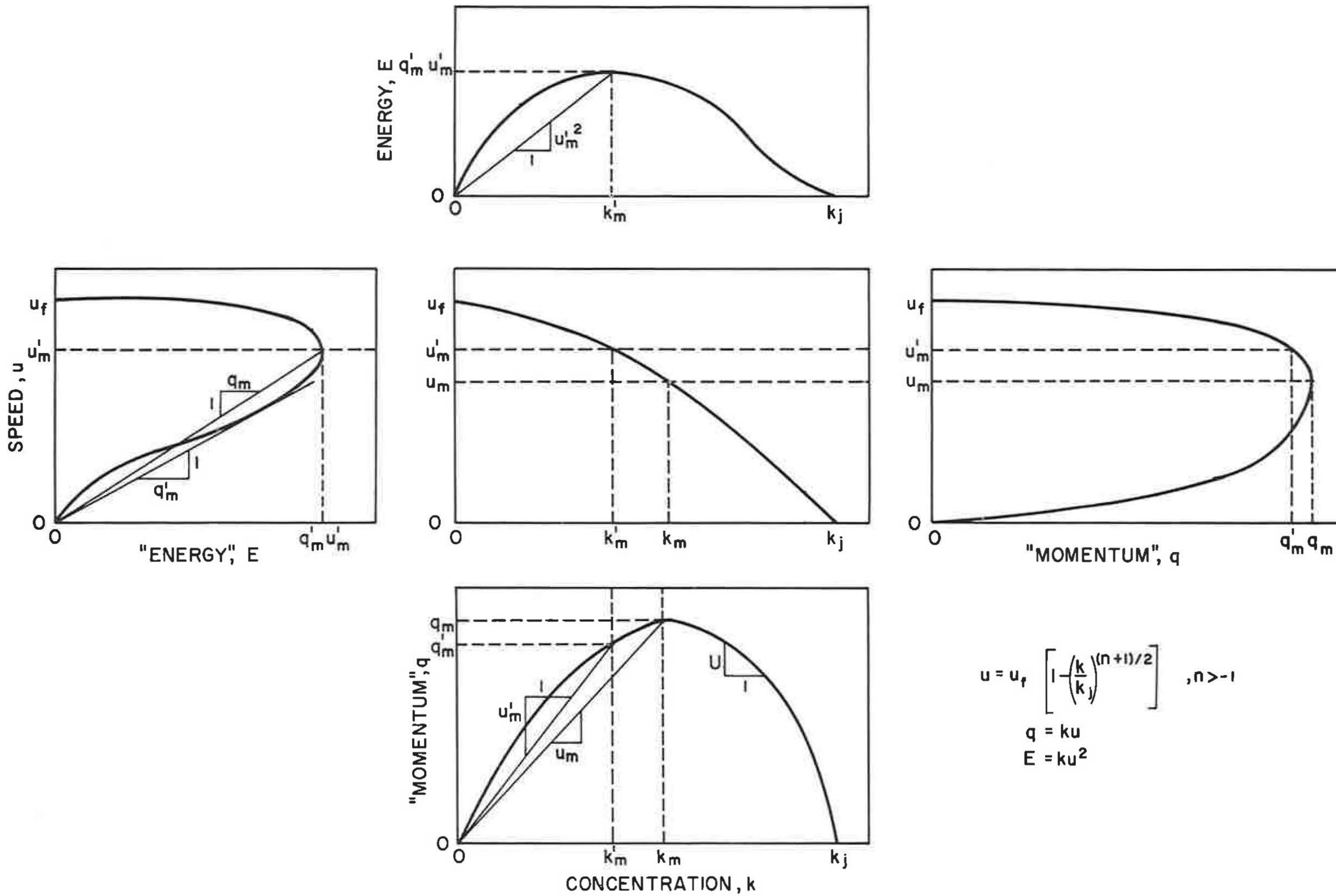
It is well known that the kinetic energy of a fluid is $1/2\ \rho v^2$). However, because of the generalized equation of motion utilized in the formulation of the traffic system model, the kinetic energy of the traffic stream will be defined as αku^2 where α is a constant. Squaring Eq. 10 and then dividing by k yields

$$E = \alpha k u_f^2 \left[1 - 2\left(\frac{k}{k_j}\right)^{(n+1)/2} + \left(\frac{k}{k_j}\right)^{(n+1)}\right],\quad n > -1 \tag{15}$$

The critical depth of flow y in an open channel may be obtained from optimizing either specific momentum M or specific energy E by setting either dM/dy = 0 or dE/dy = 0. Therefore, differentiating Eq. 15 with respect to concentration and speed, dE/dk = dE/du = 0, to get the appropriate "energy" parameters gives:

$$k'_m = (n+2)^{-2/(n+1)}\, k_j,\quad n > -1 \tag{16}$$

$$u'_m = [(n+1)/(n+2)]\, u_f,\quad n > -1 \tag{17}$$



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

$$q = ku$$

$$E = ku^2$$

Figure 6. Relationship between fundamental traffic variables and parameters.

$$q'_m = k'_m u'_m \tag{18}$$

and

$$E'_m = \alpha k'_m u'^2_m \tag{19}$$

In comparing Eqs. 11 and 12 with Eqs. 16 and 17, it is apparent that for vehicular traffic, quite unlike the case of the hydraulics of open channel flow, the parameters obtained from optimizing momentum are not equivalent to the parameters obtained from optimizing energy. The significance of this will be discussed in the applications of the model to level of service. The relationship between the fundamental traffic variables and parameters discussed so far and summarized in Table 1 is illustrated in Figure 6.

Internal Energy

In the same manner as the principle of conservation of mass, the energy conservation law merely states that within the confines of a given system or control section, energy is neither created nor destroyed, although it may appear in several forms (kinetic energy, potential energy, internal energy, etc.), and it may be transformed from one type to another. Energy appears in forms associated with a given mass (kinetic energy) or as transitory energy in the familar forms of heat and work (internal energy).

In classical dynamics, the general energy equation in concerned only with changes in energy content and form, and hence only those forms of energy that undergo change in a given system need be considered. For this reason, in this discussion of energy conservation in a traffic stream we shall obviously not be concerned with chemical, electrical, or atomic forms of energy. The forms of energy that will be considered are kinetic energy and internal energy.

Kinetic energy, ku^2, is the energy of motion of the traffic stream. If, in fact, there is an internal energy or lost energy associated with a traffic stream, it should manifest itself as either lost or erratic motion due to adverse geometrics and traffic interaction. It was seen earlier that the measure of the jerkiness of the driving in this stream is given by acceleration noise. The units of both kinetic energy and acceleration noise are those of acceleration, which adds credibility to the hypothesis that acceleration noise represents internal energy.

The conservation of energy for the traffic stream over a section of road x is simply a case, then, of the total energy T being equal to the kinetic energy E plus the internal energy I of the traffic stream. With this notation, an energy balance or accounting becomes

$$T = E + I \tag{20}$$

$$T = \alpha k u^2 + \sigma \tag{21}$$

where σ represents the acceleration noise for a vehicle on a section of highway of length x.

It is common to say that there is a loss of energy due to the effects of friction in a system, whether it is a classical system or traffic system. The general energy equation tells us that there can be no loss in energy from the system if the principle of conservation of energy is to hold true. Reexamination of the energy equation will yield a clue as to the true meaning of friction. Energy is not really lost, it is simply converted from one of the mechanical forms (kinetic energy) to internal energy, a thermal form of energy. One recalls from the second law of thermodynamics that the mechanical forms of energy, such as kinetic energy, are more valuable than an equivalent amount of thermal energy or internal energy. This is certainly true in the case of traffic flow. Thus, we can say that the forces of friction (adverse geometrics and traffic interaction) tend to convert the desirable forms of energy (traffic motion) into the less valuable forms (traffic interaction).

Two boundary conditions become important in evaluating the parameter, T and α, in Eq. 21. As the internal energy approaches zero, kinetic energy approaches the total energy of the stream and

$$T = \alpha k'_m u'^2_m = (4/27)\ \alpha k_j\ u_f^2 \tag{22}$$

The converse leads to an expression for total energy in terms of acceleration noise,

$$T = \sigma_{max} \tag{23}$$

where σ_{max} is the maximum acceleration noise. Equating Eqs. 22 and 23, we can express α as

$$\alpha = (k_j\ x)^{-1} \tag{24}$$

where

$$x = \frac{27}{4}\frac{\sigma_{max}}{u_f^2} \tag{25}$$

The units of x are those of feet. Intuitively this would be the length of section, x, over which acceleration noise should be averaged. Test runs yielding such representative values as $\sigma_{max} \sim 2\ \text{fps}^2$, $u_f \sim 60$ mph for x = 500 ft tended to substantiate this interpretation. The significance of α would be the reciprocal of the maximum number of vehicles possible on the section of highway x. Again, this seems logical; looking at Eqs. 20 and 21 it is apparent that the internal energy of the stream is being estimated by a single average vehicle, whereas the kinetic energy is being estimated by all traffic. The parameter α serves to adjust E and I so that their sum is equal to the total energy T which is a constant in keeping with the concept that energy cannot be created or destroyed.

Acceleration noise is the standard deviation of changes in speed. The form of $\sigma = f(u)$ may be deduced from Eqs. 21 and 23

$$\sigma = \sigma_{max} - \alpha k u^2 \tag{26}$$

From Eq. 10 it is apparent that

$$k = k_j\ [1 - (u/u_f)]^{2/(n+1)} \tag{27}$$

Substituting Eq. 27 in Eq. 26 and differentiating Eq. 26 with respect to u gives the vehicular speed u'_m at which the acceleration noise or lost energy is a minimum:

$$u'_m = [(n+1)/(n+2)]\ u_f,\quad n > -1 \tag{28}$$

This is identical to the speed given in Eq. 17 that maximizes the motion or kinetic energy of the traffic stream. For example, letting n = 1 in Eq. 27 and substituting in Eq. 26 gives

$$\sigma = \sigma_{max} - \alpha k_j\ u^2 + \alpha k_j\ u^3/u_f \tag{29}$$

for which the optimum speed would be, from Eq. 28,

$$u'_m = (2/3)\ u_f$$

Finally, σ_{min} would be obtained by using Eqs. 28 and 26

$$\sigma_{min} = \sigma_{max} - \alpha k'_m u'^2_m \tag{30}$$

The total measured acceleration noise σ_T is made up of a certain contribution due to the interaction of traffic σ_t plus the natural noise σ_N due to the geometrics of the facility. The natural noise σ_N could be obtained either by measurements made by a test vehicle under conditions when there was no traffic interaction (say at 3 a.m.) or by fitting a regression line to acceleration noise vs speed data to obtain σ_{min}. The relations between the parameters derived here are summarized in Figure 7.

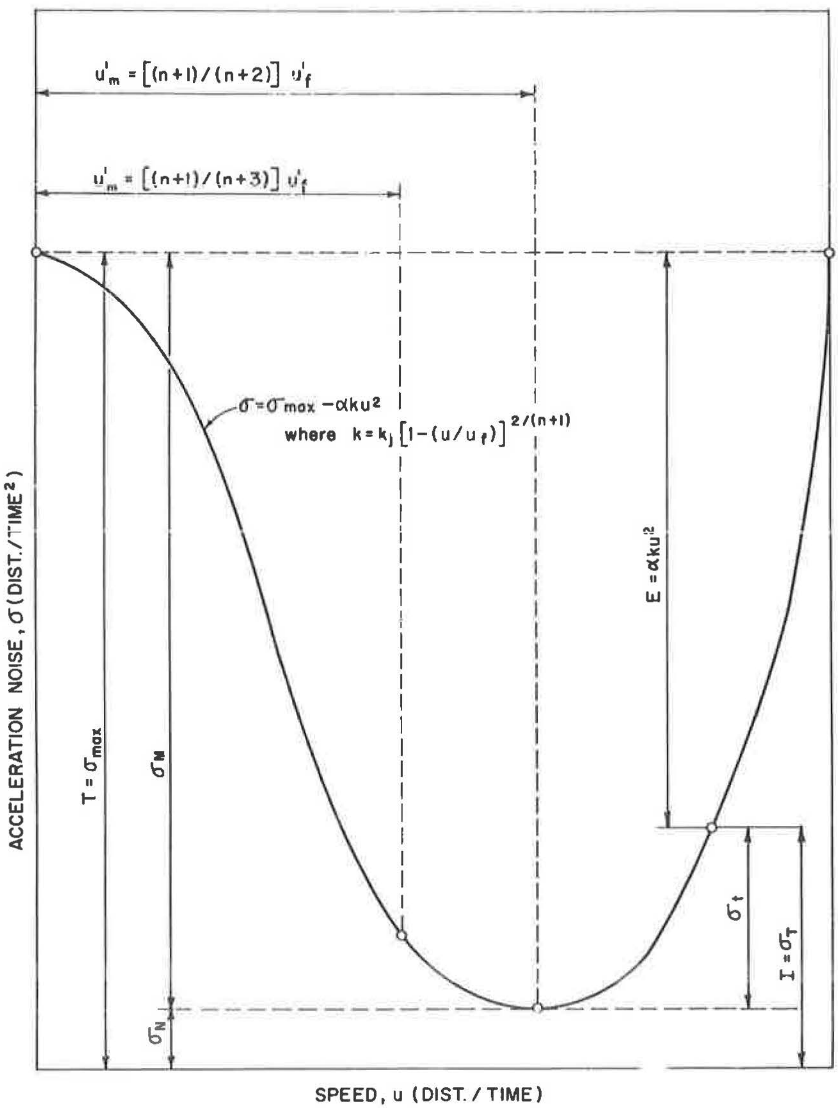


Figure 7. Relations between acceleration noise parameters for internal energy model.

The significance of the internal energy-acceleration noise model developed is that it provides estimates of all the parameters summarized in Table 1 and Figure 7 based on data collected with a single test vehicle equipped with a recording speedometer. The study procedure, data reduction and broad applications of the model are discussed in the following section.

STUDY PROCEDURES

Study Location

The Gulf Freeway in Houston, Texas, is a six-lane divided facility having 12-ft lanes and a 4-ft concrete median with 6-in. barrier type curbs. The freeway is an at-grade type and is carried over the major intersections by grade crossings producing a "roller coaster" effect on the thru lanes.

The study section of the Gulf Freeway (Fig. 8a) begins at the Reveille Interchange at the intersection of highways US 75, State 35 and State 225, and extends to the downtown distribution system. The interchanges within the study area are either full or partial diamond with the exception of the Reveille Interchange and the downtown distribution system which are directional. The distribution system consists of two four-lane one-way streets and thus affords high capacity movement. Frontage roads in the study section are continuous except at two locations for the inbound movement and at three locations outbound.

Method of Study

In the study of traffic flow on a busy freeway, it is necessary to describe the motion of a great number of vehicles. Continuing with the hydrodynamic analogy, this task is similar to that of describing the motion of an infinite number of fluid particles. A quantity such as velocity must be measured relative to some convenient coordinate system. The two methods commonly used in fluid mechanics are the Euler and Lagrangian methods of analysis.

One may choose to remain fixed in space and observe the fluid pass by a given point. This method, whereby a fixed coordinate system is established, is referred to as the Euler method analysis. The Lagrangian method of analysis involves establishing a coordinate system relative to a moving fluid particle as it flows through the continuum and measuring all quantities relative to the moving particle.

Similar methods of analysis may be used to describe a traffic system. They are summarized in Figure 8b. By careful design of a freeway study, point, instant or

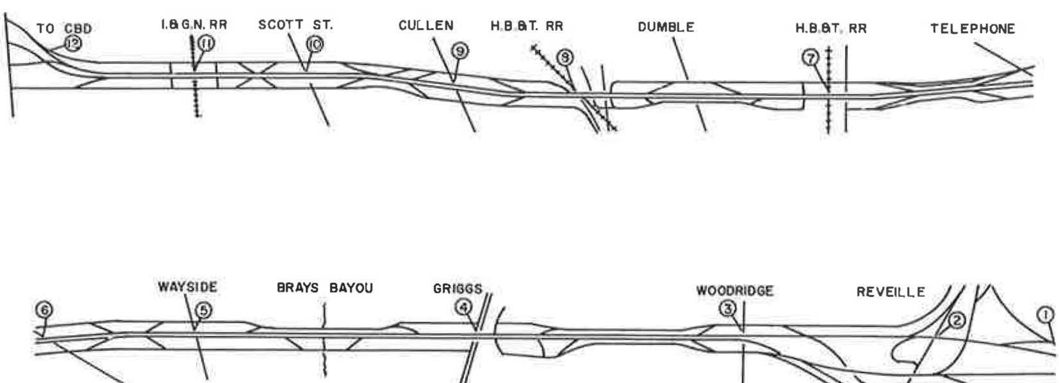


Figure 8a. Station points within the study area.

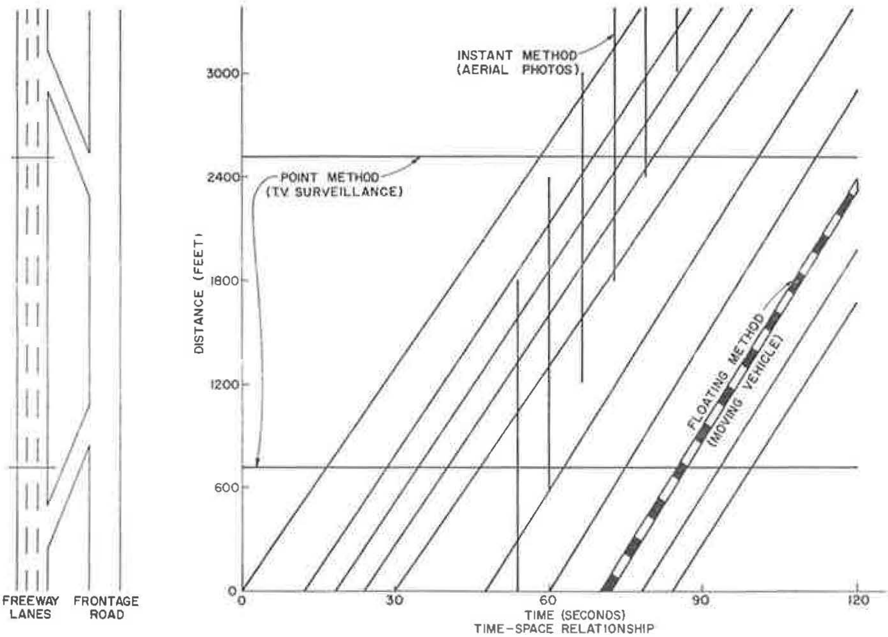


Figure 8b. Illustration of study procedures.

floating studies can be used to give essentially continuous coverage in both time and space (20). The point and instant methods are more applicable to the measurement of the macroscopic properties of the traffic stream. Moving vehicle methods are microscopic in nature, and quite analogous to the Lagrangian method of analysis used in fluid mechanics.

The use of the floating car method has long been established as providing accurate determination of the average traffic speed on a roadway. This method was also applied in this study with the assumption made that the accelerations of a floating car could also represent a good average of the accelerations of the traffic stream.

The total acceleration noise of vehicles at different locations in a platoon has been measured previously in single-lane experiments (21), and it was noted that traffic broadens the acceleration distribution function of the lead car. It was found that the dispersion down the platoon increases up to about the fifth vehicle, where the noise reaches about three times the lead car level. However, the broadening factor then remains relatively constant down the rest of the platoon. During the study periods, care was exercised to position the test vehicle in a platoon so as to represent the majority of vehicles, i.e., not closer than the fifth vehicle in a platoon during peak period runs so that the full effects of the broadening factor could be measured.

Although there may be some variance in the acceleration noise experienced by different drivers on a given road and traffic conditions, this variance was not measured as part of this study. The variance was eliminated, however, by employing the same motorist throughout this study.

To accomplish the objectives of this study, speed recordings were made:

1. During off-peak traffic flow periods (about midnight) to determine the effects of such geometrics as grades of the facility on acceleration noise.

2. During peak periods of traffic flow to evaluate the effects of traffic interaction and to document the congestion and levels of service on the facility.

3. During on-ramp control studies to determine the difference in the smoothness of flow between normal and controlled operation.

Natural Acceleration Noise.—To obtain a means of rating the various geometric features of a freeway using the acceleration noise parameter, it is necessary to eliminate the effects of traffic interaction. Thus, one method of obtaining the natural acceleration noise on a facility would be to drive the test vehicle on the facility in the absence of traffic. However, since this feat is almost an impossibility on the Gulf Freeway, it was decided to record the data at about midnight on several days. The extremely light volumes at this hour reduced the probability of any traffic interference with the test vehicle which would affect the natural noise measurement. Twenty runs were made for this measurement. Data recorded in freeway sections where the speed of the test vehicle was influenced by traffic interference were discarded.

Traffic Interaction.—Eighty-one runs were made during periods between 6:20 and 8:20 a.m. over the course of 16 days on the inbound facility to evaluate the effects of acceleration noise due to freeway traffic interaction. During these 2-hr periods, it was possible to make one complete round trip approximately every 20 min. The starting times of each run were altered daily so that the typical cross section of the total period could be measured.

An on-ramp control study (22) initiated on the Gulf Freeway in August 1964 afforded an excellent opportunity to measure the acceleration noise due to traffic interaction during controlled conditions and to compare thereby the results with those obtained during normal operation. The control area with a summary of the control plans is shown in Figure 9. Runs were made on the inbound lanes in similar fashion to the uncontrolled periods.

Equipment

In previous studies of acceleration noise, two different methods of recording the necessary data to calculate the parameter were employed. In one study (11), an accelerometer mounted in the test vehicle was photographed and σ was determined by analysis of an acceleration-time curve. Other researchers (13), realizing that this method was too time consuming, utilized a tachograph and recorded the speed of the test vehicle as it progressed in the traffic stream. Basic equations of motion were employed to approximate the formula for acceleration noise.

The tachograph had several advantages; however, a recording speedometer manufactured by Esterline-Angus (Fig. 10), was more suitable for the study on the freeway

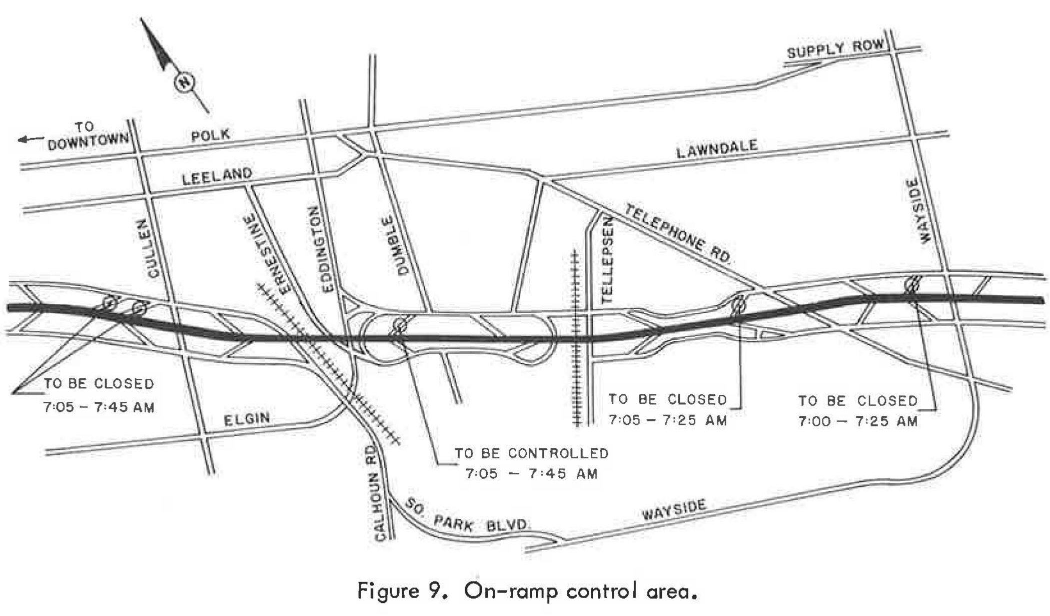
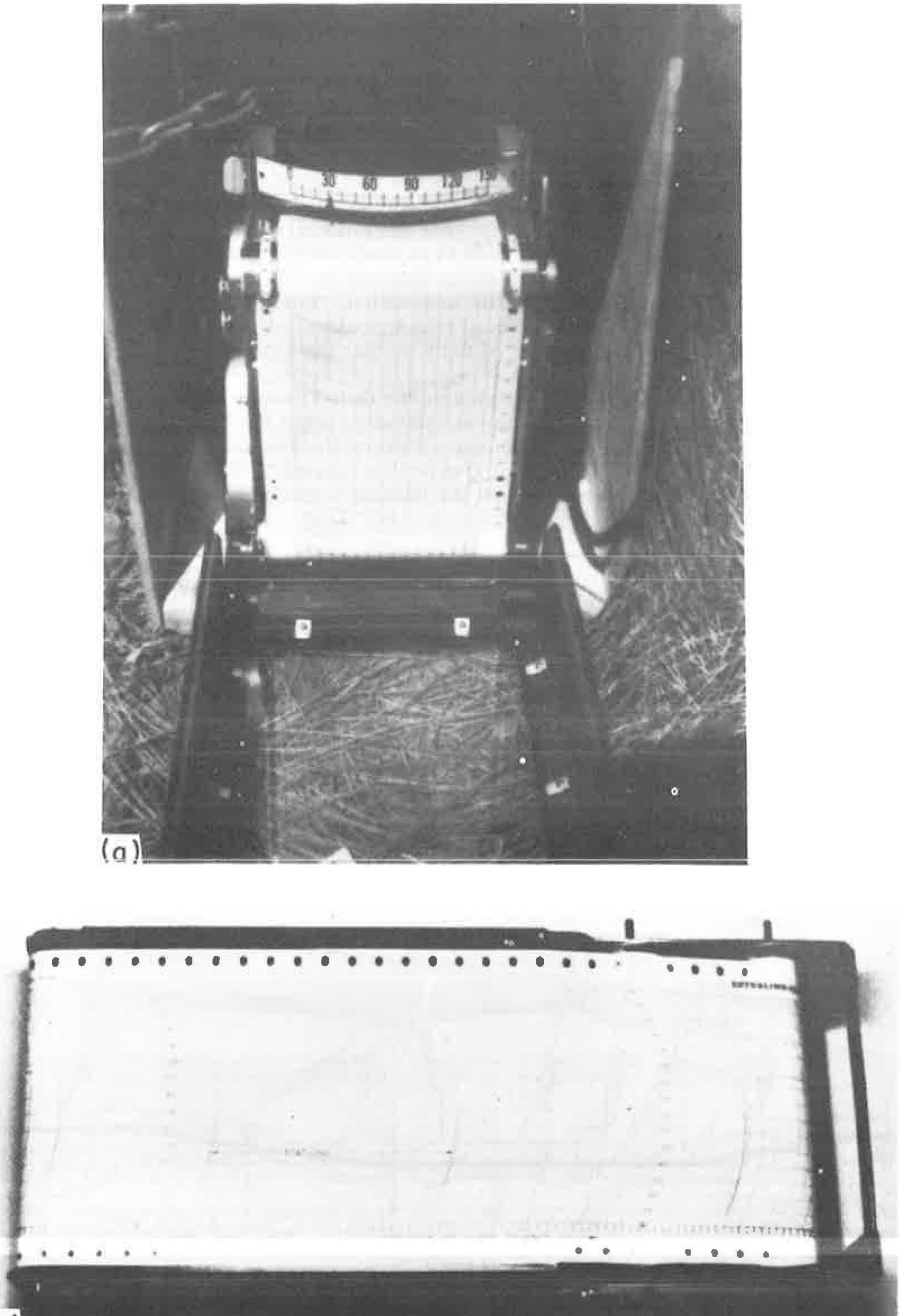


Figure 9. On-ramp control area.



(b)

Figure 10. (a) Speed recorder; (b) chart and chart inspector.

because of its flexibility. This recorder is connected by cable to the transmission of the vehicle in similar manner to the odometer and is capable of recording an analogue graph of the vehicle's speed in the traffic stream. In addition, a special event mark, which is indicated by either an upward or downward sweep of the speed recording pen, was used to identify certain reference station points along the route. These stations, which in most cases were the center of each grade separation of the freeway, were marked manually through a switching box connected to the speed recorder. The reference stations are shown in Figure 8. A second pen recorded 100-ft distance marks automatically.

The chart speed was also very flexible. A set of gears accompanying the speed recorder allowed a varied selection of chart advance rates. For this study, a chart advance rate of 1 in. per 10 sec was used to allow a more accurate reading of time, which was one of the two measured variables.

The flexibility of the equipment permitted modifications which also increased the reading accuracy of the second variable speed. A rheostat on the switching box was electronically connected to the recorder in such a fashion as to vary the range of the vertical scale. The normal chart scale is graduated in increments of 2 mph. However, the rheostat permitted the use of 1 mph increments with a full scale deflection of 75 rather than the normal 150 mph.

Data Reduction

Choice of Speed Increment. —From these speed profiles obtained with the test vehicle, the acceleration noise was easily estimated using Eq. 46 in the Appendix. A transparent template graduated in 1.0-sec increments was placed over the speed graph to determine Δt to the nearest 0.5 sec for every Δv. The term "$1/\Delta t$" was accumulated on a calculator for the entire section under study. Thus this method provided a fast means of reducing the data.

The approximating equation for acceleration noise (Appendix) is formulated on the principle that the speed increment Δv is constant and could be used throughout the evaluation of a speed-time curve while Δt is measured for every Δv. It is obvious that a selection of two separate values of Δv could result in different values for the approximation of σ. For instance, if Δv were chosen as 2 mph, the measurement would be much more sensitive than if it were taken to be 6 mph. It is conceivable that a traffic stream could experience several speed fluctuations of up to 5 mph without exceeding the 6 mph limit. The resulting calculations would produce values of σ when $\Delta v = 2$ is used, but no values with $\Delta v = 6$. Some aspects and comparisons regarding the selection of Δv for the data reduction are discussed in the Appendix.

The constant value Δv was given careful consideration. One would not expect a motorist to maintain an absolutely uniform speed but would expect minor fluctuations. It also seems apparent that a motorist does not even sense a speed change of 2 mph or less and therefore this degree of fluctuation would not effect the quality of his trip.

Based on the limitations of the equipment and data reduction methods employed in this study, an accuracy of no more than 2 mph could be expected. A constant of $\Delta v = 2$ was therefore used throughout this study. For all practical purposes then, acceleration noise was calculated to be zero unless a speed change of 2 mph occurred.

Acceleration Noise on a Short Section of Freeway. —The fundamental equation for estimating acceleration noise from the Appendix (Eq. 44) is

$$\sigma^2 = \left[\frac{(\Delta v)^2}{T} \sum_{i=0}^{T} \frac{n^2}{t_i}\right] - \left[\frac{(V_T - V_0)}{T}\right]^2 \tag{31}$$

It has been emphasized that on a long stretch of roadway the last term in this equation is zero since in most cases the starting speed and the ending speed are the same. In other cases T is very large which makes the second term insignificant. When an investigation is made of a short section of freeway (say 500 ft), however, it is not uncommon to have conditions where the speed at the beginning of the section varies considerably with the speed at the end of that same section. In these cases, the second term of the equation is significant and therefore, according to the definition of acceleration noise, it must be considered in the calculation.

Dudek (23) derived a general expression for estimating acceleration noise which does not depend on the average acceleration being zero (Appendix). It is based on finding the acceleration noise about the origin rather than the mean. The relative acceleration noise for short sections of roadway using this approach becomes

$$\sigma_0 = \left[\frac{(\Delta v)^2}{T} \sum_{i=0}^{T} \frac{1}{\Delta t_i} \right]^{1/2} \tag{32}$$

Equation 31 is identical to Eq. 32 ($V_T = V_0$), which defines acceleration noise for a relatively long trip. This identity makes it possible to use a simple systematic means of evaluating σ on any desirable roadway section of any length based on calculations from successive short sections. This property is discussed in the next section.

Additive Property of Acceleration Noise. —Suppose acceleration noise data were obtained over a long continuous roadway section. The acceleration noise on short successive sections (say 500 to 1000 ft) could be calculated using Eq. 32. If it should be desirable to increase the length of the study sections for reasons of evaluating a larger problem area, σ could easily be determined from the data readily available for the short sections. The additive property of the standard "variance" (standard deviation squared) permits the "pooling" of numerators and denominators in the equation (24). Therefore, the data for the variance on the accelerations, or acceleration noise squared,

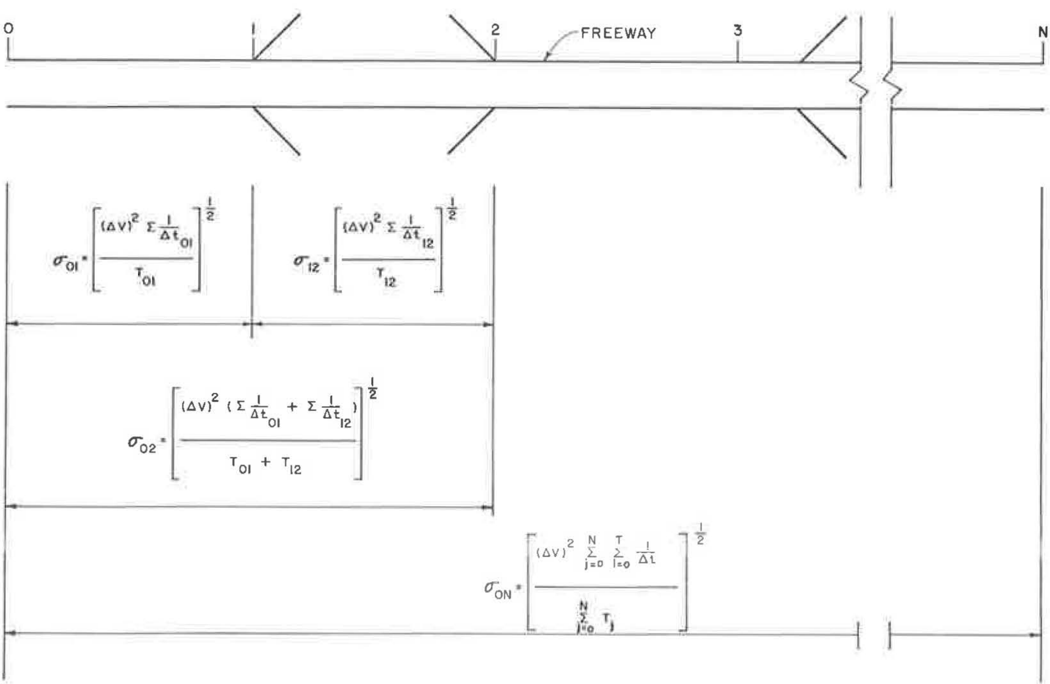


Figure 11. Schematic showing the additive property of acceleration noise.

can be pooled. For example, in Figure 11, if the values of σ were known from 0 to 1 and from 1 to 2, σ from 0 to 2 becomes

$$\sigma_{02} = \left\{ \frac{(\Delta v)^2 \sum_{i=0}^{1} \frac{1}{\Delta t_i} + (\Delta v)^2 \sum_{i=1}^{2} \frac{1}{\Delta t_i}}{T_{01} + T_{12}} \right\}^{1/2}$$

which can be written as

$$\sigma_{02} = \left\{ \frac{(\Delta v)^2 \left[\sum_{i=0}^{1} \frac{1}{\Delta t_i} + \sum_{i=1}^{2} \frac{1}{\Delta t_i} \right]}{\sum_{j=0}^{2} T_j} \right\}^{1/2}$$

Generalizing the above equation, if σ has been determined for n successive sections from j = 0, . . . , N, then σ for the entire section is

$$\sigma_{0N} = \left\{ \frac{(\Delta v)^2 \sum_{j=0}^{N} \sum_{i=0}^{T} \frac{1}{\Delta t_i}}{\sum_{j=0}^{N} T_j} \right\}^{1/2} \tag{33}$$

There are at least two important advantages of measuring σ in several successive short sections rather than measuring the noise over the total length of roadway under study. First, one can easily determine locations where problems are inherent due to either geometric deficiencies or congestion. By measuring σ on several short sections, these trouble locations can be isolated for future extensive study. Second, if σ can be related to known parameters such as speed, volume, or concentration, each of which vary between short sections of freeway, then it is also important to measure acceleration noise in short sections.

RESULTS

Natural Acceleration Noise

The objective of determining the natural acceleration noise (σ_N) on the Gulf Freeway was to determine the amount of disturbance to a vehicle's trip on the facility that can be ascribed to a driver's natural speed changes in the absence of traffic. Twenty trips were made on the freeway with the driver instructed to drive at a comfortable speed throughout the 6-mi study section. Although all the test runs were made about midnight, occasionally the speed of the test vehicle was influenced by another vehicle in certain sections. These sections were thus excluded from the total sample.

The natural acceleration noise on the inbound Gulf Freeway for the entire length of the study section was 0.38 ± 0.02 ft/sec^2. Even though the freeway has a rolling grade, the value of the natural noise is not drastically above the value (0.32 ft/sec^2) measured on an almost perfect roadbed (11).

This value, however, could be deceiving if the tangent sections of the freeway were sufficiently long to dampen the effects of geometrics. The intrinsic or natural effects in each of 500-ft continuous freeway sections were therefore determined and are presented in Figure 12.

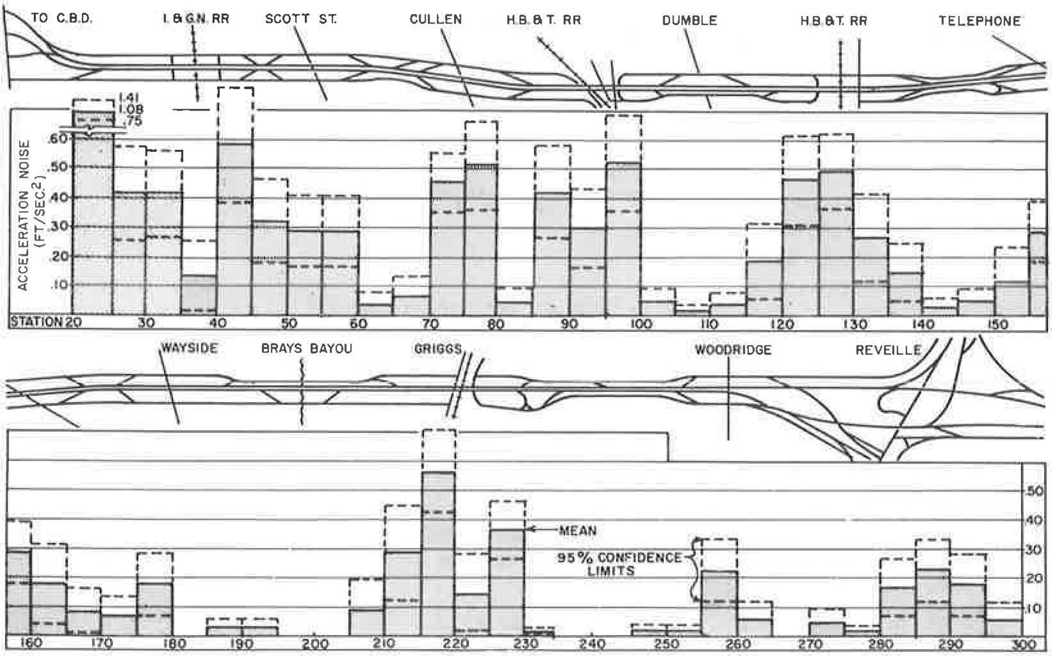


Figure 12. Natural acceleration noise (σ_N).

Each of the roads which are marked in Figure 12 crossing under the freeway represent locations where grades exist with two exceptions. Dumble is not continuous and therefore does not pass under the freeway. Also, Brays Bayou passes under the freeway but well below the existing grade. The remaining roads consitute locations of grades ranging from 1.0 to 5.0 percent.

It is noted that σ_N is approximately zero on the tangent freeway sections. However, the pattern of increased noise is evident on the grades. Further study of the relationship of acceleration noise to grades was made and the results are discussed later. The high values of σ_N at the distribution system, of course, are attributable to the rapid deceleration in anticipation of traffic signals at upcoming intersections.

An important factor in the measurement of σ_N is the variability of the parameter. After 20 test runs on the freeway, the 95 percent confidence limits of the estimate of σ_N was within ± 0.02 ft/sec^2. However, by dividing the freeway into short segments, the variability of σ_N was much higher within the segments. For the majority of the sections the 95 percent confidence limits of the estimate of σ_N was within ± 0.15 ft/sec^2 of the observed mean (Fig. 12).

Evaluation of Geometrics: Grades. —The level of service concept is predicated on the theory of providing the motorist a quality of driving conditions which includes safety and comfort. Jones and Potts (13) have clearly presented the increase of acceleration noise due to winding country roads. It is obvious that on the basis of safety and comfort, the large values of acceleration noise on these roads indicated very poor service to the motorists.

The magnitude of grade effects has usually been documented in terms of capacity reduction with respect to percentage of trucks in a traffic stream. However, little attention has been given to the effects on passenger vehicles. It is generally conceded (25) that passenger vehicles can negotiate long grades up to 7 percent at speeds exceeding 30 mph and that grades up to 7 percent have a negligible effect on the performance of passenger cars.

The level of service concept, however, places emphasis on driver comfort and safety as well as roadway capacity. Accident studies by Mullins and Keese (26) found no significant relationship between the algebraic difference in grade and the frequency of accidents and likewise no apparent relationship between the sight distance on the vertical curves and frequency of accidents. Further evaluation of high accident frequency locations, however, showed that rear-end type accidents accounted for 70 percent of all accidents on high frequency crest-sag locations and that driver tendency to follow too closely "probably" constituted a "primary or a contributing causative factor."

A major reason why motorists follow too close on vertical alignment might be explained by their natural tendency to decelerate upgrade and to accelerate downgrade. Upon reaching the crest, the drivers realize the reduction in speed due to the incline and have a tendency to accelerate downgrade sometimes to speeds beyond their desirable comfortable speeds.

This phase of the investigation tested the hypothesis that the irregular pattern in driving speeds on grades can be described in terms of acceleration noise and that acceleration noise on vertical curves is dependent on the linear combination of the two independent variables of algebraic difference in adjoining grades and length of grade. In other words, it is hypothesized that these two variables contribute to the naturalness of speed irregularities on vertical curves which can be measured in terms of acceleration noise. The dependency of acceleration noise on both or any one of these variables would permit a basic evaluation of vertical curves as related to danger potential.

The data collected to determine the natural acceleration noise were also used to test the hypothesis concerning grades. Acceleration noise over each of the nine grades was determined and the measurements were then averaged to define the mean acceleration noise over each grade. Although only nine grades were located within the study area, six of which were identical, the analyses are presented to illustrate some general trends. The data used for this analysis, confidence limits of the average acceleration noise and statistical tests are set forth in the Appendix.

Although the lack of a sufficient sample size prohibits any definite conclusions, the results in the Appendix, illustrated in Figure 13, are indicative. It is evident that if a relationship between grades and acceleration noise can be further substantiated, the element of grades as related to level of service could be documented in terms of acceleration noise. Although there did not seem to be any direct relationship between grade and accident experience in the study by Mullins and Keese (26), there is indication that there is some relationship between grade and accident potential as defined in terms of acceleration noise.

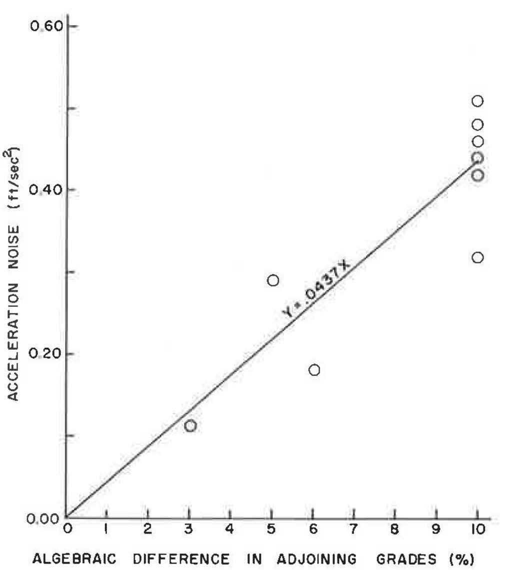


Figure 13. Relationship between algebraic difference in adjoining grades and acceleration noise.

Comparison of σ_{min} and σ_N. —The total measured acceleration noise (σ_T) is the sum of the noise due to traffic interaction (σ_t) and the natural noise (σ_N). Since σ_N is a constant, σ_T becomes a minimum when σ_t is zero. Therefore, the minimum measured acceleration noise (σ_{min}) is equal to σ_N.

As a means of substantiating the above equality, data from each of five freeway sections were used to determine the parameters of Eq. 29. Minimum acceleration noise was then computed and compared to σ_N which was measured over the same sections. Results of an analysis of variance confirmed acceptance at the 0.01 level of the null hypothesis that the difference between the two parameters is zero, that is, $\sigma_{min} = \sigma_N$.

The significance of this equality is the fact that both σ_t and σ_N can be determined

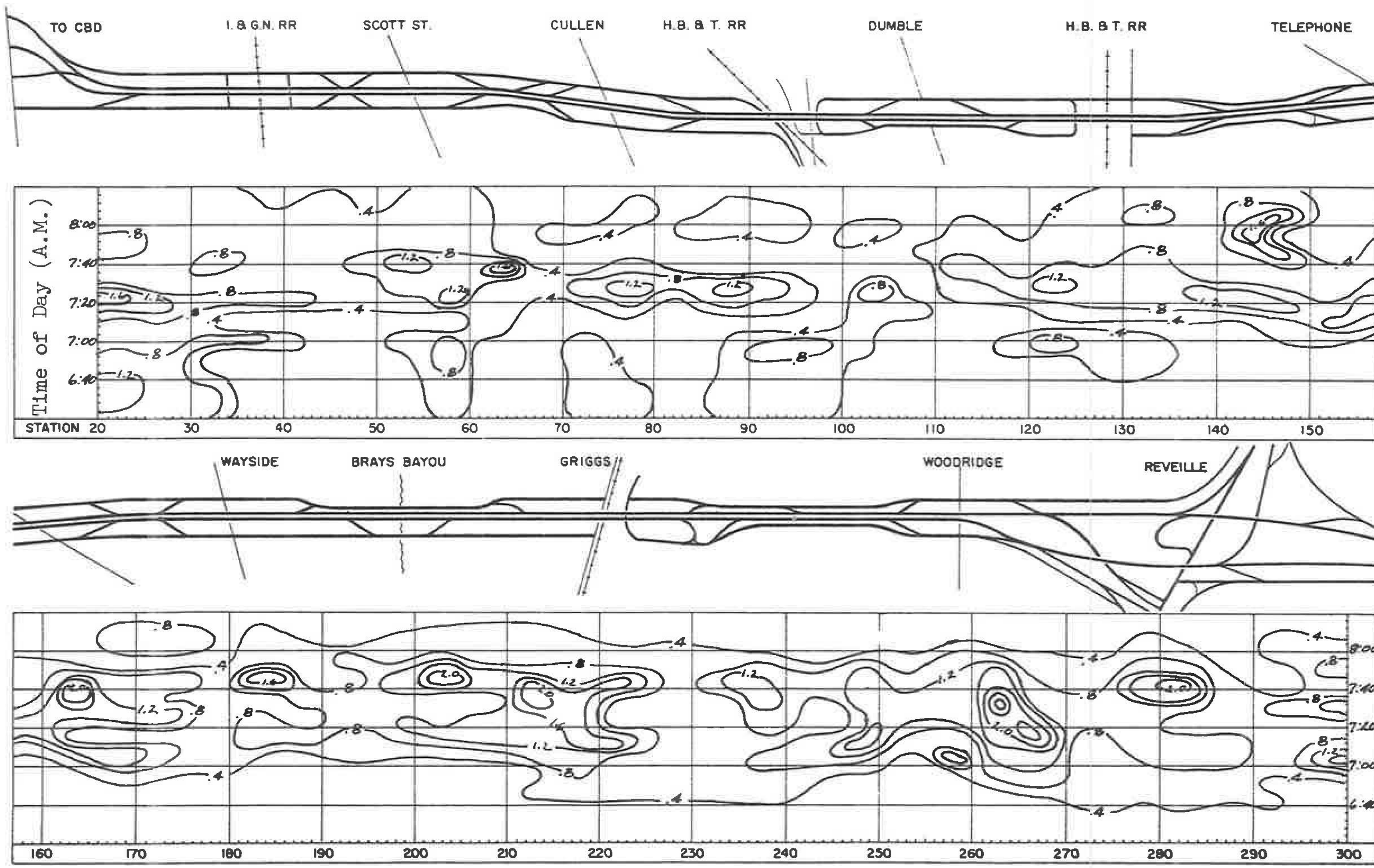


Figure 14. Total acceleration noise (σ_T) contours.

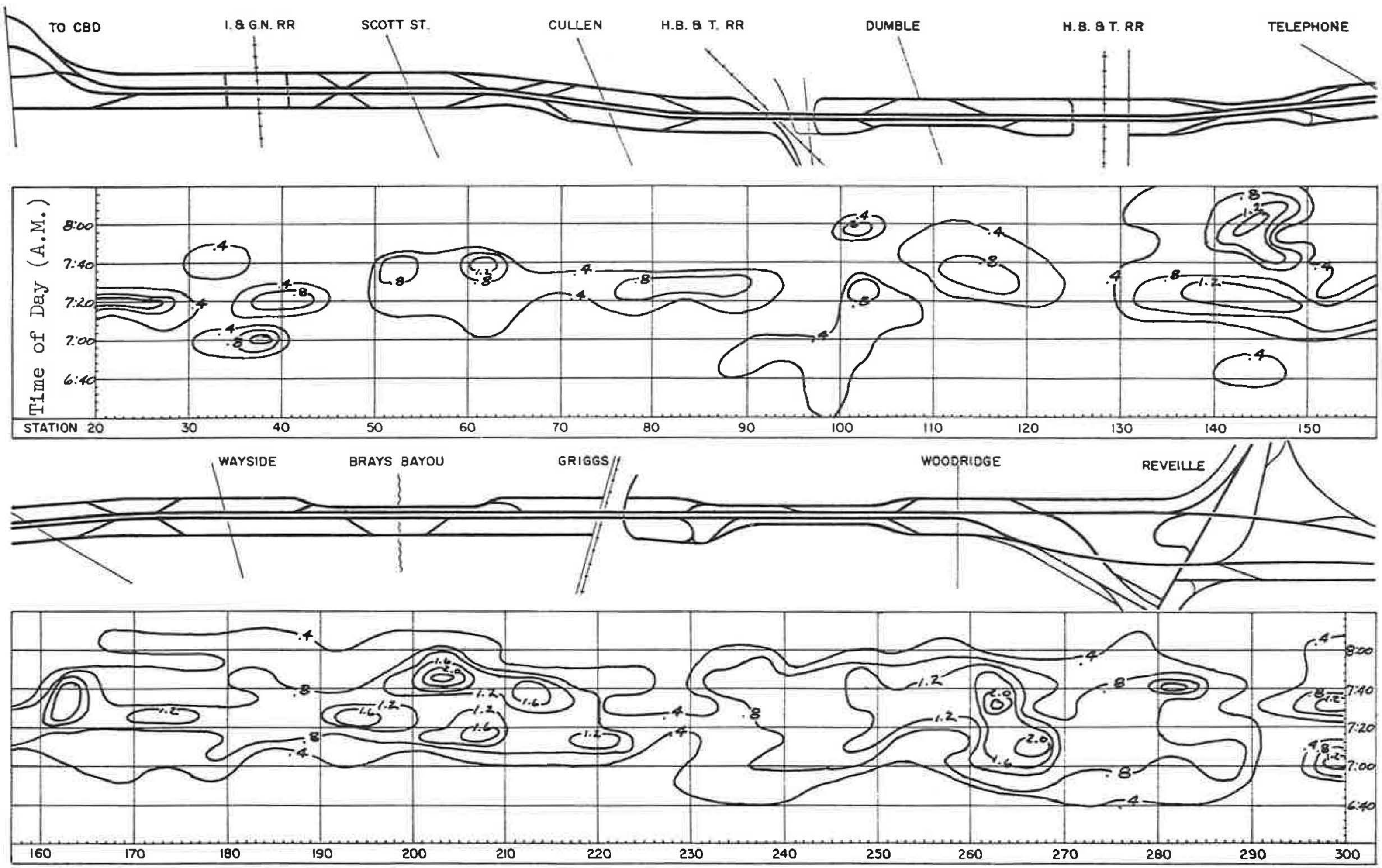


Figure 15. Contours of acceleration noise due to traffic interaction (σ_t).

by only measuring the total acceleration noise in the traffic stream (σ_T). Thus geometric effects can be assessed by measuring the acceleration noise in the traffic stream in lieu of measurements in the absence of traffic (Fig. 7).

Acceleration Noise Due to Traffic Conditions

In this phase of the investigation, the acceleration noise data collected during the morning peak period were reduced for each 500-ft section as previously established for the natural noise phase of this study. Data were collected during periods in which the entrance ramps were being metered (controlled operation) as well as during periods of normal freeway operation.

Normal Operation. —The total acceleration noise on the inbound Gulf Freeway as measured by the test vehicle can best be illustrated in terms of contours (Fig. 14). The acceleration noise due to traffic conditions (natural noise removed) is shown in Figure 15.

Figure 15 reveals that the most violent speed changes as a result of vehicle interaction occur generally from the Reveille Interchange to the south H. B. & T. Railroad overpass. The flow downstream of this area is relatively smooth. Within the former area, three critical areas are predominant: (a) downstream of Telephone Rd., (b) at Brays Bayou, and (c) at Woodridge.

Downstream of Telephone Rd., three conditions exist which could influence the instability of flow. One factor is the existence of a 5 percent grade downstream of the area; the other two factors are the presence of on and off ramps within the area. It cannot be concluded from the available data, however, whether these conditions alone or in part were the sole causes of the large amount of acceleration noise.

The second critical area is in the vicinity of Brays Bayou. The significance of this area is that it is not considered a problem spot. A bottleneck actually exists at the Griggs on-ramp which is located downstream of the Bayou. However, vehicle queues, resulting primarily from inefficiency at the Griggs ramp, usually extend beyond the Bayou during the critical peak traffic period. This condition often extends beyond the time after which the majority of the remaining freeway is moving relatively smoothly. The resulting effect of the queue buildup is an area of rapid deceleration.

The third area of high acceleration noise (Woodridge) can largely be explained by the continuous congestion at the Reveille Interchange. Beyond the bottleneck at highway 35 is an area in which motorists have the opportunity to discharge from the queue but are suddenly met with another stopped platoon. The acceleration noise at this area is ascribed to a combination of both rapid acceleration and deceleration.

The acceleration noise contour map does not actually identify the exact location of bottlenecks; it merely illustrates the danger potential areas which result from critical bottlenecks. High acceleration noise, of course, would normally occur at the end of queues which in many cases are far removed from the actual bottlenecks. However, it would not be difficult to locate the cause of bottlenecks after the critical areas have been located from the acceleration noise contours.

Controlled Operation. —In the past the basic concept of improving freeway operations by ramp control has been to maximize volume throughout with little consideration given to the quality of freeway operation. It is an accepted fact that most freeway bottlenecks can be reduced or eliminated by on-ramp control techniques, but little has been done to measure the qualitative changes due to the controls other than a measurement of travel time. It is possible to reduce the overall travel time and yet create hazardous locations where rapid decelerations occur, far removed from the original bottlenecks. As previously stated, travel time would not reflect these conditions. However, a measurement of acceleration noise could locate any hazardous locations and would indicate whether additional controls would be necessary to maintain smooth operation throughout the study area.

In an attempt to compare acceleration noise before and during the on-ramp control study conducted in the summer of 1964, the data collected on days during which any vehicular incident occurred, such as an accident or stall, were not used. As a result, four days of usable data were averaged. Figures 16 and 17 represent contour maps of

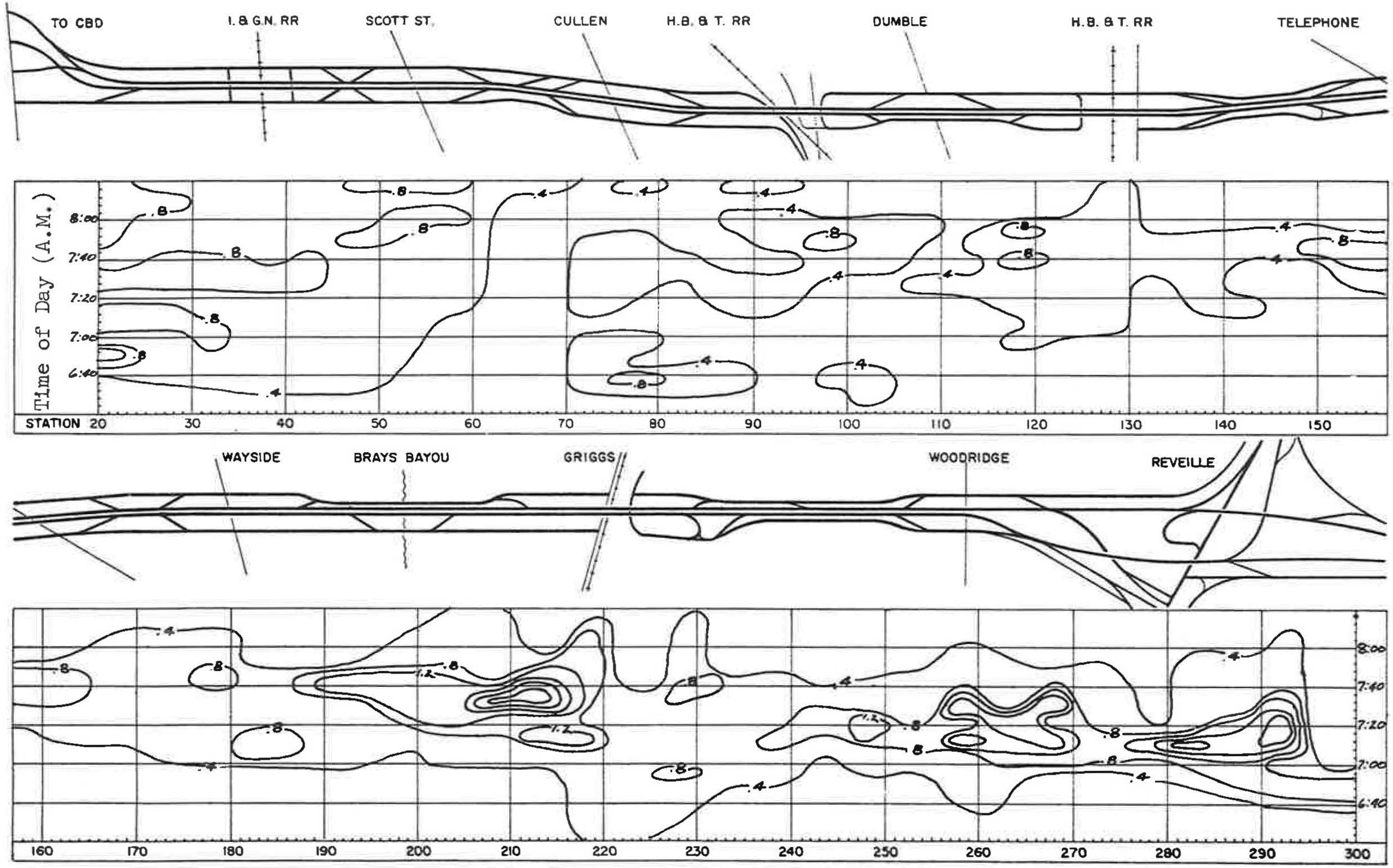


Figure 16. Total acceleration noise (σ_T) contours—control study.

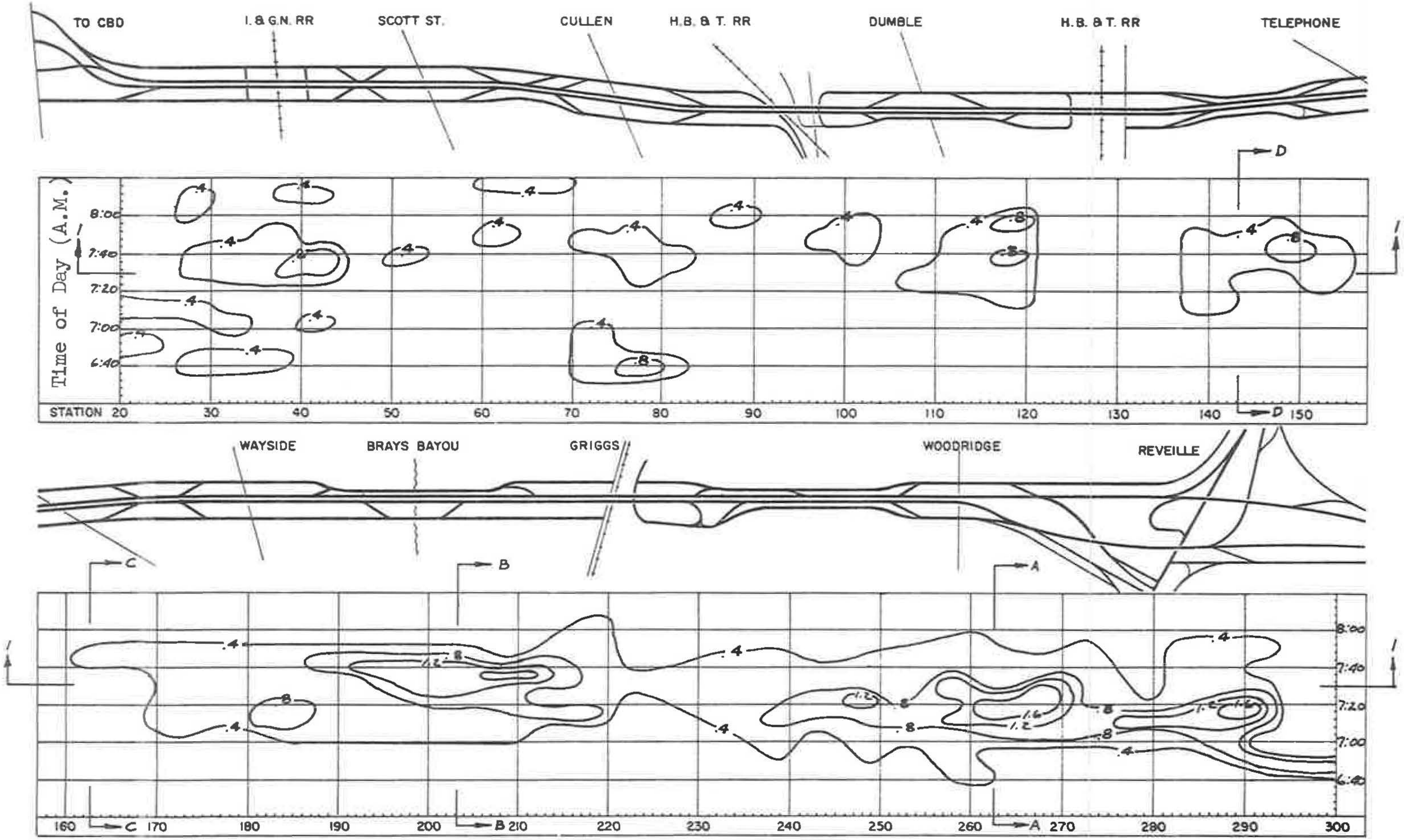


Figure 17. Contours of acceleration noise due to traffic interaction (σ_t)—control study.

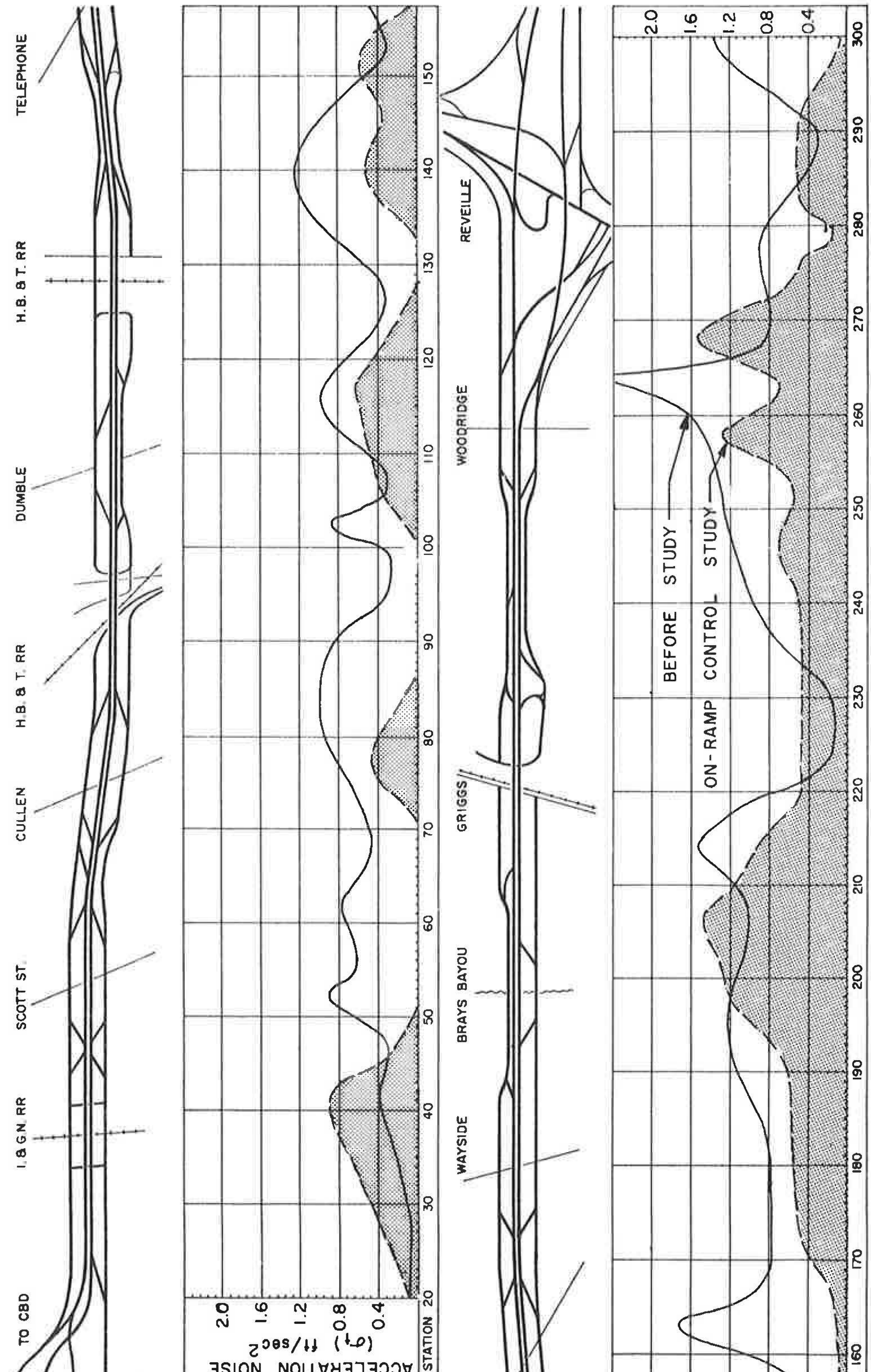


Figure 18. Acceleration noise profiles—7:30 a.m., Section 1-1.

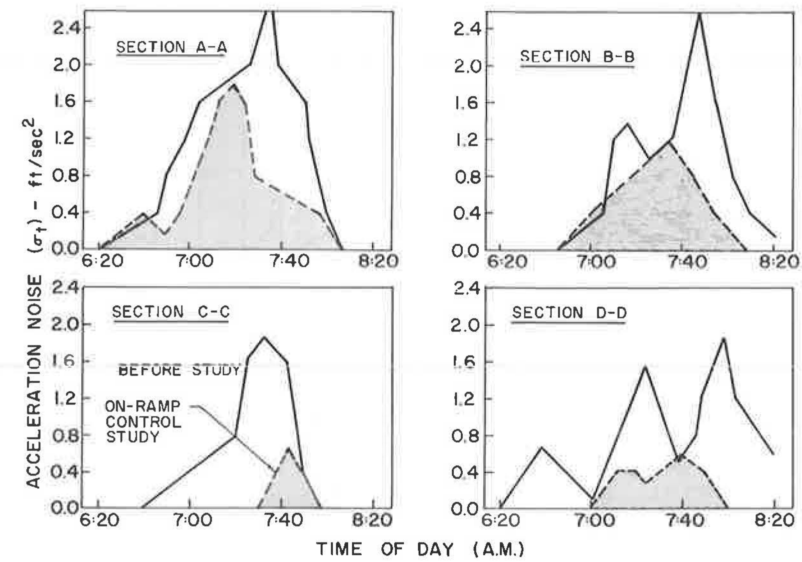


Figure 19. Acceleration noise (σ_t) profiles.

the total acceleration noise (σ_T) and the acceleration noise due to traffic interaction (σ_t), respectively, measured during the on-ramp control studies. Comparison of Figure 17 with the acceleration noise contours of the data measured before the controls (Fig. 15) indicated that the flow during controls was much smoother. To further illustrate the difference, a cross section of the contours taken at 7:30 a.m., identified as section 1-1 on Figure 17, is shown as profiles in Figure 18. The acceleration noise during controls was generally much less than without controls.

Additional investigations were made of specific locations. Sections were passed through four critical locations which were measured before controls. The resulting profiles of these sections are compared to the acceleration noise during the control study measured at the identical locations (Fig. 19). It is obvious that acceleration noise was considerably less during the control studies.

One objective of the on-ramp control study was to improve the flow upstream of the Wayside interchange. Also, minor improvement was expected downstream of Wayside. Comparison of Figures 18 and 19 confirms the fulfillment of the ramp control study objectives. The flow from Wayside to the distribution system was much smoother during the ramp controls. The high acceleration noise normally present between the two H.B. & T. Railroad tracks did not occur during the controls. Also, the smoothness of flow was noticeably better upstream of Wayside during the study. Finally, the sensitivity of the acceleration noise parameter as a means of evaluating before and after conditions is apparent.

Verification of Internal Energy Model

In the development of the internal energy model, a functional relationship between acceleration noise and speed was deduced (Eqs. 26 and 27). This relationship is expressed in Eq. 29 for the special case in which n = 1. Substituting the identities

$$\sigma_{max} = \sigma_M + \sigma_N$$

and

$$\sigma_t = \sigma - \sigma_N$$

in Eq. 29 gives

$$\sigma_t = \sigma_M - \alpha k_j u^2 + \alpha k_j u^3/u_f \tag{34}$$

TABLE 2

REGRESSION ANALYSES OF ACCELERATION NOISE

Model: $\sigma_t\ \sigma_M\ -\ C_1u^2\ +\ C_2u^3$

Section	σ_M	$C_1 \times 10^2$	Signif.	$C_2 \times 10^4$	Signif.	R^2	F Test
300-295	1.566	0.0974	**	0.0798		48.1	**
295-290	0.953	0.0119		0.0724		11.4	**
290-285	0.921	0.0317		0.0007		22.0	**
285-280	1.287	0.0047		0.0808		37.9	**
280-275	1.404	0.0927	**	0.0843	*	54.0	**
275-270	1.289	0.0657	*	0.0419		37.5	**
270-265	1.806	0.1617	**	0.1907	**	56.1	**
265-260	2.016	0.1947	**	0.2399	**	56.0	**
260-255	1.183	0.0646		0.0504		24.2	**
255-250	1.163	0.2020	**	0.2818	**	54.7	**
250-245	1.836	0.2333	**	0.3262	**	63.5	**
245-240	1.361	0.1289	**	0.1515	**	45.3	**
240-235	1.454	0.1197	**	0.1257	*	58.6	**
235-230	1.201	0.1277	**	0.1665	**	43.4	**
230-225	1.387	0.2208	**	0.3454	**	30.2	**
225-220	1.737	0.2783	**	0.4371	**	44.1	**
220-215	1.693	0.2838	**	0.4509	**	52.6	**
215-210	1.902	0.2219	**	0.3040	**	64.3	**
210-205	2.217	0.2351	**	0.3093	**	51.8	**
205-200	1.814	0.1675	**	0.2011	**	51.7	**
200-195	1.413	0.0639		0.0188		40.1	**
195-190	1.342	0.1264	**	0.1529	*	41.1	**
190-185	0.963	0.0625		0.0611		28.6	**
185-180	1.659	0.2388	**	0.3268	**	43.1	**
180-175	1.113	0.0969	*	0.1191		18.7	**
175-170	2.054	0.2788	**	0.4026	**	53.6	**
170-165	1.841	0.2256	**	0.3133	**	51.4	**
165-160	2.086	0.3181	**	0.4876	**	52.4	**
160-155	1.954	0.3080	**	0.4830	**	50.2	**
155-150	0.998	0.0984	**	0.1248	*	24.2	**
150-145	2.428	0.3599	**	0.5395	**	42.4	**
145-140	2.087	0.2453	**	0.3497	**	47.9	**
140-135	1.930	0.2783	**	0.4203	**	52.2	**
135-130	2.049	0.3133	**	0.4738	**	54.4	**
130-125	0.678	0.8079	*	0.1402	*	4.4	
125-120	0.092	-0.0230		-0.0473		3.2	
120-115	0.154	-0.0304		-0.0613		2.1	
115-110	0.213	-0.0290		-0.0709		8.4	
110-105	1.234	0.1512	*	0.2144		9.0	*
105-100	1.098	0.1034		0.1240		17.5	**
100-95	0.145	-0.0535		-0.0965		3.3	
95-90	1.221	0.1180		0.1428		16.8	
90-85	2.314	0.3159	**	0.4607	**	14.1	**
85-80	1.789	0.2179	*	0.3041	*	6.8	**
80-75	0.832	0.0768		0.0914		4.8	
75-70	1.851	0.2778	**	0.4194	**	23.0	
70-65	1.094	0.0922		0.1066		11.2	**
65-60	3.037	0.3476	**	0.4576	**	37.2	**
60-55	3.047	0.3764	**	0.5281	**	42.3	**
55-50	1.695	0.2105	**	0.2988	**	35.5	**
50-45	0.546	0.0576		0.0940		1.4	
45-40	1.333	0.1600	*	0.2183		8.0	*
40-35	2.959	0.4254	**	0.6296	**	51.7	**
35-30	1.090	0.0919		0.1073		10.0	*
30-25	2.467	0.3514	**	0.5357	**	24.5	**
25-20	1.949	0.4358	**	0.8022	**	36.0	**

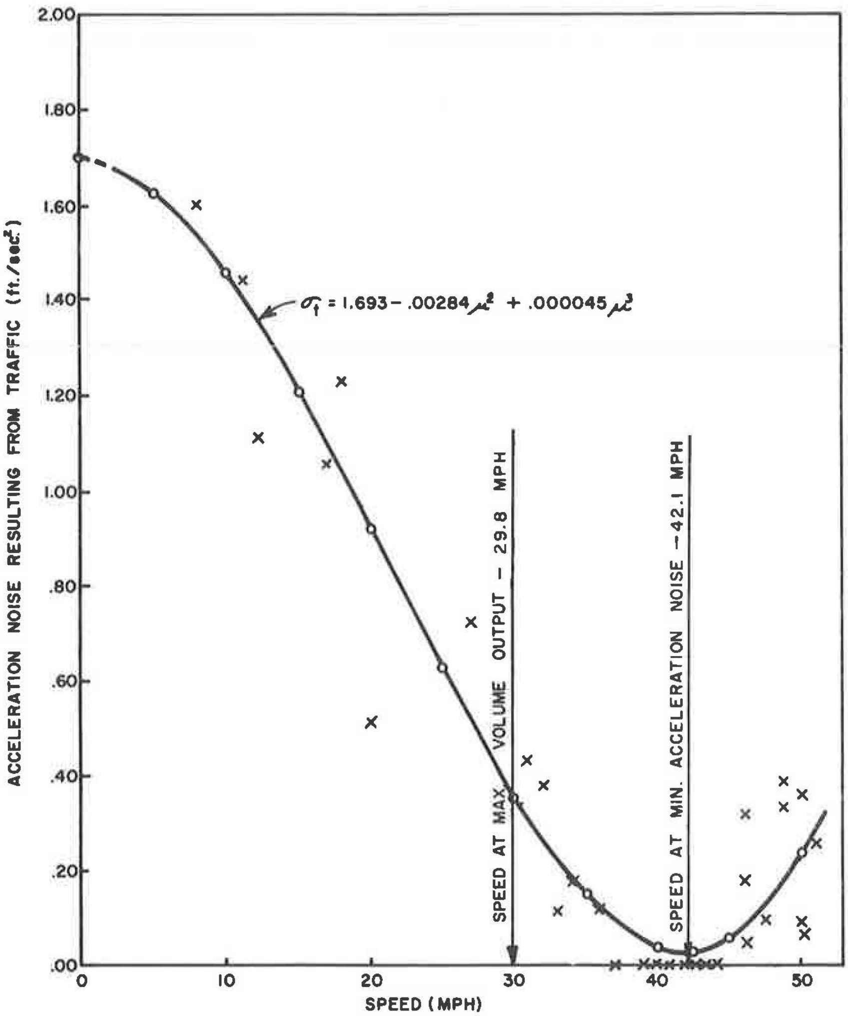


Figure 20. Relationship between speed and acceleration noise due to traffic interaction (σ_t).

Standard regression techniques were employed to test the hypothesized relationship between acceleration noise and speed expressed in Eq. 34, and thereby evaluate the internal energy model. A discussion of the statistical tests associated with the regressions is presented in the Appendix.

A review of the results tabulated in Table 2 reveals that the regressions and the partial regression coefficients are generally highly significant (0.01 level indicated by double asterisks). These and the R^2 values tend to verify the internal energy model. Two distinct continuous areas of the freeway, however, showed no significant relationship to the model. The first of these areas was between stations 300 and 285, which included the entire Reveille Interchange. A second area was located between the north and the south H.B. & T. Railroad tracks. The only explainable reason that could be formulated at this time is that the lack of data within the entire range of the regression curve resulted in an insignificant relationship. That is, the data points were generally confined to two concentrated sections of the curves which resulted in poor correlation. Further investigation of these locations would be necessary to determine the exact reason for this lack of correlation.

Figure 20 shows the final regression curve for one of the 56 freeway sections. The results show that acceleration noise increases very rapidly at the onset of congestion. Also, it is quite evident that the maximum quality of flow is not realized at maximum volume output, a fact that will be discussed in more detail in the next section.

APPLICATIONS

Freeway Operations

It seems apparent that the maximum satisfaction that could be experienced by a motorist would result from a uniform uninterrupted freeway journey. This condition can be achieved by minimizing acceleration noise in the stream. If a relationship exists between acceleration noise and any of the above quantitative traffic parameters, the optimum operating conditions based on smoothness of flow can be determined.

When the volumes on a freeway are extremely light, an individual motorist can select a desirable speed because of the freedom of movement which he experiences. The acceleration noise of his trip is due primarily to his driving abilities and the geometrics of the freeway. As volumes increase, a level is reached at which an individual motorist finds it difficult to adjust laterally. Thus his speed in effect is influenced by the vehicle in front of him, which in turn is influenced by the vehicle in front of it, and so on down-

TABLE 3

OPTIMUM SPEED (u') AND VOLUME (q_m') AT MINIMUM ACCELERATION NOISE

Section	u'_m(mph)	q'_m(vph)	Section	u'_m(mph)	q'_m(vph)
300-295			160-155	43	4710
295-290			155-150		
290-285			150-145	44	4440
285-280			145-140	47	3980
280-275			140-135	44	4530
275-270			135-130	44	4780
270-265			130-125		
265-260			125-120		
260-255			120-115		
255-250	48	4260	115-110		
250-245	48	3350	110-105		
245-240	56	3240	105-100		
240-235			100-95		
235-230	51	4360	95-90		
230-225	43	5400	90-85	46	4500
225-220	42	4880	85-80	48	
220-215	42	4300	80-75		
215-210	48	3510	75-70	44	4840
210-205	50	4390	70-65		
205-200	55	3200	65-60	51	2690
200-195			60-55	47	3790
195-190			55-50	47	3610
190-185			50-45		
185-180	48	4380	45-40		
180-175			40-35	45	4330
175-170	46	4550	35-30		
170-165	48	4030	30-25	44	3350
165-160	43	4740	25-20	36	4760

stream to the lead car in the platoon. The flow at this point approaches a uniform speed and uniform headways. Consequently, the operation is smooth and the acceleration noise is a minimum (the value of which would be dictated by the freeway geometrics). A further increase in demand is accompanied by an increase in internal friction which promotes some instability in the stream and increases acceleration noise. It is during this unstable flow condition that driver discomfort begins to be realized. Additional demand increases result in inevitable freeway breakdown and a rapid decrease in driver convenience and comfort.

Equation 28 shows that the optimum speed (u'_m) at minimum acceleration noise can be determined by calculating the points on the curve of zero slope. This accomplished by setting the first derivative of the regression equation to zero and solving for u. The equation for the curve presented in Figure 20 was determined to be

$$\sigma_t = 1.693 - 0.00428u^2 + 0.000045u^3$$

The optimum speed is thus

$$u'_m = 42.1 \text{ mph}$$

The results of similar calculations of optimum quality speed are given in Table 3. A study of Table 3 reveals that the quality of flow based on minimizing acceleration noise is a maximum at speeds ranging generally between 40 and 50 mph. A previous study (1) on the Gulf Freeway has shown that the speeds at maximum volume output range between 25 and 35 mph.

The relationship between location and optimum speed at minimum acceleration noise can best be illustrated in a profile map (Fig. 21). The distinction between optimum speed u'_m and critical speed u_m is also made in this presentation. Again, only locations where the regression equations were highly significant are presented. Examination of the speed profiles shows that the speeds at minimum acceleration noise are much higher than the speeds for maximizing volume output.

Increasing attention has been given to the possibility of and need for using variable speed control on urban freeway sections as a means of easing the accordion effects in a traffic stream as congestion develops. The speed limit signs may display normal speed limit signs or multivalue reduced speeds and would be controlled by command signals from a traffic regulating center. The optimum speed u'_m offers an ideal parameter as a basis for this type of control. Two neon matrices would be utilized with the left-hand or tens matrix capable of displaying digits 2, 3, 4, or 5. The right hand or units matrix would display digits 0 and 5. The signs would be mounted over each lane. The advisory speeds would be determined at a given location from the speed profiles (Fig. 21). The idea would be to induce drivers to travel voluntarily at a speed that would minimize acceleration noise and perhaps postpone or eliminate breakdowns due to congestion.

Because the control of vehicles entering the freeway, as against the control of vehicles already on the freeway, offers a more positive means of preventing congestion, considerable emphasis has been placed on the technique of ramp metering. The utilization of the acceleration noise parameter as a means of evaluating freeway control and before-and-after studies has been demonstrated as reported in the previous section. It is anticipated that this parameter will play an important role in the future in determining which of the many procedures for ramp and freeway control are the most effective.

Capacity and Level of Service

Greater dependency on motor vehicle transportation has brought about a need for greater efficiency in traffic facilities. The ability to accommodate vehicular traffic is a primary consideration in the planning, design, and operation of streets and highways. It is, however, not the only consideration. The individual motorist, for example, seldom interprets the efficiency of a facility in terms of the volume accommodated. He evaluates efficiency in terms of his trip—the service to him.

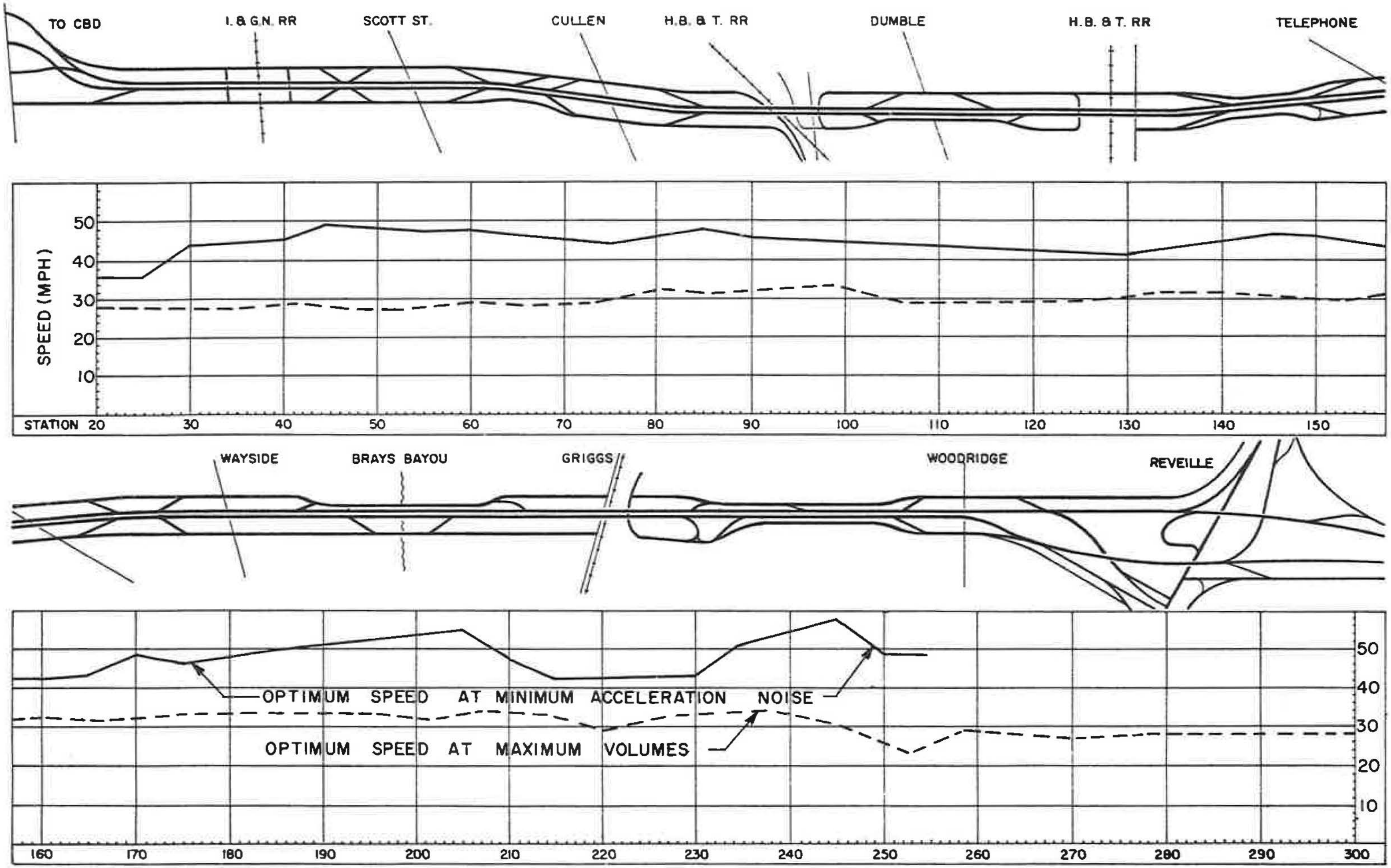


Figure 21. Optimum speed.

The original edition of the Highway Capacity Manual defined three levels of roadway capacity—basic capacity, possible capacity, and practical capacity. It was considered of prime importance that traffic volumes be accurately related to local operating conditions so that particular agencies could decide on the "practical" capacities for facilities within their jurisdiction. The manual recognized that practical capacity would depend on the basis of a subjective evaluation of the quality of service provided.

The present Capacity Committee of the Highway Research Board has elected, in the new edition, to define a single parameter—possible capacity—for each facility. Possible capacity is simply the maximum number of vehicles that can be handled by a particular roadway component under prevailing conditions. The practical capacity concept has been replaced by several specific "service volumes" which are related to a group of desirable operating conditions referred to as level of service.

Ideally, all the pertinent factors—speed, travel time, traffic interruptions, freedom to maneuver, safety, comfort, convenience and economy should be incorporated in a level of service evaluation. The Committee has, however, selected speed and the service volume-to-capacity (v/c) ratio as the factors to be used in identifying level of service because "there are insufficient data to determine either the values or relative weight of the other factors listed."

Six levels of service, designated A through F from best to worst, are recommended for application in describing the conditions existing under the various speed and volume conditions that may occur on any facility. Level of Service A describes a condition of free flow; Level of Service E describes an unstable condition at or near capacity; Level of Service F gives a condition of forced flow. Levels of Service B, C, and D describe the zone of the stable flow with the upper limit set by the zone of free flow and the lower limit defined by Level of Service F. Although definitive values are assigned to these zone limits for each type of highway in the new manual, no explanation is given as to how these values were obtained. This is in no way intended as a criticism since it is recognized that the function of any manual is essentially that of a handbook and therefore should not include a methodical discussion of the facts and principles involved and conclusions reached for every value between its covers.

TABLE 4

LEVELS OF SERVICE AS ESTABLISHED BY ENERGY-MOMENTUM CONCEPT

Level of Service Zone		Zone Limits (See Fig. 22) Upper	Lower	Description
Free Flow	A	u_f	$0.91u_f$, $0.35q_m$	Speeds controlled by driver desires and physical roadway conditions. This is the type of service expected in rural locations.
Stable Flow	B	$0.91u_f$, $0.35q_m$	$0.83u_f$, $0.55q_m$	Speed primarily a function of traffic density.
	C	$0.83u_f$, $0.55q_m$	$0.75u_f$, $0.75q_m$	The conditions in this zone are acceptable for freeways in suburban locations.
	D	$0.75u_f$, $0.75q_m$	u'_m, q'_m	The conditions in this zone are acceptable for urban design practice. The lower limit u'_m, q'_m represents the critical level of service.
Unstable Flow	E_1	u'_m, q'_m	u_m, q_m	A small increase in demand (flow) is accompanied by a large decrease in speed leading to high densities and internal friction.
	E_2	u_m, q_m	$0.33u_f$, q'_m	This type of high density operation cannot persist and leads inevitably to congestion.
Forced Flow	F	$0.33u_f$, q'_m	0	Flows are below capacity and storage areas consisting of queues of vehicles form. Normal operation is not achieved until the storage queue is dissipated.

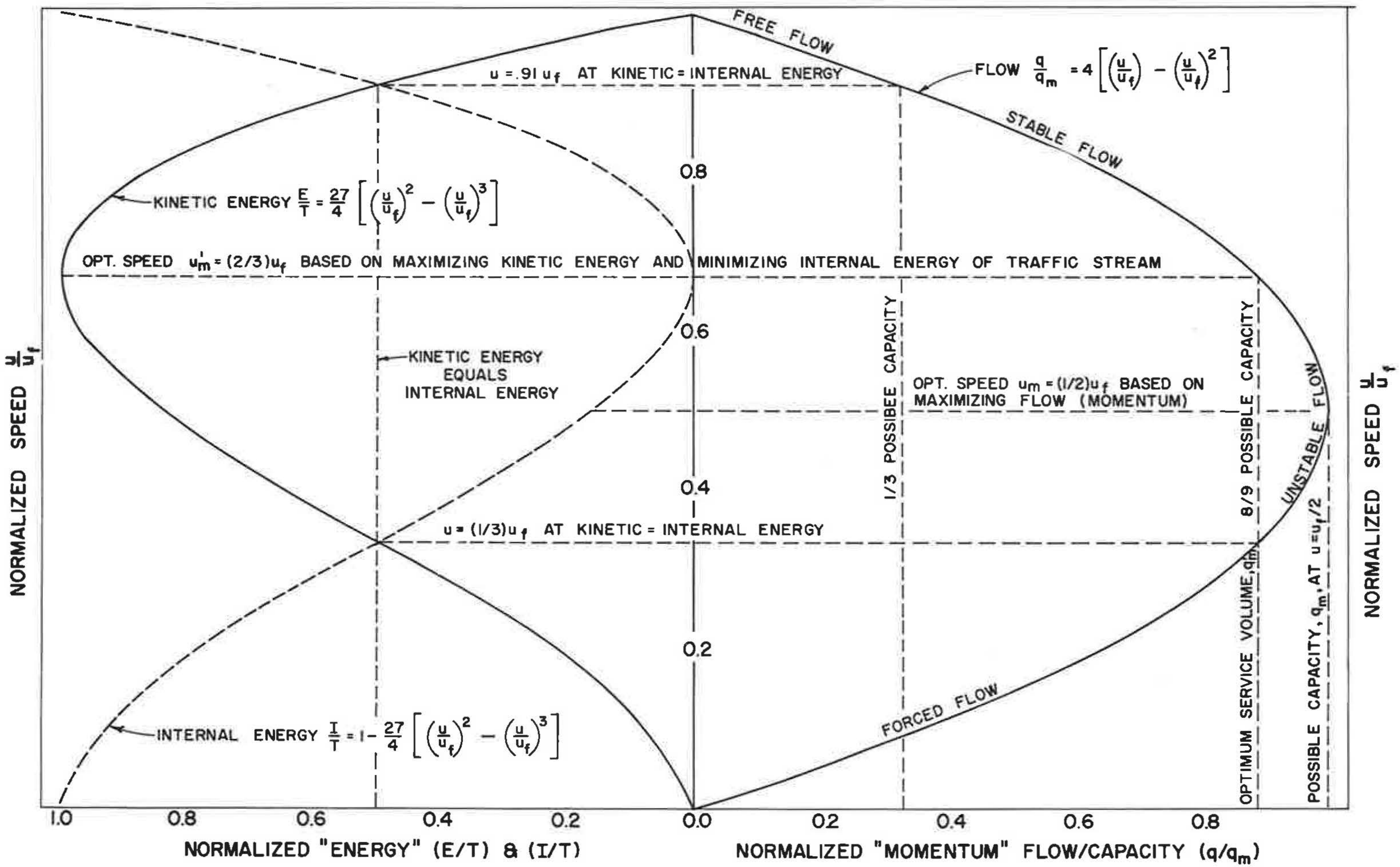


Figure 22. Quantitative approach to level of service using the "total energy"—momentum analogy.

The authors feel that much of the traffic engineer's dilemma can be attributed to his inability to relate capacity and level of service. If these volumes cannot be related quantitatively for an existing facility, there can be little hope for the designer to relate them for a facility that is still on the drafting board! The acceleration noise-internal energy model developed in this report provides a simple, rational means for measuring freeway capacity and level of service.

If Eqs. 10 and 15 are expressed in terms of speed only, and then normalized, they become (for the special case of n = 1)

$$\frac{q}{q_m} = 4\left[\left(\frac{u}{u_f}\right) - \left(\frac{u}{u_f}\right)^2\right] \tag{35}$$

and

$$\frac{E}{T} = \frac{27}{4}\left[\left(\frac{u}{u_f}\right)^2 - \left(\frac{u}{u_f}\right)^3\right] \tag{36}$$

The curves of Eqs. 35 and 36 are plotted in Figure 22. The right side of the graph is the well known volume-speed relationship normalized so that the abscissa is the ratio of flow to capacity (v/c ratio) and the ordinate the ratio of speed to free speed. Substituting Eq. 20 in Eq. 36 gives

$$\frac{I}{T} = 1 - \frac{27}{4}\left[\left(\frac{u}{u_f}\right)^2 - \left(\frac{u}{u_f}\right)^3\right] \tag{37}$$

which is also plotted in Figure 22. Equating Eqs. 36 and 37 gives the two speed parameters for which the kinetic energy equals the internal energy of the traffic stream. These values are $u = 1/3\ u_f$ and $u = 0.91\ u_f$. On the right side of the graph the corresponding values for flow are $q = 8/9\ q_m$ and $q = 0.328\ q_m$ rounded off to $q = 0.35 q_m$. These two points plus the point defined as u'_m, q'_m serve to establish the four levels of service zones defined by the 1965 Highway Capacity Manual—free, stable, unstable and forced flow. In Table 4, the foregoing fundamental level of service criteria are summarized.

The significance of Figure 22 is that it provides a rational basis for defining level of service and relating it to the other traffic variables—speed, flow, and density. The relationships between level of service and traffic volume (flow) are analogous to the relationships in classical hydrodynamics between energy and momentum. Efficiency in a classical system is measured by the ratio of useful energy to total energy, E/T. Optimum operation occurs when lost energy I is at a minimum. In a traffic system, this concept of efficiency is manifest by maximizing the kinetic energy of the stream as a whole and minimizing the acceleration noise of the individual vehicles (internal energy).

The objective of freeways and other expressways is to provide high levels of service for high volumes of traffic. The traffic conditions existing at maximum E/T and minimum I/T, therefore, might logically be termed "critical level of service." Referring to the right side of Figure 22, it is seen that a small increase in demand above the volume existing at this critical level of service tends to greatly increase the density of the traffic stream accompanied inevitably by a sharp decrease in operating speed. That the traffic conditions k'_m and u'_m at the critical level of service are superior to those at possible capacity can be shown explicitly by dividing Eqs. 16 and 17 by 11 and 12 for n = 1:

$$k'_m = (2/3)\ k_m \tag{38}$$

$$u'_m = (4/3)\ u_m \tag{39}$$

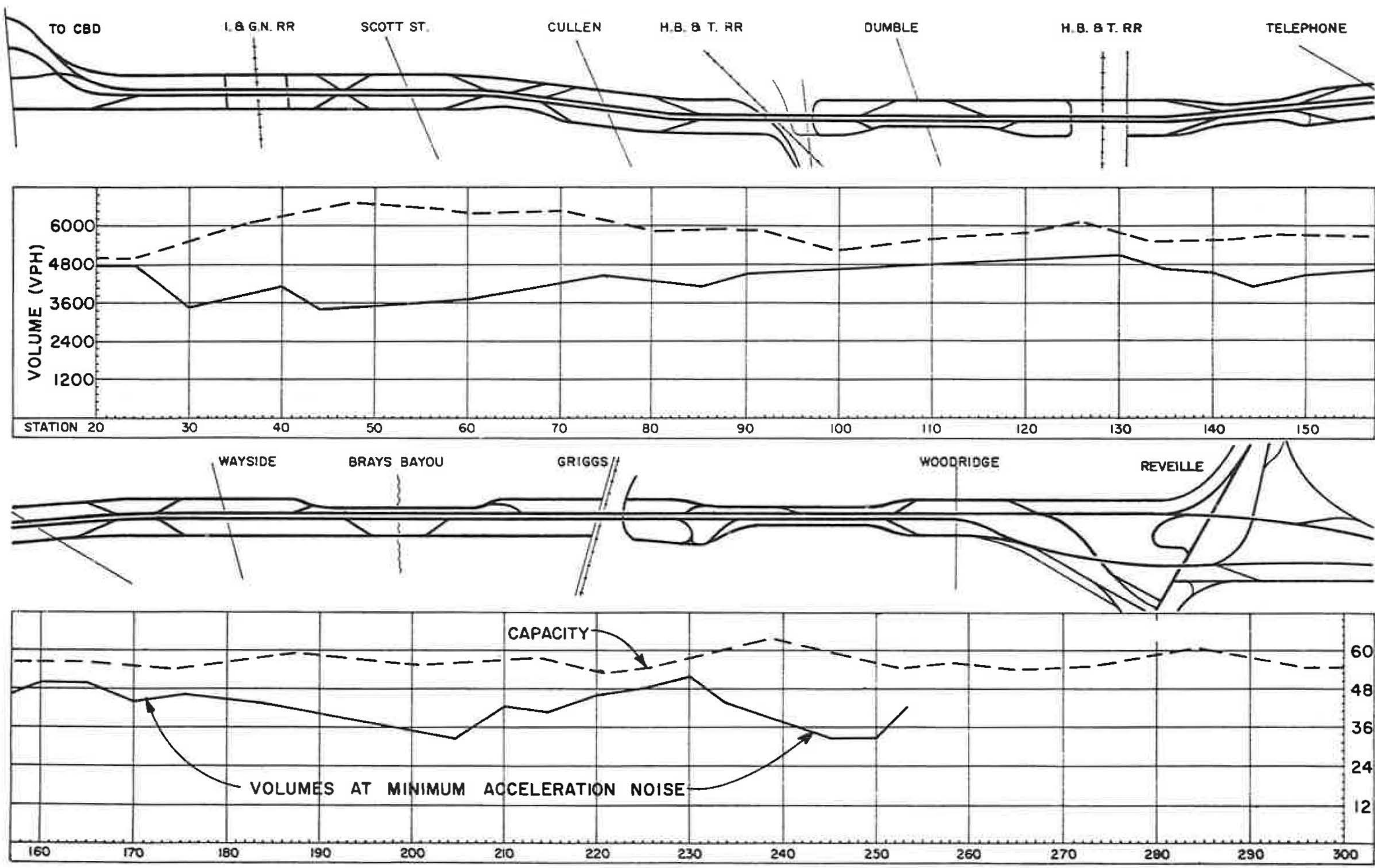


Figure 23. Optimum volumes.

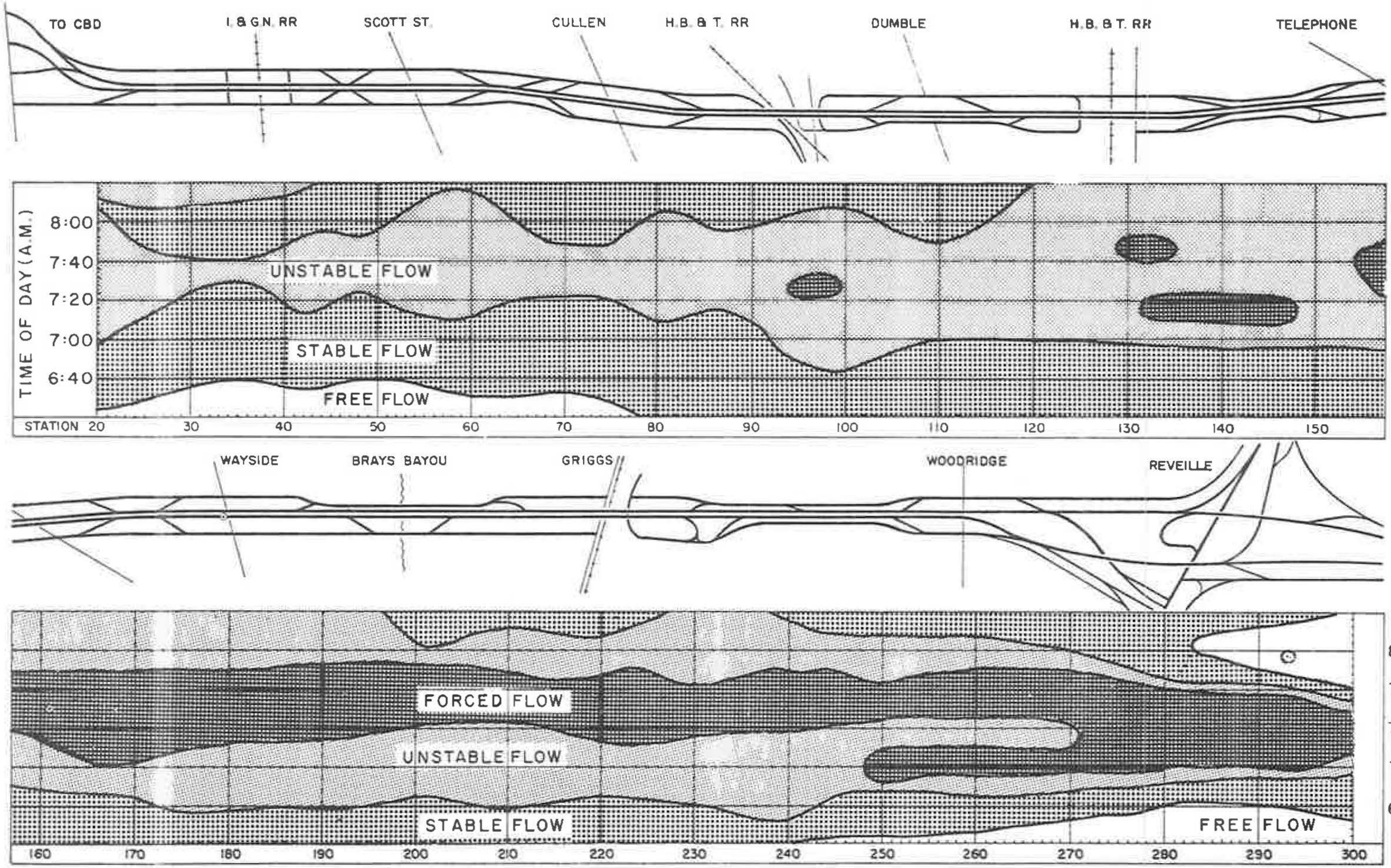


Figure 24. Level of service contours, Tuesday, inbound.

The density at the critical level of service is only 2/3 the density at possible capacity and the operating speed is 1/3 higher. Of course, this is accomplished at a sacrifice in the traffic volume accommodated since

$$q'_m = (8/9)\ q_m \tag{40}$$

The acceleration noise measurements taken on the Gulf Freeway (Fig. 21) tend to substantiate the relationship stated in Eq. 39 regarding the speed u'_m at critical level of service as compared to the speed u_m at critical flow (capacity). Figure 23 illustrates a similar comparison between the critical service volume q'_m and the capacity q_m. As a basis for ramp metering, for example, it places operation of the facility in the unstable zone of operation, and provides absolutely no safety factor against breakdowns due to statistical variability in demand.

Figure 24 illustrates the operation of the three inbound lanes of some 6 mi of the Gulf Freeway during the morning peak 2 hr as obtained using the moving vehicle study procedure described in this report. The figure represents a complete documentation of level of service in both time and space. This simple procedure affords a rational means for describing the level of operation on the facility—free flow, stable flow, unstable flow and forced flow—as established by the energy-momentum concepts illustrated in Figure 22.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions may be drawn:

1. The standard deviation of the acceleration of a vehicle is called acceleration noise, σ. This parameter can be calculated from a speed-time graph of a vehicle's trip. If the acceleration noise for the vehicle is obtained in the absence of traffic, this factor can be ascribed to the geometrics of the facility, and is therefore called the natural noise, σ_N. If the acceleration noise for the vehicle is obtained during periods of normal freeway operation, the amount of acceleration noise in excess of the natural noise of the facility is due to the existing traffic interaction, σ_t.

2. Kinetic energy, $\alpha k u^2$, is the energy of motion of the traffic stream. Internal energy is the lost energy of the traffic stream, and it is given by the acceleration noise, σ. The sum of kinetic energy E and internal energy I is the total energy T; the units of traffic "energy" are those of acceleration. Energy as expressed in terms of E and I is consistent with the level of service concept described in the 1965 Highway Capacity Manual. Thus, the kinetic energy of the stream fulfills the first level of service factor (speed and travel time) whereas internal energy of acceleration noise takes into account such level of service factors as traffic interruption, freedom to maneuver, safety, comfort, and operation costs.

3. If any two of the traffic variables—k, u, q, or σ—can be measured, then by using the energy model the graphs of Figure 22 can be drawn for short homogeneous freeway sections and the following traffic parameters calculated: k_m, u_m, q_m, k'_m, u'_m, q'_m, and σ_N. Thus, it was possible for a driver in a test vehicle equipped with a recording speedometer measuring u and σ to obtain these 7 parameters for a 6-mi section of the Gulf Freeway, plus a complete description of the level of service for the facility over the 2-hr morning peak, by simply driving over the facility.

4. Acceleration noise can be a useful tool for measuring the changes in smoothness of flow resulting from on-ramp control and metering procedures. Since there is a good indication that acceleration noise is linearly related to the absolute difference in adjoining grades, it is apparent that this parameter might be useful in measuring the effects of geometric changes. Thus, acceleration noise has practical application in both operations and design.

Recommendations for future research include the following:

1. Further research is needed to determine the reason for lack of correlation to the tested models of acceleration noise within certain locations of the inbound Gulf Freeway.

2. More extensive study should be made to substantiate the relationship between freeway grades and acceleration noise.

3. The effects of other geometric considerations such as entrance and exit ramps as related to acceleration noise should be studied to determine the efficiency of various configurations.

4. Special studies should be made at locations with high acceleration noise measurements, especially in relation to accident experience.

5. Research is necessary to establish the level of service on parallel arterials to the Gulf Freeway during the peak period, using the energy model employed for establishing the level of service on the freeway.

6. The level of service on parallel arterials should be measured in conjunction with control or ramp metering experiments in order to develop data which can be compared to normal periods of operation on the Gulf Freeway. This would provide a cost-benefit approach to measuring the improvements provided by automatic surveillance and control for use by highway engineers and administrators.

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Appendix

MATHEMATICAL APPROACH

The following is a development of an equation used by Jones and Potts (13) for approximating acceleration noise. If v(t) and a(t) are the speed and acceleration of a car at time t, then the average acceleration of the car for a trip of time T is

$$\alpha_{avg} = \frac{1}{T}\int_0^T a(t_i)\,dt = \frac{1}{T}\,[v(T) - v(0)]$$

The standard deviation of a set of n numbers $x_1, x_2, \ldots, x_n$ is denoted by s and is defined by

$$s = \left[\frac{1}{n}\sum_{i=1}^{n}(x_1 - \bar{x})^2\right]^{1/2}$$

where $\bar{x}$ is the mean of the x's.

From Figure 25 it is seen that if $a(t_i)$ denotes the acceleration of a vehicle at time t_i, the square of the difference between any acceleration and the average acceleration is denoted by

$$\left[a(t_i) - \alpha_{avg}\right]^2$$

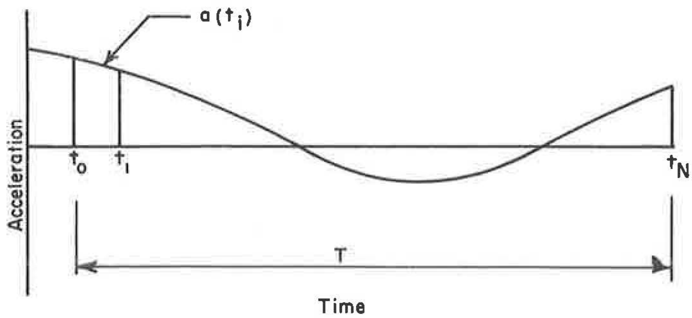

Figure 25. Acceleration vs time.

The summation of these differences over a time T becomes

$$\int_0^T \left[a(t_i) - \alpha_{avg}\right]^2 dt$$

The number of t's in the sample is equal to

$$\sum_{i=0}^{T} t_i = T$$

Therefore the acceleration noise (σ) can be written

$$\sigma = \left[\frac{1}{T}\int_0^T \left[a(t_i) - \alpha_{avg}\right]^2 dt\right]^{1/2} \tag{41}$$

and

$$\sigma^2 = \frac{1}{T}\int_0^T \left[a(t_i) - \alpha_{avg}\right]^2 dt \tag{42}$$

Expanding Eq. 42

$$\sigma^2 = \frac{1}{T}\int_0^T \left[a(t_i)\right]^2 dt - \frac{1}{T}\int_0^T 2a(t_i)\,\alpha_{avg}\,dt + \frac{1}{T}\int_0^T \left[\alpha_{avg}\right]^2 dt$$

$$\sigma^2 = \frac{1}{T}\int_0^T \left[a(t_i)\right]^2 dt - 2\alpha_{avg}\frac{1}{T}\int_0^T a(t_i)\,dt - \left[\alpha_{avg}\right]^2$$

$$\sigma^2 = \frac{1}{T}\int_0^T \left[a(t_i)\right]^2 dt - 2\left[\alpha_{avg}\right]^2 + \left[\alpha_{avg}\right]^2$$

$$\sigma^2 = \frac{1}{T}\int_0^T \left[a(t_i)\right]^2 dt - \left[\alpha_{avg}\right]^2 \tag{43}$$

The value of σ^2 can be estimated by approximating Eq. 43 with

$$\sigma^2 \approx \frac{1}{T} \sum_{i=0}^{T} \left[\frac{\Delta v}{\Delta t}\right]^2 \Delta t - \left[\frac{(V_T - V_0)}{T}\right]^2$$

where V_0 and V_T are the initial and final velocities, respectively. If Δv is taken as a constant (say 2 mph) throughout the measurement, then

$$\sigma^2 = \frac{(\Delta v)^2}{T} \sum_{i=0}^{T} \frac{n^2}{\Delta t_i} - \left[\frac{(V_T - V_0)}{T}\right]^2 \tag{44}$$

where n is the number of speed changes of 2 mph in time Δt.

If the final velocity is the same as the initial velocity, the average acceleration is zero and the second term of Eq. 44 drops out. When the acceleration noise is measured on a long stretch of highway where T is very large, the second term is very small and can be ignored (13). Therefore for relatively long trips

$$\sigma^2 = \frac{(\Delta t)^2}{T} \sum_{i=0}^{T} \frac{n^2}{\Delta t_i}$$

If Δt is measured for each successive speed change of 2 mph, i.e., n is equal to 1, the approximation equation of acceleration noise as developed by Jones and Potts (13) becomes

$$\sigma = \left[\frac{(\Delta v)^2}{T} \sum_{i=0}^{T} \frac{1}{\Delta t_i}\right]^{1/2} \tag{45}$$

To calculate σ in units of feet/second/second (ft/sec^2), Eq. 5 becomes

$$\sigma = \left[\frac{(1.465)^2\ (\Delta v)^2}{T} \sum_{i=0}^{T} \frac{1}{\Delta t_i}\right]^{1/2} \tag{46}$$

where Δv is in units of miles/hour, T is in units of seconds, and Δt is in units of seconds.

The deviations of the accelerations when a vehicle is stopped in traffic is zero. Since this type of situation would reduce the value of σ even though it adds to the annoyance and frustration of the motorist, σ is measured only while the vehicle is in motion and therefore T is taken as the running time of the vehicle.

If a continuous analogue record is made of a vehicle's speed vs time while traversing a highway, the acceleration noise of a vehicle can be determined using Eq. 46. For every change in speed (Δv) of say, 2 mph, Δt is measured from the chart and the value $1/\Delta t$ can be accumulated up to time T.

COMPARISON OF SPEED INCREMENTS

A problem arises regarding the selection of Δv for Eq. 44. It must be realized that the equation is merely an approximating equation and therefore, the smaller

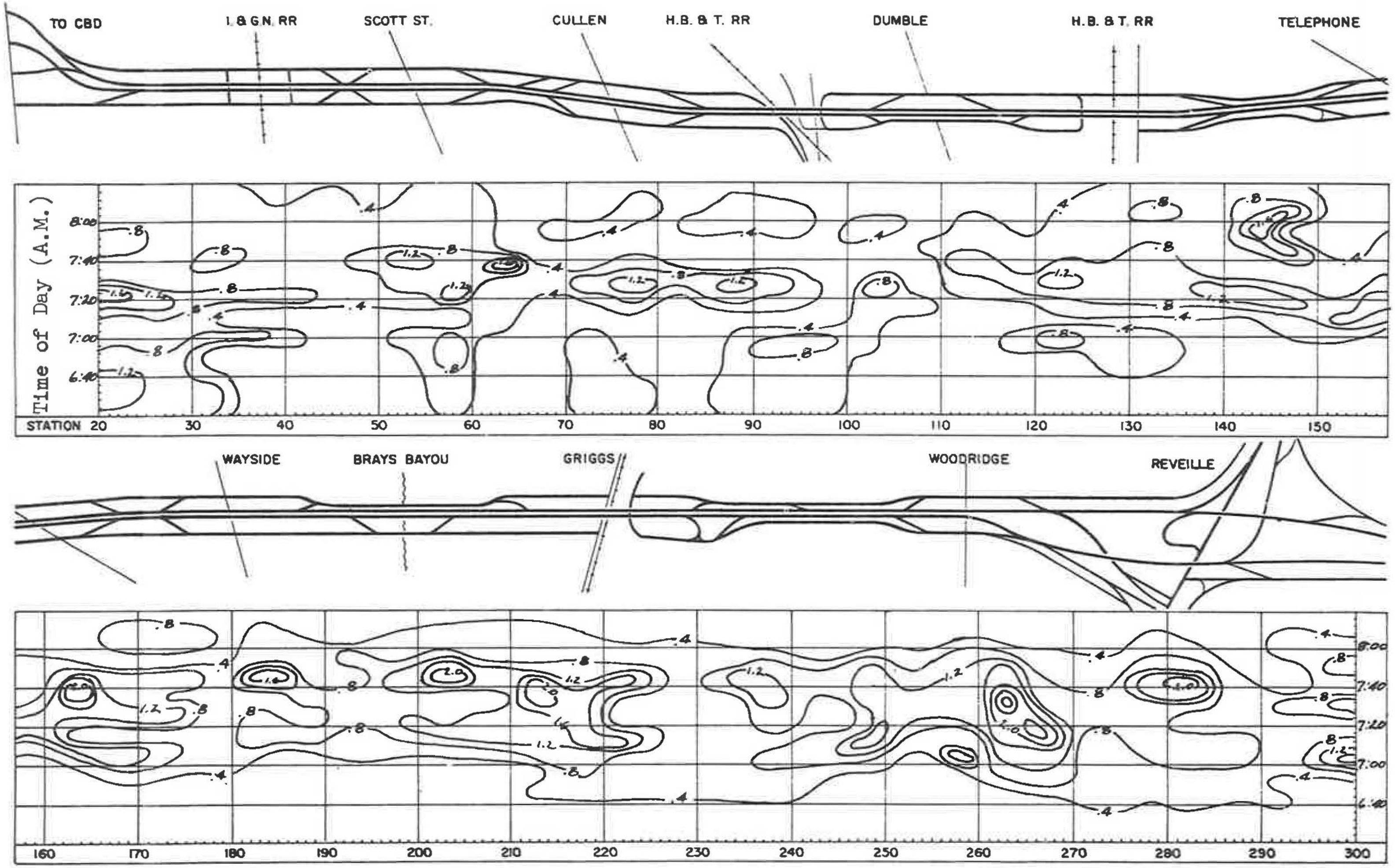

Figure 26. Total acceleration noise (σ_T) contours, $\Delta V = 2$ mph.

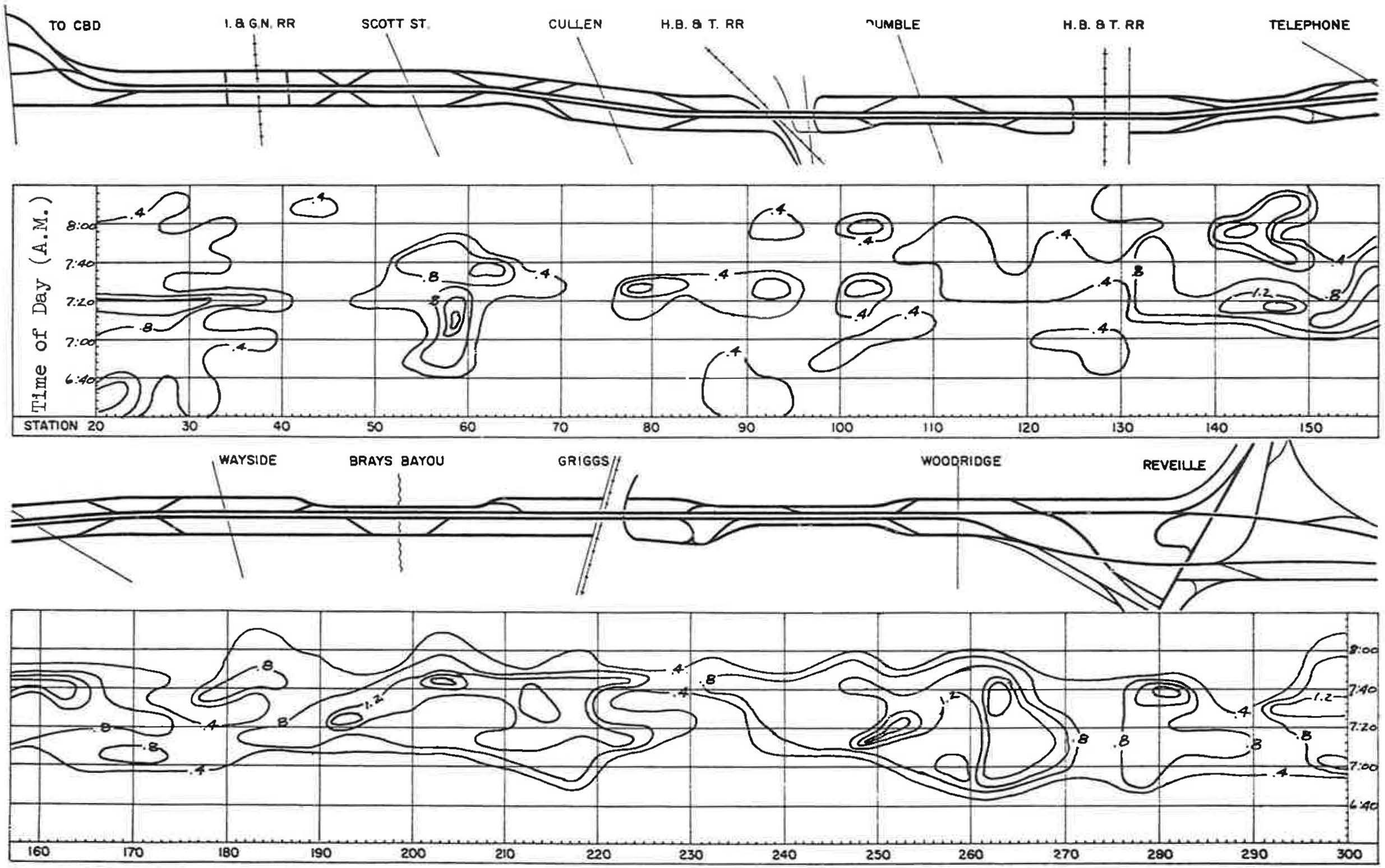

Figure 27. Total acceleration noise (σ_T) contours, $\Delta V = 4$ mph.

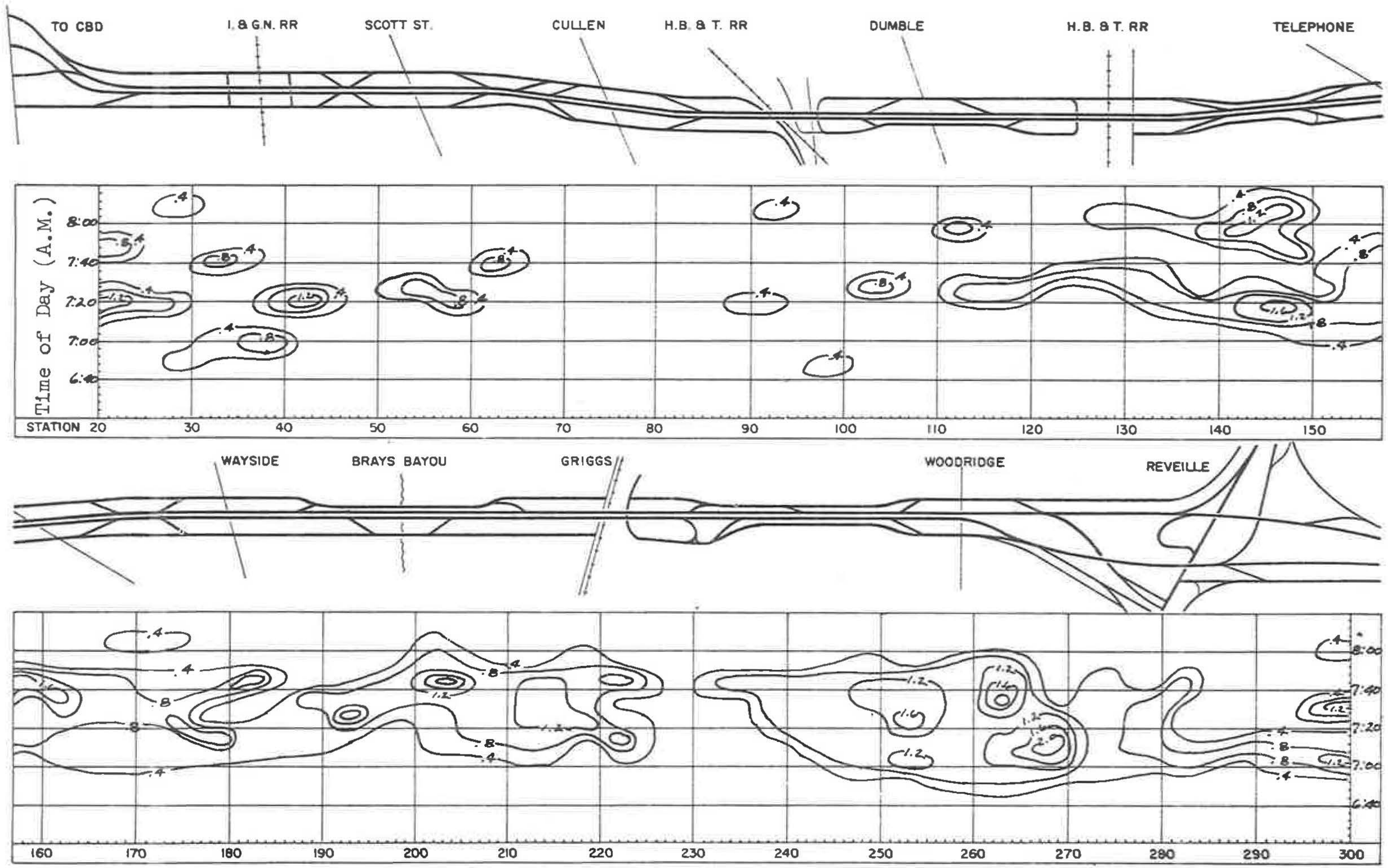

Figure 28. Total acceleration noise (σ_T) contours, ΔV = 6 mph.

the value of Δv selected, the closer the values of σ will approach the true condition.

The determinants which will affect the choice of this constant are obviously the limitations of the recording equipment and the accuracy needed for the study. Several types of speed recording equipment are available with varied degrees of speed ranges and with varied degrees of accuracy associated with reading the graphs. For example, one type of recorder is limited to 45 mph with increments of 5 mph on the chart recording. Other equipment can be adjusted to record speeds to any desired reasonable maximum with speed increments as low as 1 mph.

The purpose of the study might also influence the decision on the accuracy of the σ estimate. One may be concerned about reaching preliminary estimates for a quick comparison during certain freeway control operations. Using a small value for Δv may be too time consuming at the time for the purpose in mind. A larger value would result in larger error but would present a quick impression of where additional control measures could be taken.

It must be emphasized that the selection of Δv to be used in Eq. 44 will influence the accuracy of the final results. Therefore, care must be exercised when comparing results of other studies. Also, the values of σ determined from studies should be related to the constant chosen.

Contour maps of acceleration noise are illustrated in Figures 26, 27, and 28 based on varied constants of $\Delta v = 2$, 4, and 6, respectively. It is noticed that as Δv increases, the contours are less dense because of the decreased sensitivity of σ. Figure 28 with $\Delta v = 6$ for example, illustrates the sections of freeway where the speed changed by as much as 6 mph. Speed deviations less than this amount resulted in zero acceleration noise. Figure 26 with $\Delta v = 2$, however, indicated areas with speed changes of 2 mph or greater.

COMPARISON OF ACCELERATION NOISE IN SHORT FREEWAY SECTIONS

Assume the speed data recorded are as shown in Figure 29. Two distinct conditions are presented. In the first section the speed drops 8 mph and then returns to the original speed. The second section illustrates a 16-mph reduction in speed. It is noted that the average acceleration in the first section is zero; however, the average acceleration in the second section is 0.84 ft/sec^2 and is therefore very significant and cannot be ignored.

It is obvious that if only an acceleration or deceleration takes place within the short section, the mean acceleration takes on some value other than zero. The distribution of the accelerations is about this mean and the acceleration noise (by definition) may be very small. However, if there is a reversal of speeds within the section so that the speed at the end of the section approaches the speed at the beginning of the section, the average acceleration approaches zero and the second term of Eq. 44 vanishes.

The acceleration noise calculated in both sections as shown in Figure 29 reveals that σ is much larger in the first section even though the speed reduction in the second section was twice as much. The reason for this contradiction can be explained by the fact that, although the variance of the accelerations in the second section was lower, the distribution of the accelerations is about a mean value of 0.84 ft/sec^2, whereas the distribution of the accelerations of the first section is about a mean of zero. One cannot conclude, however, that the flow in the second section is better because of a lower σ value.

In order to evaluate the disturbance of flow in both sections the acceleration distribution must have a relative base for comparison. An appropriate base would be a common mean acceleration. Since it is intended to use acceleration noise to measure the smoothness of flow it is desirable to evaluate the sections about an ordinate of zero acceleration.

The transformation of the distribution from any mean, c, to the ordinate is accomplished by the following technique. The standard deviation in statistics is similar to the radius of gyration of an area under a frequency distribution curve about the ordinate through the centroid of that area and is equal to the standard deviation of the distribution.

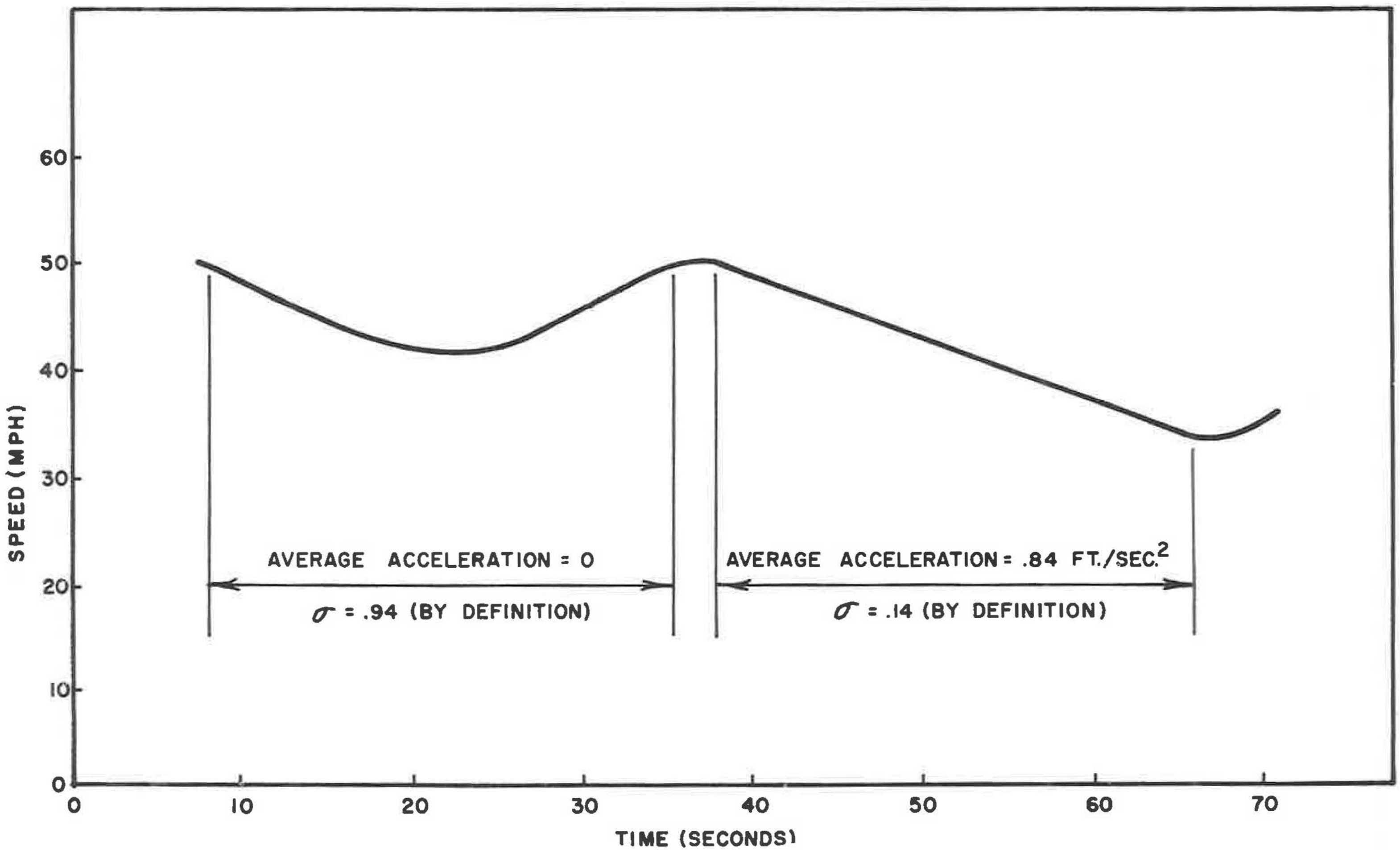

Figure 29. Acceleration noise in short freeway sections determined by definition.

The meaning of the radius of gyration of an area can be interpreted as a length which, when squared and multiplied by the area assumed to be concentrated at a point, would have the same rotational effect as the actual area with respect to the given axis:

$$I_a = Ak_a^2$$

where

I_a = moment of inertia of the area,
A = area, and
k_a = radius of gyration.

The standard deviation of a frequency distribution considered as a set of n equal particles of area is the square root of the arithmetic mean of the squares of radial distances of these particles from the centroidal axis. In other words, the standard deviation is the radius of gyration k with respect to the centroidal axis:

$$\sigma = k$$

By use of the parallel axis theorem it is evident that the moment of inertia of an area about any axis b other than the centroidal axis is equal to the moment of inertia about the centroidal axis c plus the area of rotation times the square of the distance of b from c.

$$I_b = I_c + Ad^2$$

$$Ak_b^2 = Ak_c^2 + Ad^2$$

$$k_b^2 = k_c^2 + d^2$$

Substituting σ for k,

$$\sigma_b^2 = \sigma_c^2 + d^2$$

When b is equal to zero the square of the acceleration noise about zero becomes

$$\sigma_0^2 = \sigma_c^2 + d^2 \qquad (47)$$

where d is the distance from the origin to the mean of the simple distribution, that is, the distance to the average acceleration. Substituting Eq. 44 into Eq. 47 and noting that d is equal to the average acceleration, Eq. 47 can be written as

$$\sigma_0^2 = \left[\frac{(\Delta v)^2}{T} \sum_{i=0}^{T} \frac{n^2}{\Delta t_i}\right] - \left[\frac{(V_T - V_0)}{T}\right]^2 + \left[\frac{(V_T - V_0)}{T}\right]^2$$

which reduces to

$$\sigma_0^2 = \frac{(\Delta v)^2}{T} \sum_{i=0}^{T} \frac{n^2}{\Delta t_i}$$

Assuming n is equal to 1, the relative acceleration noise for short sections of roadway becomes

$$\sigma_0 = \left[\frac{(\Delta v)^2}{T} \sum_{i=0}^{T} \frac{1}{\Delta t_i}\right]^{1/2} \qquad (48)$$

RELATIONSHIP BETWEEN GRADES AND ACCELERATION NOISE

The data used for this analysis are listed in Table 5. Confidence limits of the average acceleration noise are given in Table 6. The principle of least squares was again applied to resolve the best fit to the available data. The hypothesized equation to be tested was

$$Y = B_0 + B_1X_1 + B_2X_2$$

where

Y = acceleration noise (σ_N),
X_1 = absolute difference in adjoining grades, and
X_2 = length of grade.

The estimating equation for regression becomes

$$y = b_0 + b_1X_1 + b_2X_2$$

The standard regression program from the Data Processing Center, Texas A&M University, was utilized to determine the partial regression coefficients and to make the proper tests of significance.

The result of the regression analysis indicated that the independent variable length of grade (X_2) was not significant at the 0.05 level. The absolute value of the grades, however, was highly significant. These results prompted a second analysis of only

TABLE 5

TABLE OF GRADES WITHIN THE STUDY AREA[a]

Location	Percent Plus Grade	Percent Minus Grade	Absolute Difference	Length, ft	Acceleration Noise (σ_N), ft/sec^2
Woodridge Rd.	3.0	3.0	6.0	1200	0.18
Griggs Rd.	5.0	5.0	10.0	1900	0.44
Wayside Dr.	1.5	1.5	3.0	1200	0.11
Telephone Rd.	2.5	2.5	5.0	1200	0.29
South H. B. &T. RR	5.0	5.0	10.0	1550	0.42
North H. B. &T. RR	5.0	5.0	10.0	1500	0.49
Cullen Rd.	5.0	5.0	10.0	1200	0.51
Scott St.	5.0	5.0	10.0	1000	0.36
I. &G. N. RR	5.0	5.0	10.0	1500	0.40

[a]Excluding grades at the major interchanges.

TABLE 6

CONFIDENCE LIMITS OF NATURAL ACCELERATION NOISE (σ_N) ON GRADES

Location	Acceleration Noise (σ_N)	Variance	Confidence Limits
Woodridge Rd.	0.18	0.030	0.18 ± 0.08
Griggs Rd.	0.44	0.009	0.44 ± 0.04
Wayside Dr.	0.11	0.024	0.11 ± 0.07
Telephone Rd.	0.29	0.060	0.29 ± 0.11
South H. B. &T. RR	0.42	0.019	0.42 ± 0.07
North H. B. &T. RR	0.49	0.039	0.49 ± 0.10
Cullen Rd.	0.51	0.046	0.51 ± 0.11
Scott St.	0.36	0.021	0.36 ± 0.07
I. &G. N. RR	0.40	0.018	0.40 ± 0.07

TABLE 7

RESULTS OF REGRESSION ANALYSIS OF ACCELERATION NOISE (Y) ON GRADES

$$Y = b_0 + b_1X_1 + b_2X_2$$

Analysis	F Test	b_0	b_1	b_2	R^2
Absolute difference in adjoining grades and length of grade	12.3**	-0.09827	0.04177**	0.00008-	80.4
Absolute difference in adjoining grades only	25.3**	-0.01415	0.04523**	N/A	78.4

Note: X_1 = absolute difference in adjoining grades,
X_2 = length of grade,
- = not significant at 0.05 level,
** = highly significant relationship,
N/A = not applicable, and
R^2 = percent of variance explained by multiple regression.

TABLE 8

RESULTS OF REGRESSION ANALYSIS OF ACCELERATION NOISE (Y) ON GRADES, $b_0 = 0$

$$Y = b_1X_1$$

Analysis	F Test	b_1	R^2
Absolute difference in adjoining grades	255.8**	0.04367**	97.3

Note: See definitions of terms in Table 7.

the first independent variable X_1. The results of this second analysis showed a very significant linear relationship between the absolute difference in adjoining grades and acceleration noise. The final regression equation was

$$Y = -0.01415 + 0.04523X_1$$

A summary of these results is presented in Table 7.

The constant term b_0 in the above equation represents the y intercept of the regression curve or the value of acceleration noise at zero grade. Based on the data available for the natural noise phase of this study, it is logical to hypothesize that the noise at tangent grade is zero, that is, $b_0 = 0$. A procedure for testing the hypothesis that the intercept is zero was used to determine whether the acceleration noise at tangent grade is zero. The test was not significant at the 0.05 level and the conclusion was drawn that the regression line passes through the origin. Results of this test are presented in the following section of the Appendix.

A final regression was made to determine the value of B_1 with $B_0 = 0$. The final relationship was determined to be

$$Y = 0.4367X_1 \tag{49}$$

where Y = natural acceleration noise (σ_N), and X_1 = absolute difference in adjoining grades. Regression results are given in Table 8.

TEST TO DETERMINE WHETHER THE Y INTERCEPT OF THE LINEAR EXPRESSION OF GRADES (X_1) AND ACCELERATION NOISE (Y) IS ZERO

For a given linear equation of the form Y = A + BX where A represents the Y intercept of the line and B is the slope of the line, the hypothesis that A = 0 is rejected whenever

$$|t| = \left| \frac{a - A}{S_{y/x} \sqrt{\frac{1}{n} + \frac{x^2}{\sum_{i=1}^{n} (x_i - \bar{x})^2}}} \right| \geq t\ \infty/2,\ n - 2$$

where a is an estimate of A, and $S_{y/x}$ is an estimate of the variability about the line. $S^2{}_{y/x}$ is given by

$$S^2{}_{y/x} = \left(\sum_{i=1}^{n} (y_i - \bar{y})^2 - b \left[\sum_{i=1}^{n} (x_i - \bar{x})\ (y_i - \bar{y}) \right] \right) 1/n - 2$$

where b is an estimate of B.

For the relationship $Y = -0.01415 + 0.04523X_1$

$$t_{0.025,7} = 2.365$$

$$|t| = 0.1715 < t_{0.025,7}$$

Therefore, conclude that A = 0.

REGRESSION OF SPEED VS ACCELERATION NOISE

The resulting equation of the model was written as

$$Y = B_0 - B_1X_1 + B_2X_2$$

The problem of determining whether the variability in Y can be explained by the variability in X_1 and X_2 is basically a problem of finding an equation that best explains the data. This is accomplished by estimating the parameters (B_0, B_1, and B_2) of the curve. The best method of estimating these parameters is known as the theory of least squares. This principle applied to the above equation would result in a curve with the statistics b_0, b_1, and b_2 (partial regression coefficients) such that the sum of the squares of the deviations (residuals) between the ordinates of the data points and the ordinates of the curve are minimized:

$$R^2 = \sum_{i=0}^{N} (Y_i - B_0 + B_1X_1 - B_2X_2)^2 \tag{50}$$

where R = residual.

The normal equations found by differentiating Eq. 50 and replacing B_i by their estimates b_i are

$$Nb_0 + b_1\Sigma X_1 + b_2\Sigma X_2 = \Sigma Y_i \tag{51}$$

$$b_0\Sigma X_1 + b_1\Sigma X_1^2 + b_2\Sigma X_1X_2 = \Sigma X_1Y_1 \quad (52)$$

$$b_0\Sigma X_2 + b_1\Sigma X_1X_2 + b_2\Sigma X_2^2 = \Sigma X_2Y_i \quad (53)$$

The constant b_1 is termed the net regression of Y holding X_2 constant and b_2 is the net regression of Y holding X_1 constant. These coefficients can therefore be conveniently written

$$b_1 = b_{y1.2}$$

$$b_2 = by2.1$$

Equations 51 through 53 can thus be written

$$Nb_y + b_{y1.2}\Sigma X_1 + b_{y2.1}\Sigma X_2 = \Sigma Y_i \quad (54)$$

$$b_y\Sigma X_1 + b_{y1.2}\Sigma X_1^2 + by2.1\Sigma X_2 = \Sigma X_1Y_i \quad (55)$$

$$b_y\Sigma X_2 + b_{y1.2}\Sigma X_1X_2 + by2.1\Sigma X_2^2 = \Sigma X_2Y_i \quad (56)$$

Several techniques have been devised for the solution of a system of equations. For this study, a standard regression program from the Data Processing Center, Texas A & M University was utilized. This program employs an inverse matrix technique.

After the statistics b_i have been determined, two other significant tests must be made. First of all, an analysis of variance must be made to test the overall significance of the regression. Secondly, a more detailed test will be required of each regression coefficient in order to assess the contribution of each.

An F-ratio calculated from an analysis of variance table is used to test the significance of the apparent dependence of the dependent variables. The null hypothesis tested by the F-ratio is $B_{y1.2} = B_{y2.1} = 0$.

The quantity $(b_{y1.2} - B_{y1.2})/S_{by1.2}$ is distributed as t. To test the significance of each partial regression coefficient, the hypothesis $B_{y1.2} = B_{y2.1} = 0$ is tested by the Students t-test:

$$t_1 = b_{y1.2}/S_{by1.2}$$

$$t_2 = b_{y2.1}/S_{bt2.1}$$

TRAFFIC FLOW NOTATION

Term	Symbol	Units	Term	Symbol	Units
Acceleration	a	dist/time^2	Energy (kinetic)	E	veh-dist/time^2
Acceleration noise (internal energy)	σ	dist/time^2	Internal energy (acceleration noise)	I	veh-dist/time^2
Maximum noise due to interaction	σ_M	dist/time^2	Total energy (max. accel. noise)	T	veh-dist/time^2
Natural noise	σ_N	dist/time^2	Flow (volume, demand, momentum)	q	veh/time
Total noise	σ_T	dist/time^2	Critical flow (capacity)	q_m	veh/time
Traffic interaction	σ_t	dist/time^2	Optimum flow	q'_m	veh/time
Concentration (density)	k	veh/dist	Velocity (speed), individual vehicle	x	dist/time
Critical concentration	k_m	veh/dist	Velocity (speed), traffic stream	u	dist/time
Jam concentration	k_j	veh/dist	Critical speed	u_m	dist/time
Optimum concentration	k'_m	veh/dist	Free speed	u_f	dist/time
Constant of proportionality	c	—	Optimum speed	u'_m	dist/time

The Relationships of Vehicle Classification and Geometric Characteristics to Peak Period Freeway Volumes

VASANT H. SURTI, Assistant Professor of Civil Engineering, The Catholic University of America, Washington, D.C.

The three important parameters describing the operation of a highway facility are speed, volume and density of the traffic stream. This report is concerned with the volume parameter.

An efficient surveillance and control system for an urban freeway should have the ability to obtain information on vehicles at several locations. To locate detection equipment in each lane at several locations is an expensive proposal.

This paper describes the development of linear multiple regression models to predict total freeway volumes as well as individual lane volumes. Classification of vehicles along with the interlane volume characteristics is also considered.

•THE study site was a 3.2-mi section of John C. Lodge Freeway between the Edsel Ford and Davison interchanges in Detroit (Fig. 1). The Lodge Freeway is a depressed facility and the study section was built between 1950 and 1955. It is primarily a six-lane freeway; however, 34 percent of the study section has eight lanes. There are five on-ramps and five off-ramps for northbound (outbound from central business district) traffic and six on-ramps and five off-ramps for southbound (inbound to central business district) traffic. The study section is also the site of the National Proving Ground for Freeway Surveillance, Control and Electronic Traffic Aids, where 14 television monitors at a centrally located control center provide a continuous view of traffic conditions. The picture to each monitor is transmitted from one of 14 transistorized television cameras.

DATA COLLECTION

At two locations, 24-hr classified counts were made manually by 1-min periods on a weekday. These locations were southbound John C. Lodge at the north side of Calvert Ave. bridge and northbound John C. Lodge at the south side of Chicago Blvd. bridge.

At several other locations sample counts were made on a short-time base. The sampling process used was a 60-min classified count by lane by 1-min periods. The samples were taken on weekdays from 7:00 a.m. to 9:00 a.m. and 4:00 p.m. to 6:00 p.m. The following is a list of the locations at which the classified vehicle traffic counts were made (Fig. 2).

1. Southbound 460 ft north of E. Davison Ave. entrance ramp.
2. Southbound 80 ft north of Glendale Ave. overpass.
3. Southbound 220 ft south of Highland Ave. overpass.
4. Southbound 330 ft north of Webb Ave.
5. Southbound at north side of Calvert Ave.
6. Southbound at north side of Chicago Blvd.

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 45th Annual Meeting.

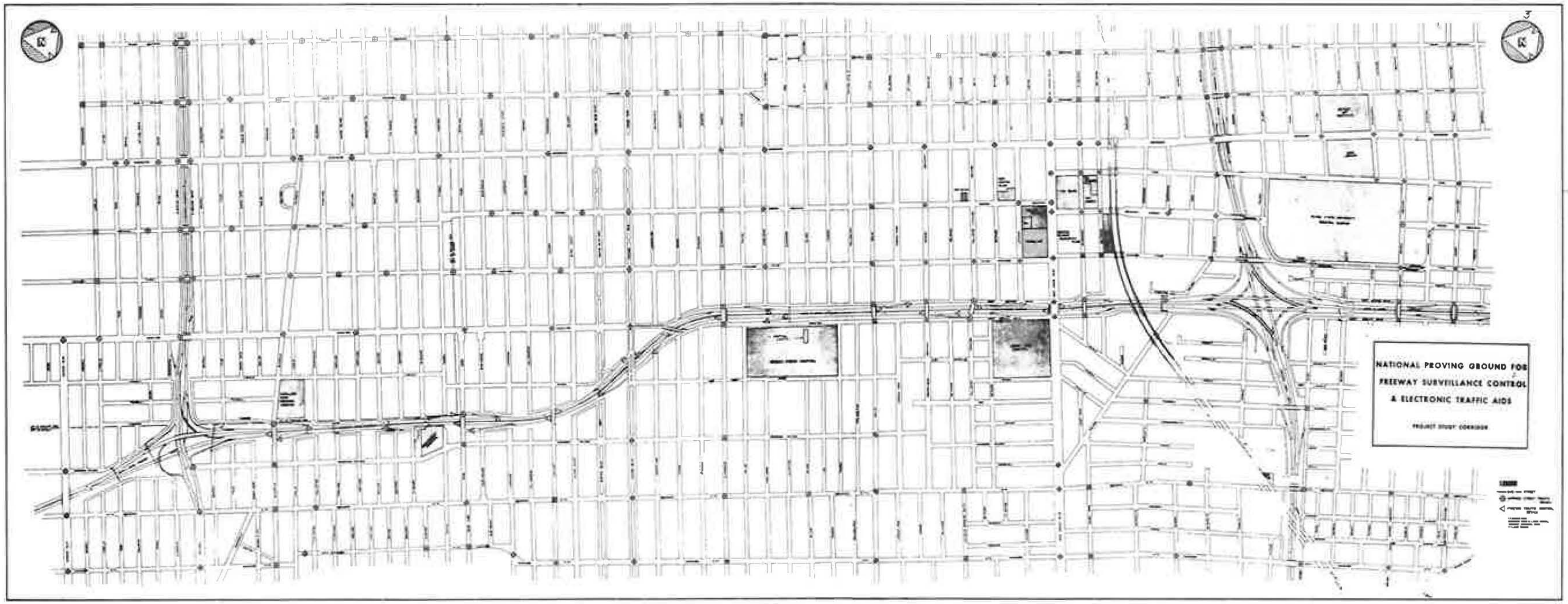
NATIONAL PROVING GROUND FOR
FREEWAY SURVEILLANCE CONTROL
& ELECTRONIC TRAFFIC AIDS
PROJECT STUDY CORRIDOR

Figure 1.

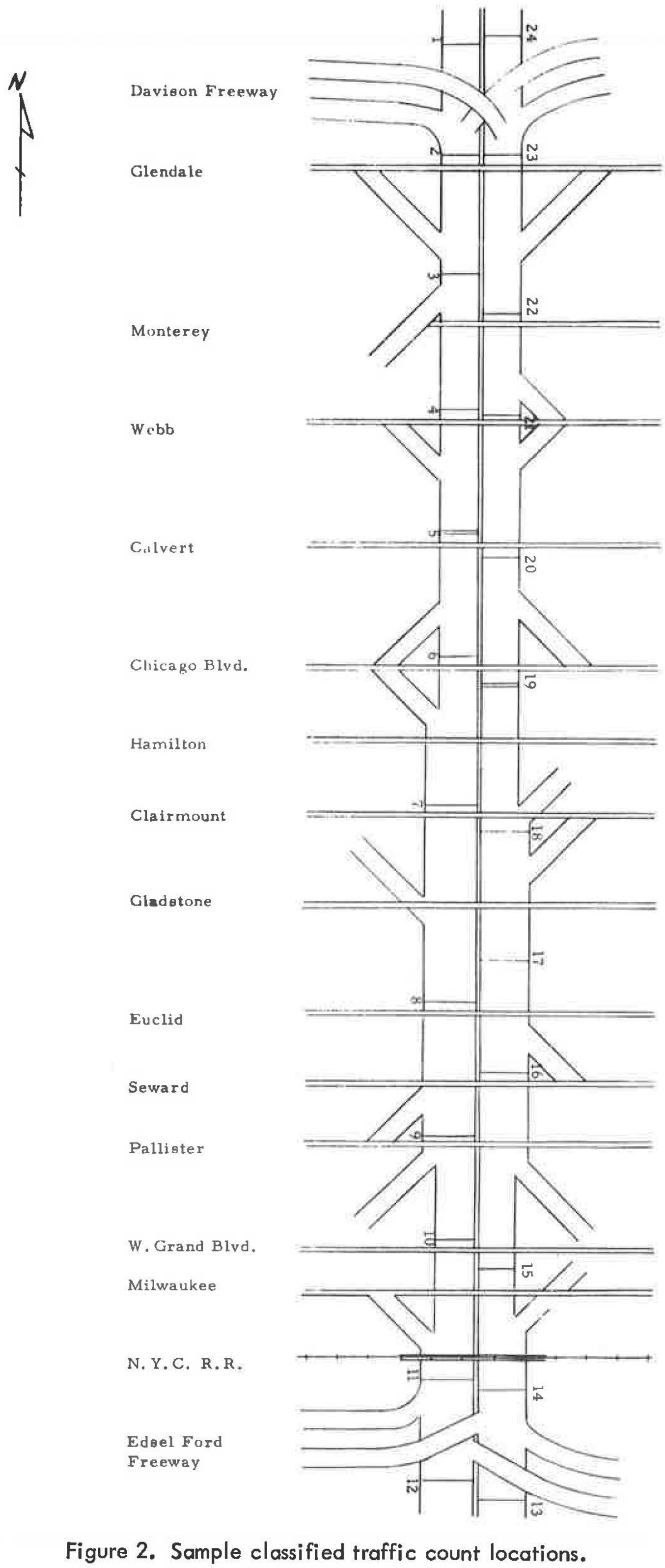

Figure 2. Sample classified traffic count locations.

7. Southbound at north side of Clairmount Ave. overpass.
8. Southbound at north side of Euclid Ave. overpass.
9. Southbound 130 ft north of Pallister Ave. overpass.
10. Southbound 80 ft north of West Grand Blvd. overpass.
11. Southbound at Holden Ave. pedestrian overpass.
12. Southbound 330 ft south of eastbound Edsel Ford exit ramp.
13. Northbound 570 ft south of northbound Edsel Ford entrance ramp.
14. Northbound 150 ft south of Holden Ave. overpass.
15. Northbound 160 ft north of Milwaukee Ave. overpass.
16. Northbound 80 ft north of Seward Ave. overpass.
17. Northbound 610 ft south of Gladstone Ave. overpass.
18. Northbound 110 ft south of Clairmount Ave. overpass.
19. Northbound at south side of Chicago Blvd. overpass.
20. Northbound at south side of Calvert Ave. overpass.
21. Northbound 80 ft north of Webb Ave.
22. Northbound 80 ft north of Monterey Ave.
23. Northbound 80 ft north of Glendale Ave.
24. Northbound 50 ft south of Davison Ave. entrance ramp.

The field data were classified into three general classes and nine specific types: (a) passenger vehicles—standard, compact, and foreign; (b) trucks—panels and pickups, single unit, and tractor-trailer combinations; and (c) miscellaneous—buses, motorcycles, and car and trailer combinations.

To base the analysis on operating characteristics of the vehicles rather than their specific use or designation, the initial classification was further investigated. The Automobile Manufacturers Association was contacted to determine the differences with respect to acceleration, deceleration, and maneuverability between passenger cars and those vehicles generally referred to as "panel and pickup" trucks. It was found that there was no significant difference and hence they were classified along with the passenger vehicles. The car and trailer and motorcycles represented an extremely small number of vehicles and were eliminated from the study. The buses were grouped with single-unit trucks. Thus the final classification was as follows: (a) light—passenger vehicles, panels and pick-ups; (b) single unit—single-unit trucks and buses; and (c) combination—combination trucks.

ANALYSIS PERIODS

Due to manpower limitations, it was not possible to locate observers at all selected stations at the same time for either morning or afternoon data collection. This difficulty was overcome by staggering the hour of counts for each location. For example, at one location, data were collected from 7:00 a.m. to 8:00 a.m., the next 7:15 a.m. to 8:15 a.m., and so forth. Since the traffic behavior and characteristics for the entire high-volume period (7:00 a.m. to 9:00 a.m. or 4:00 p.m. to 6:00 p.m.) were not uniform and since the counts were staggered over a 2-hr period, it was necessary to select some period for each sample 1-hr count that would represent typical composition of traffic for each location. The selections of an arithmetic, geometric or running 15-min average would have a high or low bias effect depending on the time when the data were collected. Some period during the hour had to be obtained which represented peaking characteristics of the traffic. In a previous study (1) it was found that a definite peak period existed within the peak hour for vehicle arrivals at urban intersections. It was decided to determine if such a definite peak period existed during the peak hour for the arrival of vehicles at selected locations on the freeway, and if so, to use this peak period within the peak hour as a basis for analysis. The existence and duration of such a peak period within the peak hour was determined in the following manner.

At each location, 1-hr counts were available by 1-min intervals. These counts showed a wide fluctuation in volumes; hence they were grouped into 5-min intervals. A graph of time against total lane volumes for the 5-min intervals was plotted. On this graph was superimposed the value of a 5-min arithmetic mean volume. The longest period that exceeded this value was identified as the peak period. The duration of the peak period was read from the graph to the nearest 5 min.

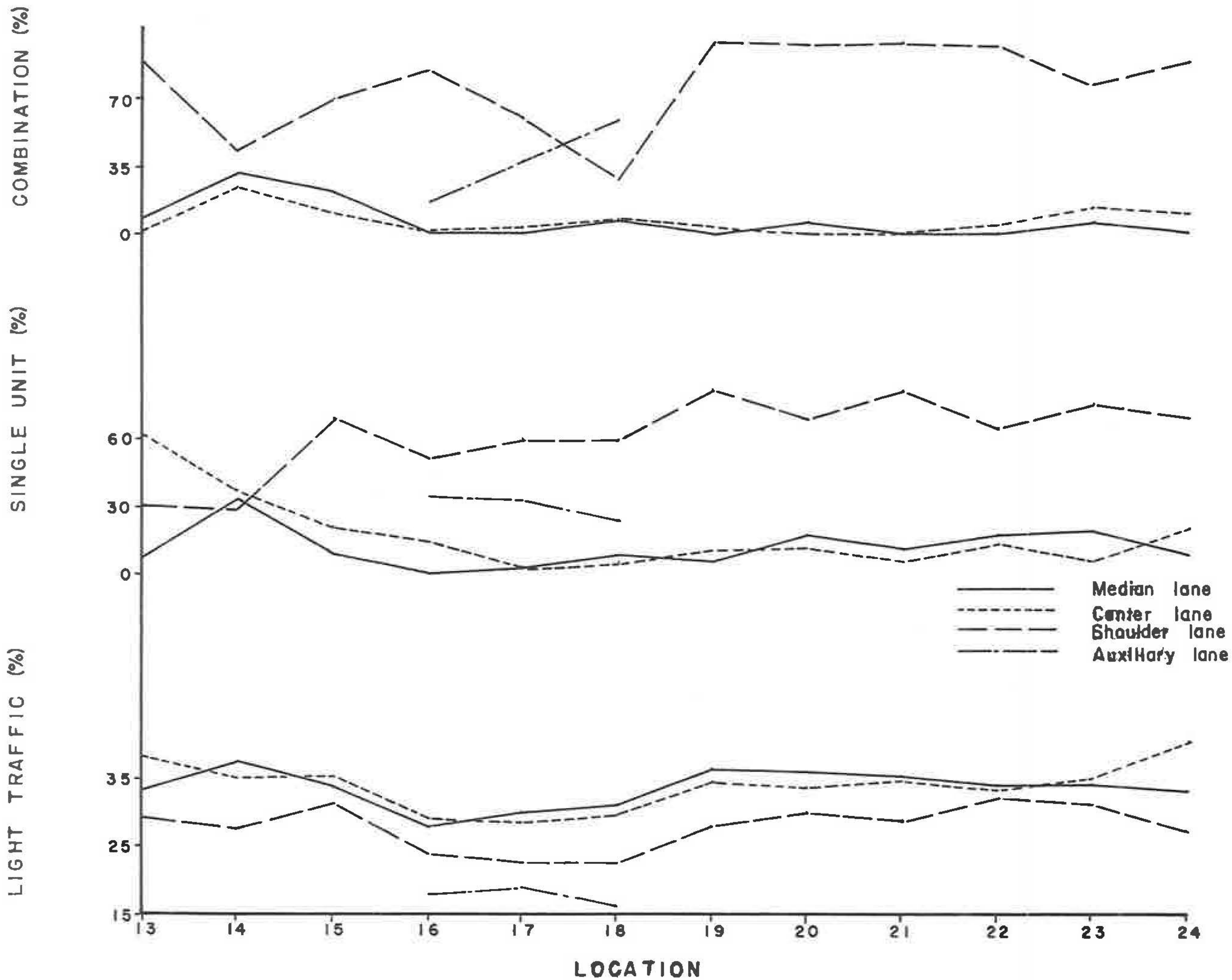

Figure 3. Classified vehicle distribution, afternoon peak—northbound.

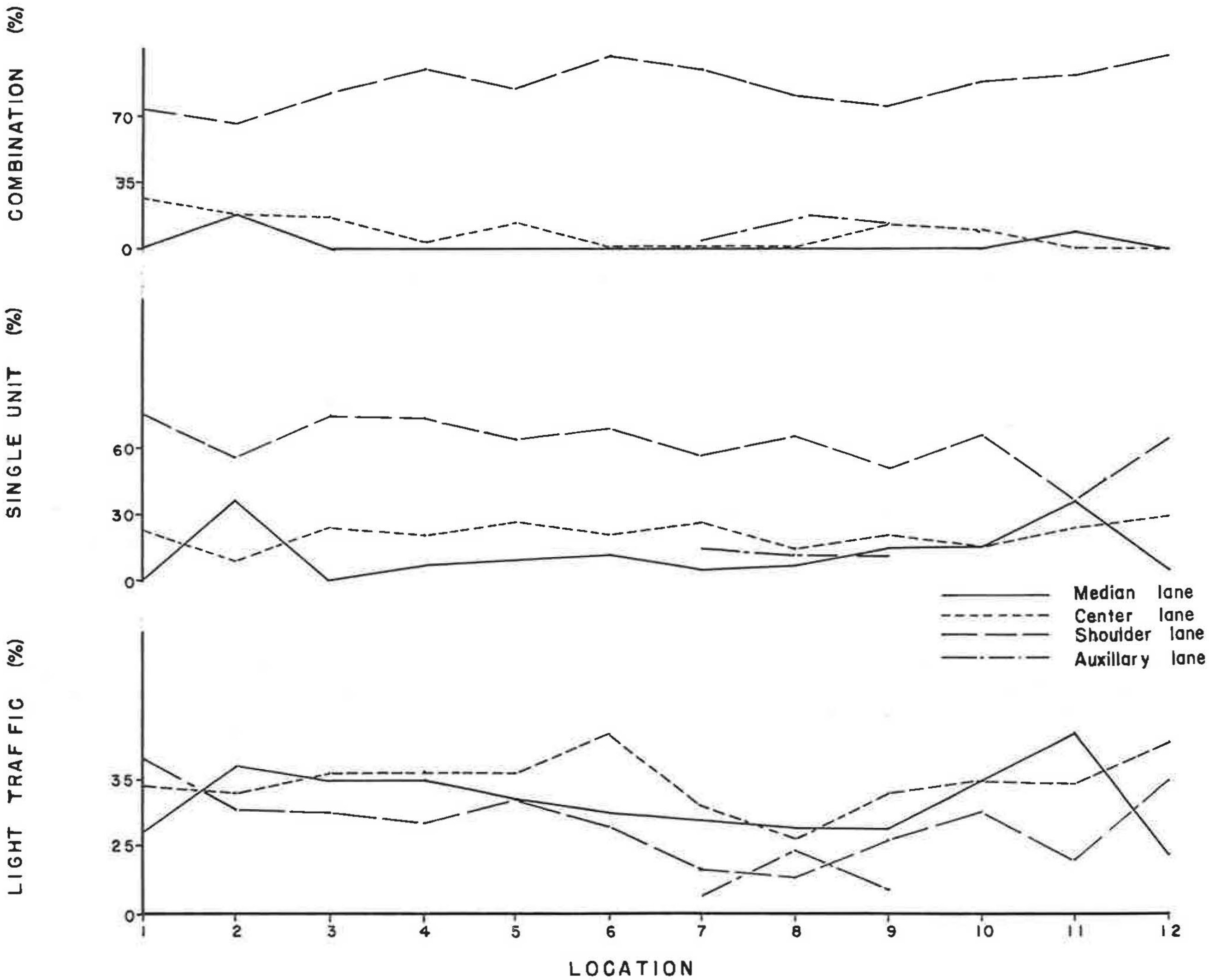

Figure 4. Classified vehicle distribution, morning peak—southbound.

TABLE 1

VEHICLE CLASSIFICATION DISTRIBUTION, A.M. PEAK—SOUTHBOUND

Location	Classification	Median Lane (%)	Center Lane (%)	Shoulder Lane (%)	Auxiliary Lane (%)
1	Light	27.2	34.2	38.6	
	Single unit	0.0	25.0	75.0	
	Combination	0.0	25.0	75.0	
	All vehicles	26.6	34.0	39.4	
2	Light	37.0	32.4	30.6	
	Single unit	35.3	9.8	54.9	
	Combination	16.7	16.7	66.6	
	All vehicles	36.6	31.3	32.1	
3	Light	34.5	35.2	30.3	
	Single unit	0.0	24.0	76.0	
	Combination	0.0	16.7	83.3	
	All vehicles	34.0	35.0	31.0	
4	Light	35.0	36.4	28.6	
	Single unit	7.1	19.1	73.8	
	Combination	0.0	3.3	96.7	
	All vehicles	34.1	35.8	30.1	
5	Light	31.8	36.2	32.0	
	Single unit	8.0	28.0	64.0	
	Combination	0.0	14.3	85.7	
	All vehicles	31.2	35.9	32.9	
6	Light	29.6	42.1	28.3	
	Single unit	12.0	20.0	68.0	
	Combination	0.0	0.0	100.0	
	All vehicles	29.1	41.5	29.4	
7	Light	29.0	31.2	21.8	18.0
	Single unit	5.4	26.8	55.3	12.5
	Combination	0.0	0.0	96.9	3.1
	All vehicles	28.4	30.8	23.0	17.8
8	Light	28.3	26.5	20.5	24.7
	Single unit	7.1	14.3	67.9	10.7
	Combination	0.0	0.0	80.0	20.0
	All vehicles	27.7	26.1	21.9	24.3
9	Light	27.7	27.8	25.7	18.8
	Single unit	16.2	18.9	51.4	13.5
	Combination	0.0	12.5	75.0	12.5
	All vehicles	27.3	27.5	26.5	18.7
10	Light	34.5	35.0	30.5	
	Single unit	16.7	16.7	66.6	
	Combination	0.0	11.0	89.0	
	All vehicles	34.0	34.5	31.5	
11	Light	43.1	34.3	22.6	
	Single unit	38.1	23.8	38.1	
	Combination	9.0	0.0	91.0	
	All vehicles	42.8	33.9	23.3	
12	Light	23.6	41.3	35.1	
	Single unit	6.5	29.0	64.5	
	Combination	0.0	0.0	100.0	
	All vehicles	23.3	40.9	35.8	

TABLE 2

VEHICLE CLASSIFICATION DISTRIBUTION, P.M. PEAK—NORTHBOUND

Location	Classification	Median Lane (%)	Center Lane (%)	Shoulder Lane (%)	Auxiliary Lane (%)
13	Light	32.4	38.4	29.2	
	Single unit	7.7	61.5	30.8	
	Combination	8.6	2.9	88.5	
	All vehicles	31.3	37.8	30.9	
14	Light	37.5	35.0	27.5	
	Single unit	33.3	38.0	28.7	
	Combination	31.3	25.0	43.7	
	All vehicles	37.2	34.6	28.2	
15	Light	33.4	35.4	31.2	
	Single unit	10.0	20.0	70.0	
	Combination	20.0	10.0	70.0	
	All vehicles	32.6	34.9	32.5	
16	Light	28.5	29.2	24.0	18.3
	Single unit	0.0	15.0	50.0	35.0
	Combination	0.0	0.0	83.5	16.5
	All vehicles	27.8	28.8	24.8	18.6
17	Light	29.5	28.7	22.6	19.2
	Single unit	3.1	3.1	61.0	32.8
	Combination	0.0	3.8	57.7	38.5
	All vehicles	28.4	27.6	24.0	20.0
18	Light	31.0	30.8	22.4	15.8
	Single unit	10.3	6.9	58.6	24.6
	Combination	7.2	7.2	28.6	57.0
	All vehicles	30.3	30.0	23.5	16.2
19	Light	36.6	34.8	28.6	
	Single unit	7.9	10.1	82.0	
	Combination	0.0	3.8	96.2	
	All vehicles	35.6	34.0	30.4	
20	Light	36.2	33.5	30.3	
	Single unit	18.0	12.0	70.0	
	Combination	5.3	0.0	94.7	
	All vehicles	35.8	33.1	31.1	
21	Light	35.8	35.3	28.9	
	Single unit	11.1	5.5	83.4	
	Combination	0.0	0.0	100.0	
	All vehicles	35.2	34.6	30.2	
22	Light	33.9	33.6	32.5	
	Single unit	18.9	16.2	64.9	
	Combination	0.0	5.5	94.5	
	All vehicles	33.5	33.2	33.3	
23	Light	33.8	34.9	31.3	
	Single unit	20.0	5.0	75.0	
	Combination	7.1	14.2	78.7	
	All vehicles	33.5	34.5	32.0	
24	Light	32.5	40.5	27.0	
	Single unit	10.0	20.0	70.0	
	Combination	0.0	10.0	90.0	
	All vehicles	32.3	40.3	27.4	

The peak periods for two locations for the northbound afternoon peak and southbound morning peak are presented as examples in Appendix A. The shortest and longest durations of the peak periods were found to be 10 min and 40 min respectively. The average duration of the peak period within the peak hour was higher for the afternoon peak (20.4 min) than the corresponding morning peak period (17.2 min).

VEHICLE DISTRIBUTION

Figures 3 and 4 represent the classified vehicle distribution for the afternoon and morning peak periods respectively. Median and center lanes carried a higher percent of light traffic than either shoulder or auxiliary lane throughout the freeway study section during the afternoon peak, and the same was true for the morning peak with the exception of two boundary locations. On the average, the center lane was a major light-vehicle traffic carrier during the morning peak, whereas approximately equal percentage of light-vehicle traffic was carried by median and shoulder lanes during the afternoon peak.

Single-unit and combination trucks were primary users of the shoulder lane throughout the study section with the exception of location 13 during the afternoon peak, when the center lane carried a higher percentage of single-unit trucks than the shoulder lane. This was probably due to prior knowledge of entrance ramps from left and right just a short distance ahead at the Davison interchange and hence, to avoid delay in merging, there was a tendency to stay in the middle lane.

The existence of an auxiliary lane considerably reduced the usage of the shoulder lane by combination trucks during the afternoon peak. This was also true for single-unit trucks but the reduction was not as significant as in the case of combination trucks. The auxiliary lane helped in redistributing lane traffic for all vehicle classifications.

Tables 1 and 2 give the percentage of vehicle classification distribution by lane, location, time and direction. The average percentage of center-lane usage by all vehicles for the entire study area was slightly higher during the morning peak than the afternoon peak (35.5 percent a.m. peak, 3-lane section; 35.2 percent p.m. peak, 3-lane section; 29.3 percent a.m. peak, 4-lane section; 28.8 percent p.m. peak, 4-lane section). However, in the case of the median lane, usage was reversed (34.1 percent p.m. peak, 3-lane section; 32.4 percent a.m. peak, 3-lane section; 28.9 percent p.m. peak, 4-lane section; 27.8 percent a.m. peak, 4-lane section).

Tables 3 and 4 give the composition of traffic by location, time and direction. Light vehicles comprised the highest traffic volumes, ranging from a low of 94.5 percent to

TABLE 3

COMPOSITION OF TRAFFIC, A.M. PEAK—SOUTHBOUND

Location	Vehicle Classification		
	Light	Single Unit	Combination
1	97.9	1.6	0.5
2	94.5	3.7	1.8
3	98.5	1.2	0.3
4	97.4	1.5	1.1
5	97.6	1.5	0.9
6	97.8	1.6	0.6
7	97.3	1.7	1.0
8	97.3	2.3	0.4
9	97.3	2.0	0.7
10	97.5	1.9	0.6
11	97.8	1.4	0.8
12	97.6	2.1	0.3

TABLE 4

COMPOSITION OF TRAFFIC, P.M. PEAK—NORTHBOUND

Location	Vehicle Classification		
	Light	Single Unit	Combination
13	95.4	2.0	2.6
14	94.5	1.6	3.9
15	96.6	2.3	1.1
16	97.6	1.9	0.5
17	96.0	2.8	1.2
18	96.5	2.4	1.1
19	97.1	2.2	0.7
20	98.3	1.2	0.5
21	97.7	1.8	0.5
22	98.0	1.3	0.7
23	98.4	0.9	0.7
24	98.5	1.3	0.2

TABLE 5

SIMPLE CORRELATIONS OF FACTORS INFLUENCING LANE VOLUMES—AFTERNOON PEAK

Factor	No. of Comb. Veh in Shoulder Lane/5 Min	No. of S.U. Veh in Shoulder Lane/5 Min	Total Veh/5 Min in Median Lane	Total Veh/5 Min in Center Lane	Total Veh/5 Min in Shoulder Lane	Dist to Nearest Upstream Entrance Ramp (ft)	Dist to Nearest Downstream Exit Ramp (ft)	Time of Day	Total Veh/5 Min in All 3 Lanes
No. of comb. veh in shoulder lane/5 min	1.0000	0.2014	0.0937	0.1917	0.0132	-0.0017	-0.0521	0.0854	0.1167
No. of S.U. veh in shoulder lane/5 min		1.0000	0.2651	0.2169	0.2183	0.1807	0.0329	-0.3314	0.2702
Total veh/5 min in median lane			1.0000	0.7227	0.6564	-0.0939	-0.6642	-0.0744	0.9280
Total veh/5 min in center lane				1.0000	0.5060	-0.1514	-0.3741	0.1367	0.8592
Total veh/5 min in shoulder lane					1.0000	-0.2681	-0.4661	0.1087	0.8128
Dist to nearest upstream entrance ramp (ft)						1.0000	0.3526	-0.4531	-0.1180
Dist to nearest downstream exit ramp (ft)							1.0000	-0.0242	-0.5868
Time of day (to nearest 5 min)								1.0000	0.0554
Total veh/5 min in all 3 lanes									1.0000

TABLE 6

SIMPLE CORRELATIONS OF FACTORS INFLUENCING LANE VOLUMES—MORNING PEAK

Factor	No. of Comb. Veh in Shoulder Lane/5 Min	No. of S.U. Veh in Shoulder Lane/5 Min	Total Veh/5 Min in Median Lane	Total Veh/5 Min in Center Lane	Total Veh/5 Min in Shoulder Lane	Dist to Nearest Upstream Entrance Ramp (ft)	Dist to Nearest Downstream Exit Ramp (ft)	Time of Day	Total Veh/5 Min in All 3 Lanes
No. of comb. veh in shoulder lane/5 min	1.0000	-0.0228	-0.0269	-0.0685	-0.0258	0.0192	0.0653	0.3172	-0.0563
No. of S.U. veh in shoulder lane/5 min		1.0000	-0.0205	-0.1340	0.0535	-0.2411	-0.2513	0.3029	-0.0492
Total veh/5 min in median lane			1.0000	0.1882	0.3651	0.0643	-0.2641	-0.2246	0.7969
Total veh/5 min in center lane				1.0000	0.3326	-0.0663	0.3773	-0.4498	0.7084
Total veh/5 min in shoulder lane					1.0000	-0.2766	-0.3057	-0.3470	0.7687
Dist to nearest upstream entrance ramp (ft)						1.0000	0.5417	-0.1614	-0.1252
Dist to nearest downstream exit ramp (ft)							1.0000	-0.1360	-0.0769
Time of day (to nearest 5 min)								1.0000	-0.4701
Total veh/5 min in all 3 lanes									1.0000

a high of 98.5 percent. The range of traffic volumes for single-unit and combination vehicles was quite broad. At most locations the percentage of single-unit traffic was higher than that of combination truck traffic.

CORRELATION ANALYSES

A simple correlation coefficient, r, indicates the extent to which two characteristics are associated. It is a numerical index that varies from 0 to -1.00 in the negative direction, and from 0 to +1.00 in the positive direction. The closer the number is to ±1.00, the stronger is the degree of association between two characteristics. If the sign of r is positive between two factors influencing lane traffic volumes, it means that as one factor increases in magnitude, the other factor is also expected to increase. Where r has a negative value, it means that as one factor increases in magnitude, the other factor should decrease. Perfect correlation between the factors would be indicated by a value of ± 1 and no correlation by a value of zero. Tables 5 and 6 give simple correlation coefficients between factors influencing freeway traffic volumes. This simple correlation coefficient must be interpreted with caution. It does not reveal the extent to which one of the factors causes the change in the other factor. It merely indicates how much, as one of the factors varies in a given direction, the other factor varies also in the same or opposite direction. The highest correlations were found to exist among individual lane and total traffic volumes. The correlations were higher during the afternoon peak than during the morning peak.

STATISTICAL MODELS

During the morning and afternoon rush periods the traffic volumes carried on an urban freeway are dependent on several factors. Use of statistics is made here to analyze selected traffic and geometric freeway characteristics at various locations along a section of an urban freeway.

Multivariate analyses of the data are based on the multiple regression method. The primary aim is to develop an equation by which values of the criterion or dependent variable can be computed by knowledge of the independent or predictor variables. In general, therefore, with a dependent or criterion variable Y, and P independent variables X_1, S_2, . . . , X_p, the relationship will be of the form

$$Y = a + b_1X_1 + b_2X_2 + b_3X_3 + \dots + b_pX_p$$

where b_1, b_2, . . . , b_p are referred to as partial regression coefficients.

The b values are determined such that the sum of squares of deviation between actual values of the criterion variable, Y, and the value computed by the equation is a minimum. Thus it is required to minimize the errors of estimation and this leads by the method of least squares to p simultaneous equations. For three independent variables, the simultaneous equation will be of the form

$$b_1\Sigma X_1^2 + b_2\Sigma X_1X_2 + b_3\Sigma X_1X_3 = \Sigma X_1Y$$

$$b_1\Sigma X_2X_1 + b_2\Sigma X_2^2 + b_3\Sigma X_2X_3 = \Sigma X_2Y$$

$$b_1\Sigma X_3X_1 + b_2\Sigma X_3X_2 + b_3\Sigma X_3^2 = \Sigma X_3Y$$

The solution of these equations gives the values of b_1, b_2 and b_3 to be inserted into the prediction equation

$$Y = b_1X_1 + b_2X_2 + b_3X_3$$

The method can be extended to any number of variables. However, as the number of equations increases the solution of simultaneous equations becomes extremely laborious and the use of a computer becomes essential.

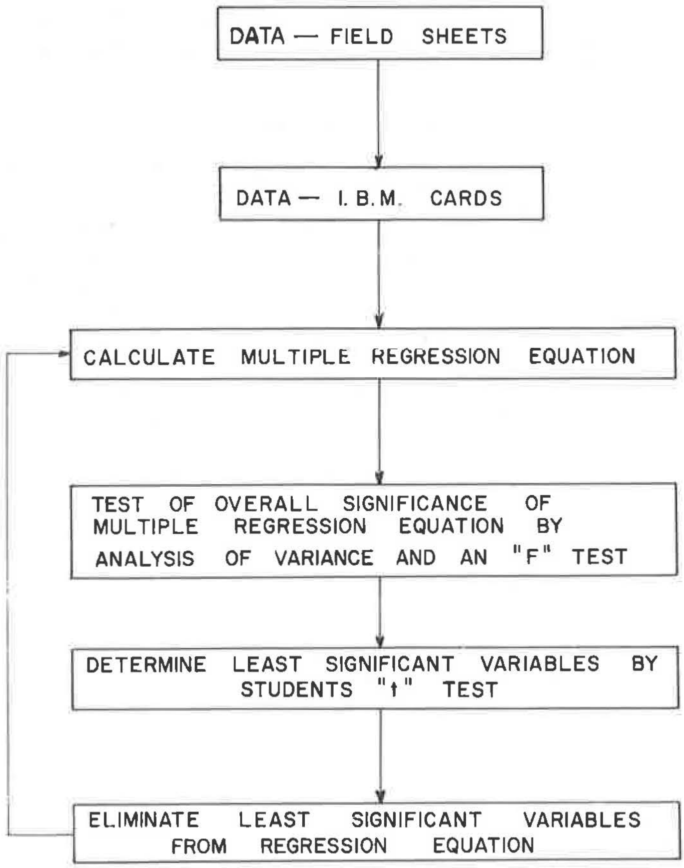

Figure 5. Development of the statistical models.

The data were summarized for each location and transferred to IBM cards. Michigan State University's CDC 3600 computer was used for the analysis. A schematic of the development of the regression models is shown in Figure 5.

The variables used in developing the models were (a) number of combination vehicles in shoulder lane per 5 min, (b) number of single-unit vehicles in shoulder lane per 5 min, (c) median-lane volume per 5 min, (d) center-lane volume per 5 min, (e) shoulder-lane volume per 5 min, (f) distance to nearest upstream entrance ramp, (g) distance to nearest downstream exit ramp, (h) time of day and (i) total volume per 5 min.

First a regression equation was computed to predict an individual lane volume or total three-lane volume as the dependent variable; all the remaining variables were considered as independent variables. Student's t test was used to test the significance of partial regression coefficients for each of the variables. The regression coefficients that were found to be insignificant at a significance level of 10 percent or better were eliminated from the equation and a new equation computed.

Statistics computed along with the regression equation were beta weights, determining coefficient (R^2), R^2 deletes, multiple correlation coefficient (R), standard error of predicted value and partial correlation coefficients.

The beta weights indicate the respective weight given to each of the independent variables in the regression equation. Thus a variable with the highest beta weight is of most importance.

The determining coefficient (R^2) is a measure of how closely the variability in the predicted value is explained by the composite knowledge of independent variables. The values of R^2 delete correspond to each independent variable as R^2 which would have been obtained had the independent variable not been included in the regression equation. The multiple correlation coefficient (R) is the square root of the determining coefficient and is similar to the simple correlation coefficient discussed previously. It represents the degree of association between the dependent variable and the independent variables.

The standard error of predicted value measures how closely the predicted value agrees with the observed value. A smaller value of the standard error would indicate a better model.

The partial correlation coefficient indicates the relationship between the dependent variable and an independent variable when the effect of remaining independent variables is controlled.

An overall test of significance for each regression equation was made using the analysis of variance technique and an F-test. The results are shown in Appendix B.

RESULTS

Median-Lane Volume Models—Afternoon Peak

Two models for predicting median-lane (lane 1) volume are described.

$$X_3 = -0.4216 + -0.0022\, X_6 - 0.0044\, X_7 + 0.3576\, X_9 \qquad (1)$$

In Model 1 median-lane volume is closely related to total traffic volume in all three lanes, distance to nearest downstream exit ramp and distance to nearest upstream entrance ramp. It is interesting to note the opposite signs of the regression coefficients for these two distances. With the increase in the nearest upstream entrance ramp distance the median-lane volume increases, whereas the increase in the nearest downstream exit ramp distance causes the median-lane volume to decrease. The multiple correlation coefficient for this model is 0.95 and the determining coefficient, 0.90. The test for overall significance shows that this model is highly significant (0.01 level of significance). The beta weights indicate that total three-lane traffic volume, distance to nearest downstream exit ramp, and distance to nearest upstream entrance ramp are of importance in that order. Statistical data for this model are given in Table 7, Appendix B.

Model 2 relates median-lane volume to center-lane volume, shoulder-lane volume, distance to nearest upstream entrance ramp and distance to nearest downstream exit ramp:

$$X_3 = 35.3481 + 0.5273\, X_4 + 0.3634\, X_5 + 0.0031\, X_6 - 0.0081\, X_7 \qquad (2)$$

The multiple correlation coefficient and the determining coefficient values are 0.88 and 0.78 respectively. With an F ratio of 6.01, the model is highly significant at 0.01 level of significance. The beta weights of the four variables considered are nearly the same; that is, each variable is equally important in predicting the median-lane volume. The standard error of predicted median-lane volume is 10.77. Statistical data for this model are given in Table 8, Appendix B.

Median-Lane Volume Models—Morning Peak

Only one model for predicting median-lane volume during the morning peak gave reasonably satisfactory results. In this model median-lane volume is found to be closely related to distance to nearest upstream entrance ramp, distance to nearest downstream exit ramp, and total three-lane volume per 5 min:

$$X_3 = 0.4349 + 0.0055\, X_6 - 0.0048\, X_7 + 0.3289\, X_9 \qquad (3)$$

This model is highly significant at 0.01 level of significance. The multiple correlation and determining coefficients are 0.80 and 0.64 respectively. The reliability of this model is not as good as Model 1 for the afternoon peak. Inspection of beta weights indicates that total three-lane volume was the most significant factor in predicting the median-lane volume. Statistics for this model are given in Table 9, Appendix B.

Center-Lane Volume Models—Afternoon Peak

The three-dimension model given is based on distance to nearest downstream exit ramp and total three-lane volume:

$$X_4 = -17.3155 + 0.0031 X_7 + 0.3629 X_9 \tag{4}$$

The multiple correlation and determining coefficients are 0.87 and 0.76 respectively. With an F ratio of 55.2 this model is highly significant at 0.01 level. The statistics for this model are shown in Table 10, Appendix B.

Center-Lane Volume Models—Morning Peak

The best model for predicting center-lane volume during the morning peak relates distance to nearest upstream entrance ramp, distance to nearest downstream exit ramp and total three-lane volume:

$$X_4 = 2.5122 - 0.0045 X_6 + 0.0073 X_7 + 0.3404 X_9 \tag{5}$$

Unlike the model for the afternoon peak, distance to nearest upstream entrance ramp had a significant effect on the model. The variables in the order of their importance in the prediction value are total three-lane volume, and distance to nearest upstream entrance ramp. The multiple correlation and determining coefficients are 0.87 and 0.75 respectively, and the model is highly significant at 0.01 level. The statistics for this model are given in Table 11, Appendix B.

Shoulder-Lane Volume Models—Afternoon Peak

The best model relates shoulder-lane volume to distance to nearest upstream entrance ramp and total three-lane volume:

$$X_5 = 24.5834 - 0.0014 X_6 + 0.2669 X_9 \tag{6}$$

This model is highly significant at the 0.01 level, and the multiple correlation and determining coefficients are 0.82 and 0.67 respectively. The sign of the regression coefficient for the variable distance to the nearest upstream entrance ramp is important; the interpretation is that with the increase in distance to the nearest entrance ramp, the shoulder-lane volume decreases. This is quite logical since the vehicles entering the freeway first use the shoulder lane before orienting themselves in one of the three lanes. The sign for the same variable in the models for predicting the median-lane volumes is opposite; that is, with the increase in distance to the nearest entrance ramp, the median-lane volume also increases. Statistics for this model are given in Table 12, Appendix B.

Shoulder-Lane Volume Models—Morning Peak

Unlike the model for afternoon peak, in the best model, the effect of distance to nearest upstream entrance ramp in predicting the shoulder-lane volume is insignificant. Instead, the shoulder-lane volume is closely related to distance to nearest downstream exit ramp and total three-lane volume:

$$X_5 = -4.9233 - 0.0028 X_7 + 0.3333 X_9 \tag{7}$$

The multiple correlation and determining coefficients are 0.81 and 0.65 respectively. This model is significant at the 0.01 level. Statistics for this model are given in Table 13, Appendix B.

Total Three-Lane Volume Models—Afternoon Peak

Four very satisfactory models were developed to predict the total three-lane volumes during the afternoon peak. The multiple correlation coefficient had a range of 0.94 (high) to 0.85 (low). The determining coefficient ranged from 0.88 (high) to 0.72 (low). Each model relates total three-lane volume to either median, center- or shoulder-lane volume and other time or geometric variables. The models are

$$X_9 = 133.9218 + 2.097\,X_3 - 0.0036\,X_6 \quad (8)$$

$$X_9 = -66.5754 + 2.1405\,X_3 + 0.1086\,X_8 \quad (9)$$

$$X_9 = 180.0925 + 2.2271\,X_2 + 0.0138\,X_7 + 1.9087\,X_4 \quad (10)$$

$$X_9 = 190.6431 + 2.0403\,X_5 - 0.0112\,X_7 \quad (11)$$

The best estimates for total three-lane volumes during the afternoon peak are obtained from the median- and center-lane volume (8, 9, 10) models. The shoulder-lane model is not as reliable, probably due to high rates of merging and exiting vehicles. Having the knowledge of median-lane volume and either the time of day or the distance to nearest upstream entrance ramp, one can predict the total three-lane volume with high reliability through the use of the appropriate model. To predict total three-lane volume from the information on center-lane volume, the number of single-unit vehicles in shoulder lane and the distance to nearest downstream exit ramp must also be known. All models are significant at the 0.01 level. Statistics for these models are given in Tables 14 through 17, Appendix B.

Total Three-Lane Volume Models—Morning Peak

Three models for predicting the total three-lane volume during the morning peak are given by Eqs. 12 through 14.

$$X_9 = 411.7560 - 0.0118\,X_6 + 0.0070\,X_7 + 1.6012\,X_3 - 0.2311\,X_8 \quad (12)$$

$$X_9 = 155.2981 + 0.0076\,X_6 - 0.0144\,X_7 + 1.9703\,X_4 \quad (13)$$

$$X_9 = 374.3789 = 1.5486\,X_5 - 0.1622\,X_8 \quad (14)$$

These models produced multiple correlation coefficients ranging from 0.80 to 0.83 and determining coefficients ranging from 0.64 to 0.70. All models are significant at the 0.01 level. Statistics for these models are given in Tables 18 through 20, Appendix B.

SAMPLE PROBLEMS

Described below are two sample problems for utilizing the volume models developed in this study.

1. On a six-lane urban freeway it is desired to know total three-lane volume in the outbound direction from the central business district from 4:20 p.m. to 4:25 p.m. The median-lane volume for that time period is 135 vehicles. On 24-hour clock basis, 4:25 p.m. is 1625. By substitution in Model 9, we have

$$X_9 = 66.5754 + 2.1405\,X_3 + 0.1086\,X_8$$

$$= 66.5754 + 2.1405\,(135) + 0.1086\,(1625)$$

$$\div\ 399 \text{ (total three-lane volume from 4:20-4:25 p.m.)}$$

2. Find the total three-lane volume during the morning peak period. The following information is known: center-lane volume, 120 vehicles; distance to nearest upstream entrance ramp, 800 ft; distance to nearest downstream exit ramp, 650 ft. Using model 13:

$$X_9 = 155.2981 + 0.0076\,X_6 - 0.0144\,X_7 + 1.9703\,X_4$$

$$= 155.2981 + 0.0076\,(800) - 0.0144\,(650) + 1.9703\,(120)$$

$$\doteqdot 388 \text{ (total three-lane volume for the morning peak during a 5-min period)}$$

Other models can be used to estimate volumes in a similar manner.

CONCLUSIONS

1. The operating characteristics of panel and pickup trucks are not significantly different from passenger vehicles.
2. A definite peak period existed within the peak hour for both morning and afternoon peaks. The average duration of peak period for afternoon peak was higher (20.4 min) than the corresponding morning peak period (17.2 min).
3. Generally the center lane carried a higher percentage of traffic than either the median or shoulder lane during the morning peak, whereas the median and center lane were approximately equally utilized during the afternoon peak.
4. For the afternoon peak, single-unit and combination trucks were primary users of the shoulder lane. The existence of the auxiliary lane caused considerably less use of the shoulder lane by combination trucks than by single-unit trucks.
5. The average percentage of center-lane usage by all vehicles was slightly higher during the morning peak than the afternoon peak.
6. For most of the study section, the truck traffic stayed in the shoulder lane as required by law. Even in the 4-lane sections, the shoulder lane carried a higher percentage of truck traffic than the auxiliary lane.
7. Through the use of simple correlation procedures it was found that there was a high correlation among the 5-min volumes of median, center, and shoulder lanes and total three lanes. The correlation was higher for the afternoon peak than the morning peak.
8. The prediction of individual lane volume as well as total three-lane volume can be accomplished through the use of linear regression models. In general the models for the afternoon peak had a higher explained variance (R^2) than the corresponding morning peak models. This is probably due to the fact that the afternoon rush period carries a considerably higher volume of traffic, has a longer duration and has a greater number of sustained peaks than the morning peak period.
9. Through the use of the appropriate regression model, total freeway volumes can be estimated at a given location with information from a detector in only one lane. Median- and center-lane models give better estimates of total volumes than shoulder-lane models.

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Appendix A

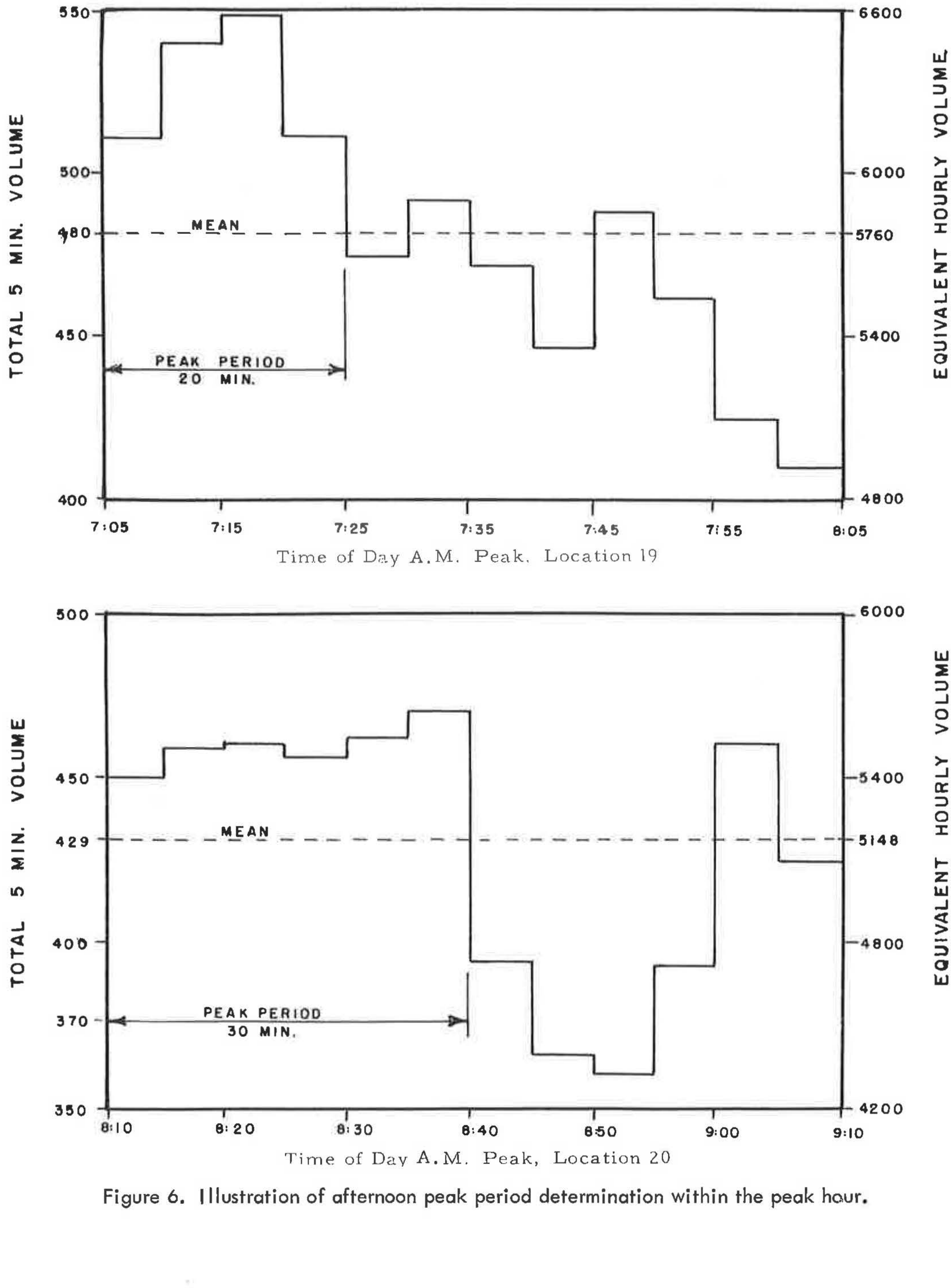
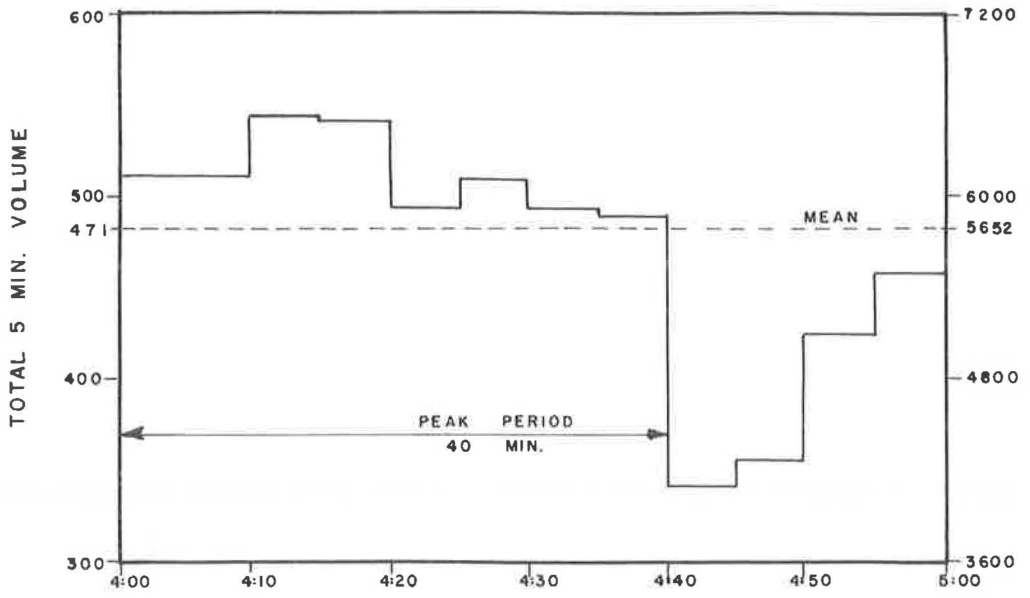
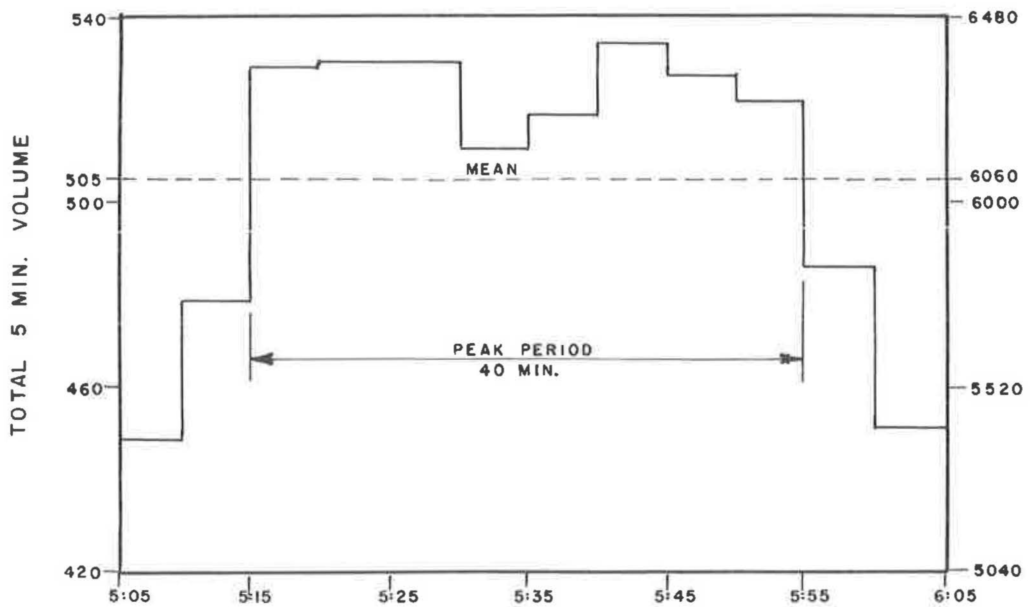


Figure 6. Illustration of afternoon peak period determination within the peak hour.



Time of Day P.M. Peak, Location 3



Time of Day P.M. Peak, Location 4

Figure 7. Illustration of morning peak period determination within the peak hour.

Appendix B

TABLE 7

MEDIAN LANE VOLUME RELATED TO TOTAL THREE LANE VOLUME, DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, AND DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP-AFTERNOON PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.3498×10^4	51.5
Regression	4	0.3175×10^5	7940
Total	72	0.3524×10^5	

$F = 7940/51.5 = 154 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_3 = -0.4216 + 0.0022\, X_6 - 0.0044\, X_7 + 0.3576\, X_9 \quad (1)$$

X_3 = Median lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.), X_7 = Distance to nearest downstream exit ramp (ft.), X_9 = Total three lane volume per five minutes

Multiple Correlation Coefficient R = 0.95

Determining Coefficient, R^2 = 0.90

Standard Error of Predicted Median Lane Volume = 7.17

	X_6	X_7	X_9
Beta Weight	0.143	-0.235	0.817
Partial Correlation Coefficient	0.390	-0.499	0.903
R^2 Delete	0.883	0.868	0.464

TABLE 8

MEDIAN LANE VOLUME RELATED TO CENTER LANE VOLUME, SHOULDER LANE VOLUME, DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP AND DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP—AFTERNOON PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.7778×10^5	1142
Regression	4	0.2747×10^5	6870
Total	72	0.3524×10^5	

$F = 6870/1142 = 6.01 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_3 = 35.3481 + 0.5273\,X_4 + 0.3634\,X_5 + 0.0031\,X_6 - 0.0081\,X_7$$

X_3 = Median lane volume per five minutes, X_4 = Center lane volume per five minutes, X_5 = Shoulder lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.), X_7 = Distance to nearest downstream exit ramp (ft.)

Multiple Correlation Coefficient, R = 0.88

Determining Coefficient, $R^2 = 0.78$

Standard Error of Predicted Median Lane Volume = 10.77

	X_4	X_5	X_6	X_7
Beta Weight	0.068	0.072	0.062	0.068
Partial Correlation Coefficient	0.629	0.432	0.373	0.612
R^2 Delete	0.635	0.729	0.744	0.645

TABLE 9

MEDIAN LANE VOLUME RELATED TO DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP, DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, AND TOTAL THREE LANE VOLUME—MORNING PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.8510×10^4	125.2
Regression	4	0.1542×10^5	3860
Total	72	0.2393×10^5	

$F = 3860/125.2 = 30.8 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_3 = 0.4349 + 0.0055\, X_6 - 0.0048\, X_7 + 0.3289\, X_9$$

X_3 = Median lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.), X_7 = Distance to nearest dounstream exit ramp (ft.), X_9 = Total three lane volume per five minutes

Multiple Correlation Coefficient, $R = 0.80$

Determining Coefficient, $R^2 = 0.64$

Standard Error of Predicted Median Lane Volume = 11.19

	X_6	X_7	X_9
Beta Weight	0.379	-0.414	0.723
Partial Correlation Coefficient	0.469	-0.504	0.769
R^2 delete	0.544	0.523	0.131

TABLE 10

CENTER LANE VOLUME RELATED TO DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, AND TOTAL THREE LANE VOLUME—AFTERNOON PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.6001×10^4	88.2
Regression	4	0.1943×10^5	4870
Total	72	0.2543×10^5	

$F = 4870/88.2 = 55.2 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_4 = -17.3155 + 0.0031\, X_7 + 0.3629\, X_9$$

X_4 = Center lane volume per five minutes, X_7 = Distance to nearest downstream exit ramp (ft.), X_9 = Total three lane volume per five minutes

Multiple Correlation Coefficient, R = 0.87

Determining Coefficient, $R^2 = 0.76$

Standard Error of Predicted Center Lane Volume = 9.33

	X_7	X_9
Beta Weight	0.198	0.976
Partial Correlation Coefficient	0.314	0.852
R^2 delete	0.738	0.140

TABLE 11

CENTER LANE VOLUME RELATED TO DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP, DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, AND TOTAL THREE LANE VOLUME—MORNING PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.6466×10^4	95.0
Regression	4	0.1958×10^5	4900
Total	72	0.2604×10^5	

$F = 4900/95 = 51.6 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_4 = 2.5122 - 0.0045\,X_6 + 0.0073\,X_7 + 0.3404\,X_9$$

X_4 = Center lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.), X_7 = Distance to nearest downstream exit ramp (ft.), X_9 = Total three lane volume per five minutes

Multiple Correlation Coefficient, R = 0.87

Determining Coefficient, $R^2 = 0.75$

Standard Error of Predicted Center Lane Volume = 9.75

	X_6	X_7	X_9
Beta Weight	-0.298	0.594	0.717
Partial Correlation Coefficient	-0.448	0.708	0.819
R^2 delete	0.689	0.502	0.246

TABLE 12

SHOULDER LANE VOLUME RELATED TO DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP AND TOTAL THREE LANE VOLUME—AFTERNOON PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.6824×10^4	100.4
Regression	4	0.1413×10^5	3540
Total	72	0.2096×10^5	

$F = 3540/100.4 = 35.2 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_5 = 24.5834 - 0.0014\, X_6 + 0.2669\, X_9$$

X_5 = Shoulder lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.), X_9 = Total three lane volume per five minutes

Multiple Correlation Coefficient, R = 0.82

Determining Coefficient, R^2 = 0.67

Standard Error of Predicted Shoulder Lane Volume = 9.94

	X_6	X_9
Beta Weight	-0.120	0.790
Partial Correlation Coefficient	-0.201	0.806
R^2 delete	0.661	0.072

TABLE 13

SHOULDER LANE VOLUME RELATED TO DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, AND TOTAL THREE LANE VOLUME—MORNING PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.7942×10^4	117
Regression	4	0.1488×10^5	3720
Total	72	0.2282×10^5	

$F = 3720/117 = 31.8 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_5 = -4.9233 - 0.0028 X_7 + 0.3333 X_9$$

X_5 = Shoulder lane volume per five minutes, X_7= Distance to nearest downstream exit ramp (ft.), X_9 = Total three lane volume per five minutes

Multiple Correlation Coefficient, R = 0.81

Determining Coefficient, $R^2 = 0.65$

Standard Error of Predicted Shoulder Lane Volume = 10.73

	X_7	X_9
Beta Weight	-0.248	0.752
Partial Correlation Coefficient	-0.389	0.785
R^2 delete	0.591	0.093

TABLE 14

TOTAL THREE LANE VOLUME RELATED TO MEDIAN LANE VOLUME AND DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP—AFTERNOON PEAK

Source of Variation	Degrees Freedom	Sum of Squares	Mean Square
Error	68	0.2364×10^5	348
Regression	4	0.1602×10^6	40,100
Total	72	0.1838×10^6	

$F = 40,100/348 = 115 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_9 = 133.9218 + 2.097\, X_3 - 0.0036\, X_6$$

X_9 = Total three lane volume per five minutes, X_3 = Median lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.)

Multiple Correlation Coefficient, R = 0.93

Determining Coefficient, R^2 = 0.87

Standard Error of Predicted Total Three Lane Volume = 18.51

	X_3	X_6
Beta Weight	0.918	-0.102
Partial Correlation Coefficient	0.931	-0.272
R^2 delete	0.035	0.861

TABLE 15

TOTAL THREE LANE VOLUME RELATED TO MEDIAN LANE VOLUME, AND TIME OF DAY—AFTERNOON PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.2267×10^5	334
Regression	4	0.1611×10^6	40,300
Total	72	0.1831×10^6	

$F = 40,300/334 = 120.5 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_9 = -66.5754 + 2.1405\ X_3 + 0.1086\ X_8$$

X_9 = Total three lane volume per five minutes, X_3 = Median lane volume per five minutes, X_8 = Time of day to nearest five minutes on 24-hour clock basis

Multiple Correlation Coefficient, $R = 0.94$

Determining Coefficient, $R^2 = 0.88$

Standard Error of Predicted Total Three Lane Volume = 18.12

	X_3	X_8
Beta Weight	0.937	0.125
Partial Correlation Coefficient	0.936	0.335
R^2 delete	0.003	0.861

TABLE 16

TOTAL THREE LANE VOLUME RELATED TO NUMBER OF SINGLE UNIT VEHICLES IN SHOULDER LANE, DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, AND CENTER LANE VOLUME–AFTERNOON PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean. Square
Error	68	0.3029×10^5	446
Regression	4	0.1535×10^6	38,400
Total	72	0.1838×10^6	

$F = 38,400/446 = 86 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_9 = 180.0925 + 2.2271\ X_2 - 0.0138\ X_7 + 1.9087\ X_4$$

X_9 = Total Three lane volume per five minutes, X_2 = Number of single unit vehicles in shoulder lane per five minutes, X_7 = Distance to nearest downstream exit ramp (ft.), X_4 = Center lane volume per five minutes

Multiple Correlation Coefficient, $R = 0.91$

Determining Coefficient, $R^2 = 0.84$

Standard Error of Predicted Total Three Lane Volume = 21.11

	X_2	X_7	X_4
Beta Weight	0.127	-0.325	0.710
Partial Correlation Coefficient	0.290	-0.594	0.844
R^2 delete	0.820	0.746	0.428

TABLE 17

TOTAL THREE LANE VOLUME RELATED TO SHOULDER LANE VOLUME AND DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP—AFTERNOON PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.5223×10^5	768
Regression	4	0.1316×10^6	32,900
Total	72	0.1838×10^6	

$F = 32,900/768 = 42.8 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_9 = 190.6431 + 2.0403\, X_5 - 0.0112\, X_7$$

X_9 = Total three lane volume per five minutes, X_5 = Shoulder lane volume per five minutes, X_7 = Distance to nearest downstream exit ramp (ft.)

Multiple Correlation Coefficient, R = 0.85

Determining Coefficient, $R^2 = 0.72$

Standard Error of Predicted Total Three Lane Volume = 27.51

	X_5	X_7
Beta Weight	0.689	-0.266
Partial Correlation Coefficient	0.753	-0.404
R^2 delete	0.344	0.661

TABLE 18

TOTAL THREE LANE VOLUME RELATED TO DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP, DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, MEDIAN LANE VOLUME, AND TIME OF DAY—MORNING PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.3511×10^5	516
Regression	4	0.8036×10^5	20,090
Total	72	0.1155×10^6	

$F = 20,090/516 = 39 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_9 = 411.7560 - 0.0118\, X_6 + 0.0070\, X_7 + 1.6012\, X_3 - 0.2311\, X_8$$

X_9 = Total three lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.), X_7 = Distance to nearest downstream exit ramp (ft.), X_3 = Median lane volume per five minutes, X_8 = Time of day to nearest five minutes on 24-hour clock basis.

Multiple Correlation Coefficient, R = 0.83

Determining Coefficient, R^2 = .70

Standard Error of Predicted Total Three Lane Volume = 22.89

	X_6	X_7	X_3	X_8
Beta Weight	-0.373	0.273	0.729	-0.330
Partial Correlation Coefficient	-0.481	0.359	0.766	-0.495
R^2 delete	0.604	0.651	0.264	0.597

TABLE 19

TOTAL THREE LANE VOLUME RELATED TO DISTANCE TO NEAREST UPSTREAM ENTRANCE RAMP, DISTANCE TO NEAREST DOWNSTREAM EXIT RAMP, AND CENTER LANE VOLUME—MORNING PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.3743×10^5	551
Regression	4	0.7805×10^5	19,550
Total	72	0.1155×10^6	

$F = 19,550/551 = 35.5 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_9 = 155.2981 + 0.0076\, X_6 - 0.0144\, X_7 + 1.9703\, X_4$$

X_9 = Total three lane volume per five minutes, X_6 = Distance to nearest upstream entrance ramp (ft.), X_7 = Distance to nearest downstream exit ramp (ft.), X_4 = Center lane volume per five minutes

Multiple Correlation Coefficient, R = 0.82

Determining Coefficient, R^2 = 0.68

Standard Error of Predicted Total Three Lane Volume = 23.46

	X_6	X_7	X_4
Beta Weight	0.240	-0.560	0.936
Partial Correlation Coefficient	0.316	-0.584	0.819
R^2 delete	0.640	0.508	0.016

TABLE 20

TOTAL THREE LANE VOLUME RELATED TO SHOULDER LANE VOLUME, AND TIME OF DAY—MORNING PEAK

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Error	68	0.4181×10^5	616
Regression	4	0.7377×10^5	18,450
Total	72	0.1155×10^6	

$F = 18{,}450/616 = 30.0 > F_{0.01}$ $\therefore$ Highly Significant

Predictor Model:

$$X_9 = 374.3789 + 1.5486\ X_5 - 0.1622\ X_8$$

X_9 = Total three lane volume per five minutes, X_5 = Shoulder lane volume, X_8 = Time of day to nearest five minutes on 24-hour clock basis.

Multiple Correlation Coefficient, R = 0.80

Determining Coefficient, $R^2 = 0.64$

	X_5	X_8
Beta Weight	0.688	-0.231
Partial Correlation Coefficient	0.732	-0.339
R^2 delete	0.221	0.591

Operational Effects of Overall Geometrics on Highway Safety

W. R. BELLIS, Director of Research and Evaluation, New Jersey State Highway Department, Trenton

This study concludes that accidents, injuries and fatalities can be significantly reduced on state highways by the conversion of these highways to freeways. The author contends that this is the only sure way of reducing accident experience. A system of freeways in addition to the state highway system will not reduce accident experience because the accident rate will continue to increase on the non-converted state highways and the overall result of accidents, injuries and fatalities will not be less than they were before the construction of freeways.

•THE MOTOR vehicle makes it possible for man to travel in a manner which is satisfactory both for convenience and economics. Practically everyone in the United States does most of his traveling by motor vehicle, and thus traffic volumes continue to increase on our highways, roads and streets at rates about equal to the increase of the last 40 years. The motor vehicle must be credited with producing our present economic status, but along with this are the undesirable features of accidents, injuries and fatalities.

THE PROBLEM

Although strong efforts have been made to reduce accidents, the results are not satisfactory. However, the accident rate can be reduced significantly. It has been said that the automobile saves more lives than the lives that are lost on our highways. It may be said that each accident was unnecessary. Nevertheless, accidents occur and will continue to occur. It was hoped that accidents, injuries and fatalities could be reduced satisfactorily by present methods, but they have been proved ineffective.

Reviewing these efforts one can see areas for improvement, or areas where the results would have been more disastrous if some improvement had not been made. It is also evident that safety can be a reality if one is willing to spend money and effort.

Most drivers feel that the other driver is the one who causes accidents. When mistakes are made while traveling, a disastrous accident may result. How does one prevent these mishaps? Perhaps drivers need more safety education or more police enforcement, or is better engineering the answer? All of these are essential. In the past much emphasis was placed on enforcement and education. It is possible that enforcement has been exercised to the maximum desirable degree. Too much enforcement may result in either a loss of motor vehicle advantage or an increase in accidents. Increases in accidents have occurred when enforcement was intensified.

Highway education is limited. The greatest educator is experience itself and people in this country are exposed to automotive experience at an early age. Yet, the most experienced drivers are not necessarily the better drivers.

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 45th Annual Meeting.

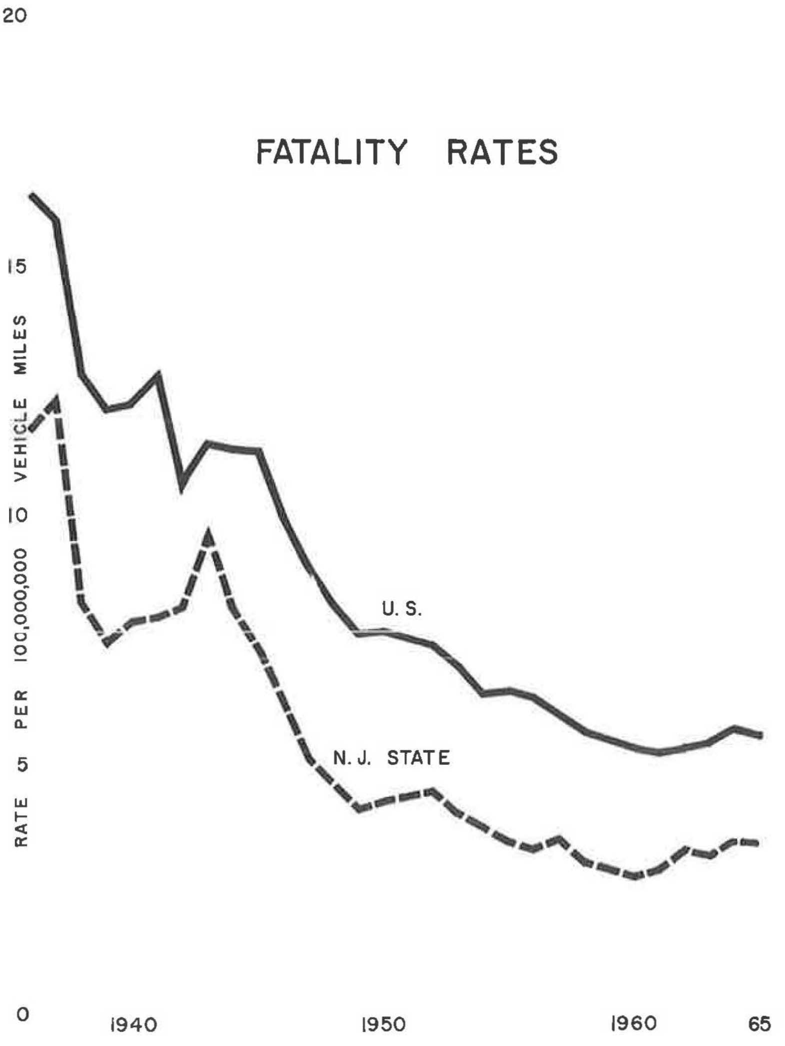


Figure 1.

SCOPE

Engineering improvements are necessary, although in the past they have not resulted in the expected safety. A comparison of state highway systems, where much money has been spent for highway safety, with other roads, where money has not been spent for safety, illustrates that safety has not really been attained by these expenditures. Only on highway systems where freeway design has been utilized 100 percent was there an indication of real safety results.

PROCEDURE

This paper presents a procedure which will reduce overall accidents, injuries and fatalities. The basic data are principally illustrated by experiences in New Jersey, but it is quite certain that such experience has also been found in other states. Most highway policies are quite similar, and it can be expected that what can be shown for New Jersey would also apply to other states and the country as a whole.

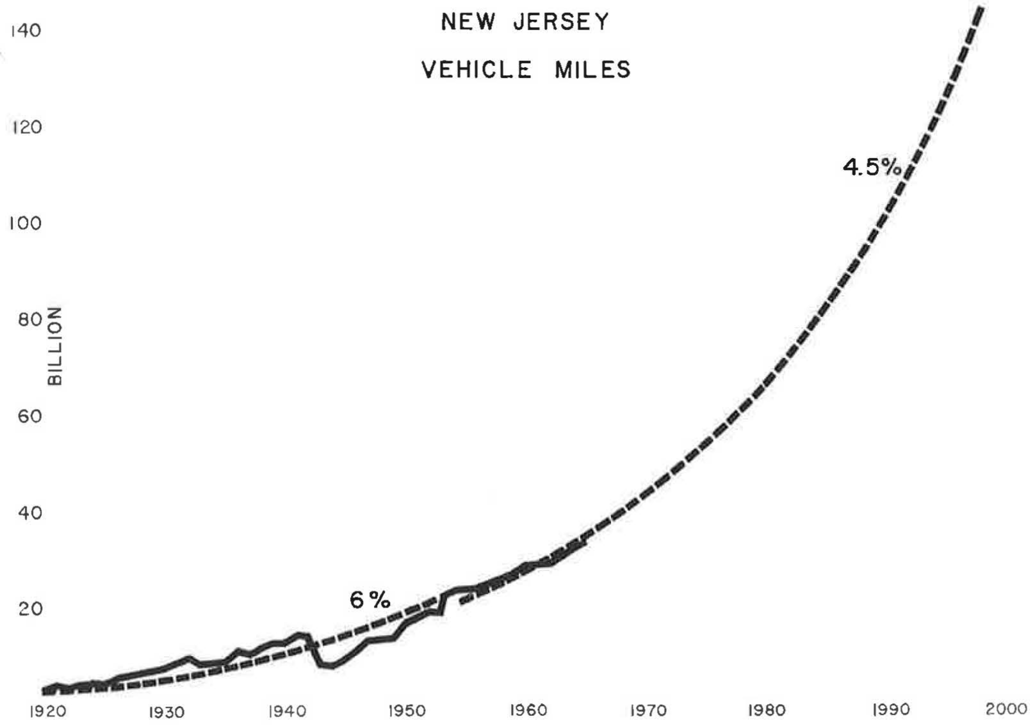
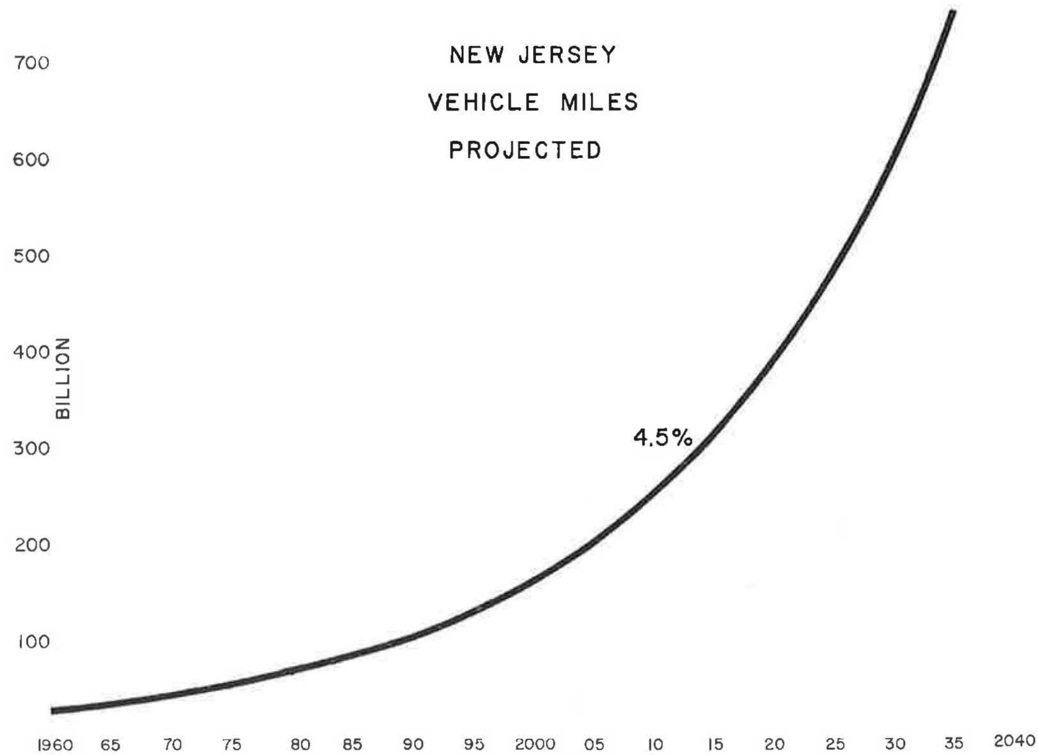


Figure 2.



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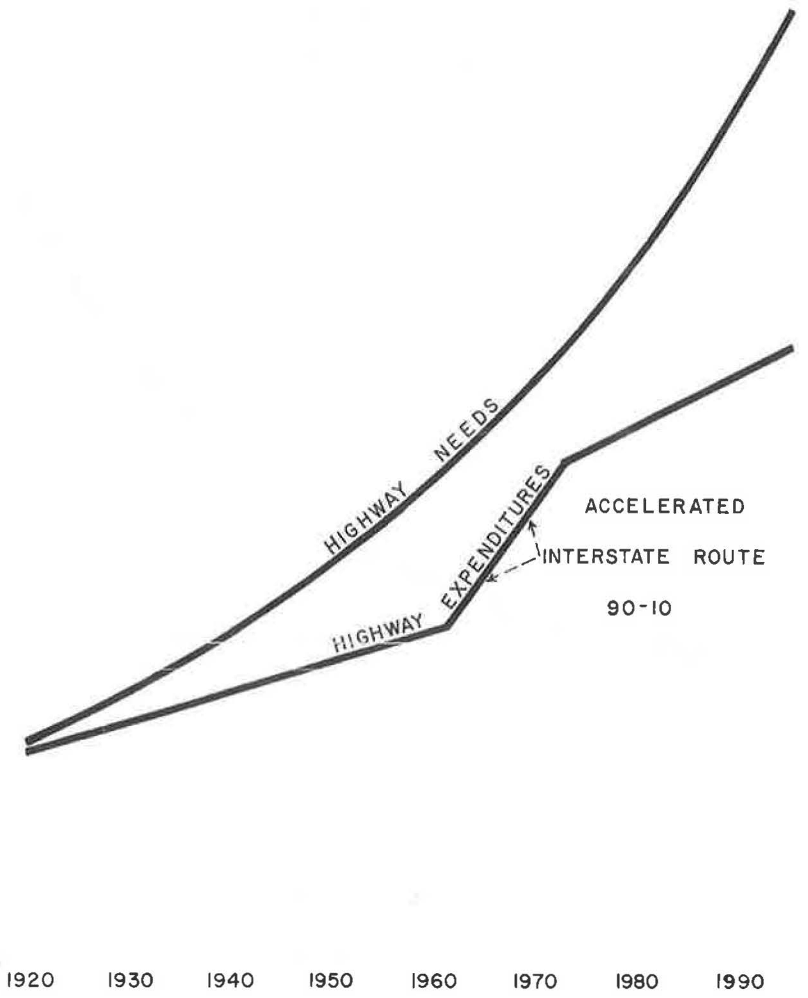


Figure 4.

Facts

Figure 1 shows a similarity between New Jersey fatality rates and the rates for the United States from 1936 through 1965. The fatality rates realized in New Jersey have later been realized for the average of the country. This seems logical since New Jersey has the highest density of traffic and therefore experiences traffic problems in advance of other places in the United States.

Figure 2 shows traffic volume growth in New Jersey from 1920 through 1965. This growth has been at a rate of 6 percent compounded yearly, and is expected to continue at a rate of $4\frac{1}{2}$ percent compounded yearly. Growth curves such as this, when projected into the future, seem quite shocking, but this should not be so. In plotting such a curve, the first impression is that in the projected future the curve seems to be going straight up, but this simply is a matter of the scale which is selected to illustrate present conditions. Imagine instead of being in the 1960's, it is 40 yr later and plot this same curve, assuming that from 1960 to the year 2000 is past history. By

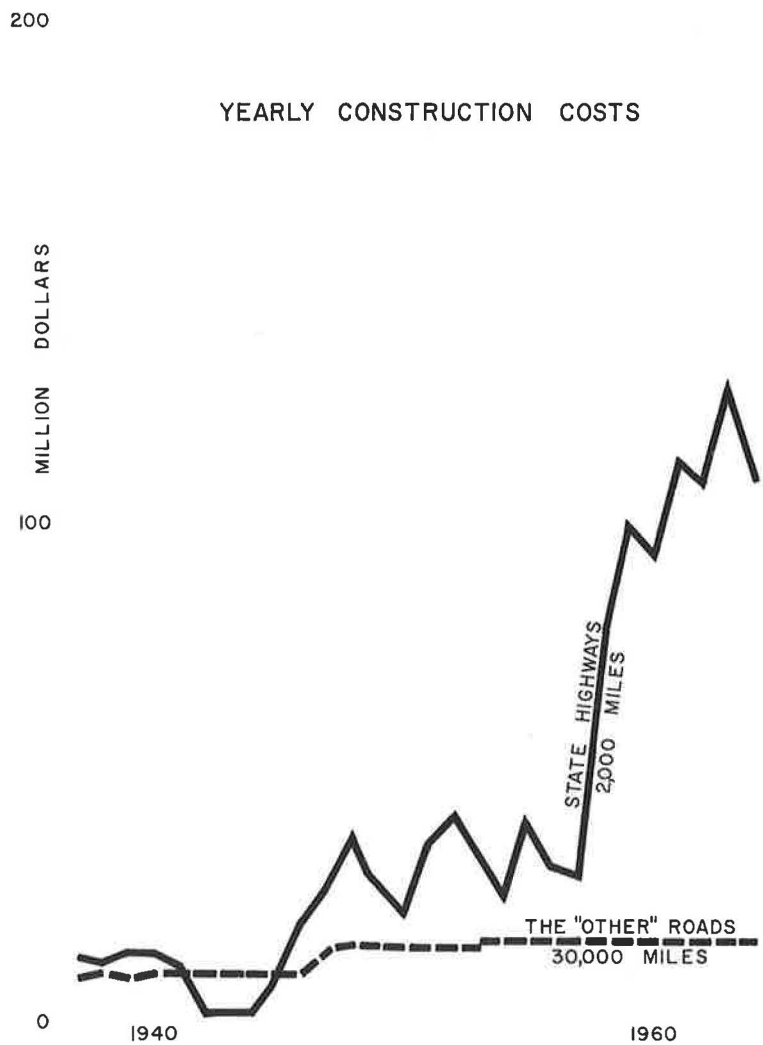


Figure 5.

continuing that curve as in the former figure, one would get a curve (Fig. 3), which can be superimposed on the other curve.

Figure 2 shows a growth of 30-billion car-miles between 1900 and 1963, and it can be expected that there will be another 30-billion car-miles added to our traffic volumes during the next 15 yr. An additional 30-billion car-miles will be added in the following 9 yr, taking us up to the year 1987. As this trend continues, it indicates that in less than 100 yr there will be 30-billion car-miles added each year instead of over a period of 63 yr. This type of illustration emphasizes the need for a readiness to cope with the accelerated pace in our future way of living.

It will be necessary to build the same amount of roads in a few years that used to be built over a period of many years. Expansion problems are increasing faster and faster. It is difficult to accept a 100-yr projection, but all of the barriers will have to be hurdled by future generations.

Certainly the year 1980 will have more traffic than there was in 1960. Also, it is logical to assume that the year 2000 will have more traffic than the year 1980, and so on into the future. We hear of "overpopulation" and "where can we build more highways, the whole state will be paved solid." Although New Jersey is the most densely populated state in the country, at least 83 percent of our land area is used for farming and woodlots. Also 50 percent of the municipalities in New Jersey have population densities of less than two people per acre.

Monies for highway improvements have never kept pace with highway needs. Figure 4 shows where highway expenditures from 1920 to 1960 are straight line increases. The same amount of money is provided each year for highway improvements, whereas highway needs are growing at a compounded rate per year which produced an upward curve diverging from the highway expenditure curve. The vertical distance between highway needs and the highway expenditure curve (Fig. 4) illustrates increased congestion and increased accident experience year after year. When this lag or deficiency accumulates to an intolerable condition, then monies are provided at a higher rate than in former years. This accelerated rate is illustrated in Figure 4 from 1960 to 1972 in which the Interstate program is such an accelerated program. This is providing improvements possibly at a faster rate than highway needs are currently increasing, but it still may not match all of our highway needs. At the end of this accelerated program, it is logical to assume that improvement programs will again continue on equal amounts of money each year, possibly more money each year than had been applied from 1920 to 1960, but still at a rate which allows the gap between highway needs and highway expenditures to widen rapidly.

It is hard to imagine this growth on a compound rate rather than on a straight line rate, but this must be faced in order to overcome the needs. This is not a hopeless job, but efforts must be made to educate the public to this condition.

Many engineers in responsible positions who have done admirable jobs in providing good highways refer to the wonderful improvements that have been made through the years and are content to rest with these achievements, but the problem continues to grow. The younger engineers do not have the picture of the past as do the older engineers, and therefore are not frightened by the contrast between the future and the past.

Figure 5 shows the money spent for state highway construction yearly from 1937 to 1964 and also the state aid to counties and municipalities. The expenditures for less than 2,000 mi of state highways are enormous compared to the expenditures for 30,000 mi of other roads.

The proposed solution to significantly reduce our accident experience is to convert the state highway system to freeways. This is illustrated by classifying the roads in New Jersey into three systems: (a) toll roads, which includes the New Jersey Turnpike and the Garden State Parkway; (b) the New Jersey State Highway System; and (c) all other roads. The toll-road system has been in use for over 10 yr and carries a significant volume of traffic, about 10 percent of the total car-miles traveled in New Jersey. Figure 6 shows the increasing vehicle-miles traveled on each of the three systems.

Figure 7 shows the accident rates on each of the systems. The accident rate on the toll-road system is about 20 percent of the accident rate on the state highway system. The accident experience on many other freeways throughout the country is about the same as shown for New Jersey toll roads, and the accident experience on state highway systems in other states is relatively similar to that of the New Jersey state highways, although the accident experience on the New Jersey state highway system is lower than on most other state highway systems. The accident rate on the toll roads has not been increasing. It is relatively constant, whereas the state highway system accident rate has been increasing, and the accident rate on the other roads has been increasing still faster than on the state highway system.

The injury rate (Fig. 8) indicates that the toll-road injury rate does not increase. It remains constant, whereas for the state highway system and other roads it increases rapidly. The injury rate on the state highway system is no better than the injury rate on other roads. If the state highway system had the same injury rate as the toll roads, there would be more than an 80 percent reduction in injuries.

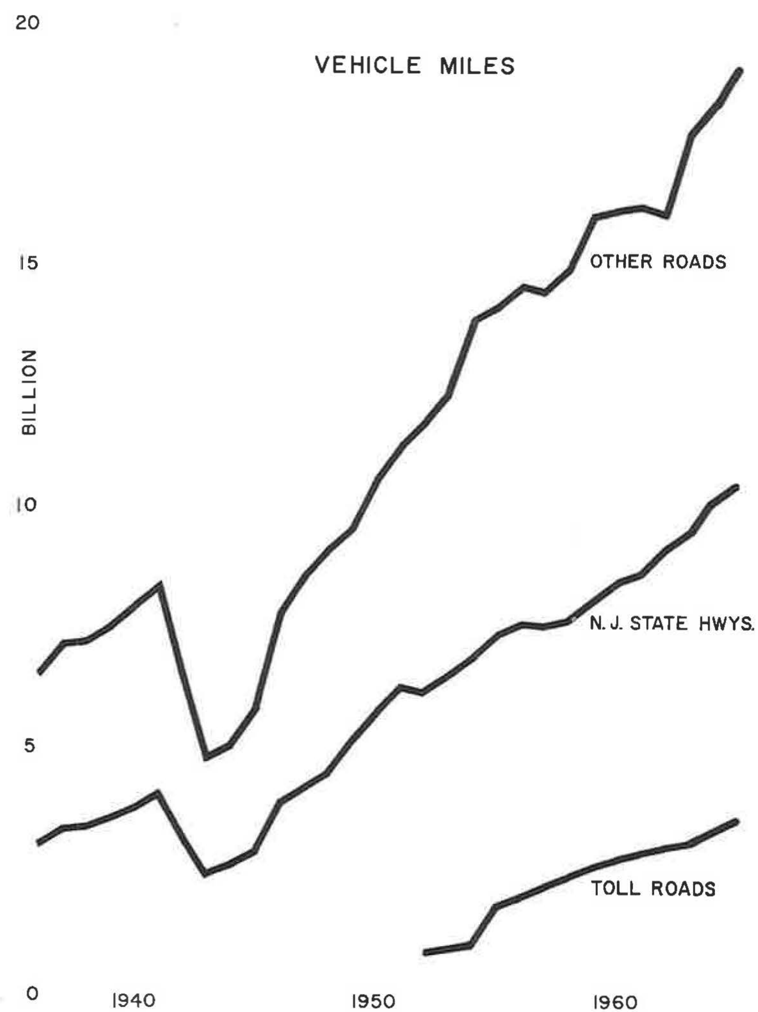
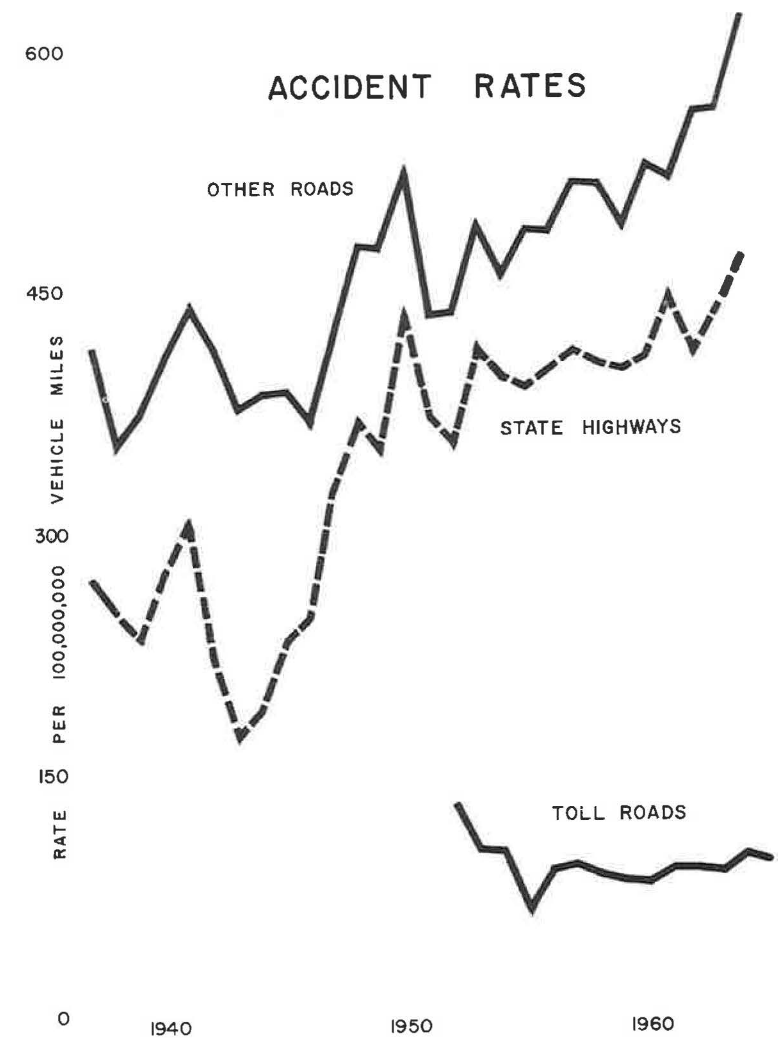


Figure 6.



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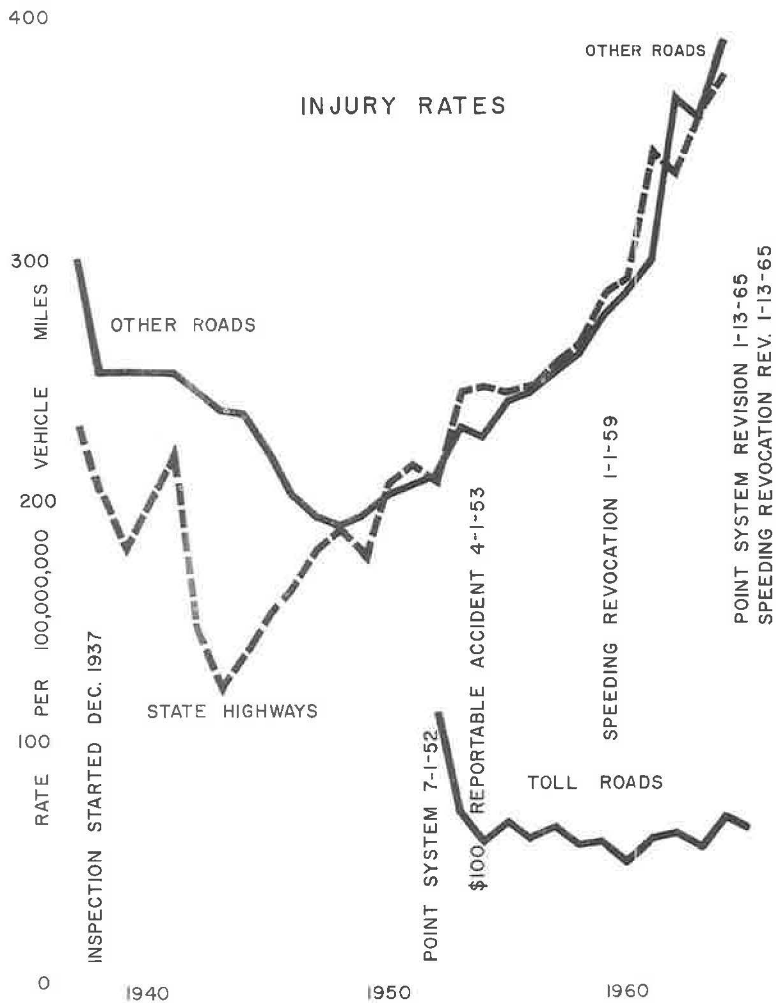


Figure 8.

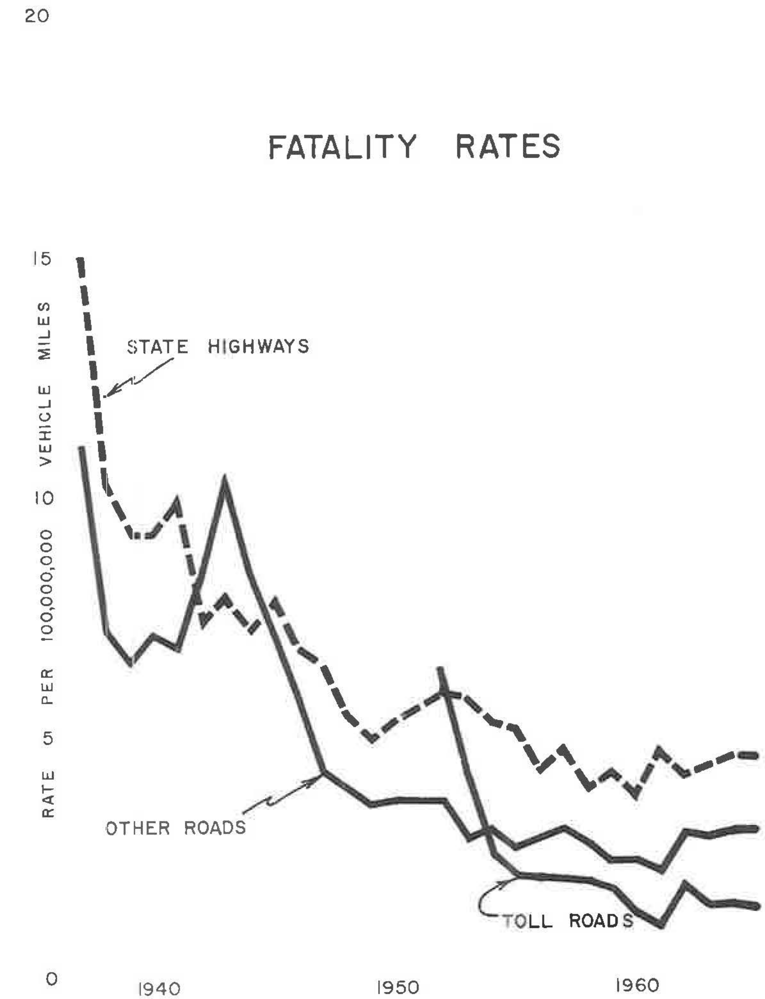


Figure 9.

The fatality rate (Fig. 9) shows a still dimmer picture for the state highway system; here the other roads show up safer than the state highway system. The toll roads are the safest system, having a fatality rate that is about one-third that on the state highway system. This means that if the state highway system had a fatality rate as low as the toll-road system, there would be a 67 percent reduction in fatalities.

ANALYSIS

Some people say the reason accidents are increasing is that speeds are greater. This reasoning is not valid as illustrated by the fact that our highest speed system has the safest accident, injury and fatality record. Some people say that the increase is a result of the popularity of the small car. This is not supported by these illustrations because the small car is used on the toll-road system, probably at about the same rate that it is used on other systems. If the small car was a major factor, the toll-road rates would also show an increase.

One factor stands out as different between the toll roads and the state highway system and also the other road system. It is the fact that marginal friction is reduced to a bare minimum. The toll-road system is fenced to discourage animals and people from crossing the roadways. Pedestrians are prohibited. There are no driveways. There are no roads intersecting except under well designed conditions, whereas the other systems have a maximum of marginal friction. In fact, marginal friction can eliminate the use of a state highway for vehicular traffic. Parades are allowed, block dances, baseball games along the edge of the road, car sales, junk yards, carnivals and such are permitted, wherein the state highway becomes secondary to the marginal friction activities. Generally, marginal friction is not this severe, but it still exists and has been increasing through the years at rates equal to the increase in traffic volume. Years ago a state highway may have been considered equivalent to a freeway because fills and cuts were very severe making it difficult to use the adjacent land. But even this barrier to land development is eliminated as intense marginal friction develops.

In some locations medians have been provided, 100 or 150 ft wide, but these have been used by business which doubles the amount of marginal friction because the driveways are on both left and right of the roadway. On these state highways there is a conflict between the purpose of the road, that is, the use of the road for through traffic, and the use of the road to satisfy purely local interests.

Accidents may be classified in two categories, marginal friction and internal friction. At the present time it appears that 80 percent of accidents and injuries are caused by marginal friction, and 20 percent are caused by internal friction on state highways. Also two-thirds of the fatalities on state highways are a result of marginal friction and one-third the result of internal friction. Indications from the past trends are that, unless major steps are taken, the fatality rate will increase year after year.

Figure 10 shows that the injury rate is increasing at a faster rate than the accident rate. Figure 11 shows the projection of the accident rate and the injury rate on state highways, illustrating that in 1974 the injury rate will be equal to the accident rate, and thereafter the injury rate will exceed the accident rate. Based on Figure 12, which shows that there was one fatality for every 100 accidents in 1963, and on Figure 13, which shows that there were 10 fatalities for every 1, 000 injuries in 1963, it can be shown that in 1975 the state highway fatality rate will be 4. 8 instead of the 4. 5 in 1964. By 1980 the rate may possibly be around 5. 0.

Freeways are recognized as the safest type of design so far experienced. But the mere construction of freeways does not produce safety on the other roads in the state. Freeways themselves would have a low accident experience, but the rates continue to grow on the other road systems.

The New Jersey Turnpike parallels a state highway route throughout its length. This route consists of US 130 from the Delaware Memorial Bridge to the vicinity of New Brunswick, and then US 1 onto the George Washington Bridge.

Figure 14 shows that the accident rate on the parallel route may have been slightly influenced by construction of the New Jersey Turnpike. The accident rate of the two routes combined is also shown. In Figure 15 the injury rate on the parallel route does

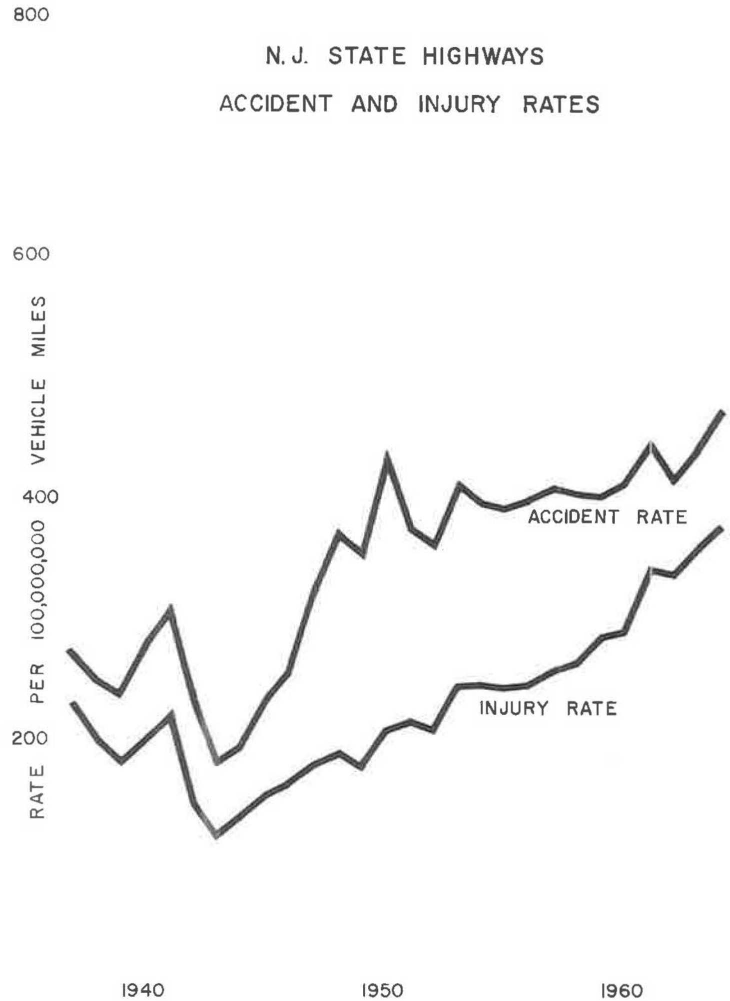


Figure 10.

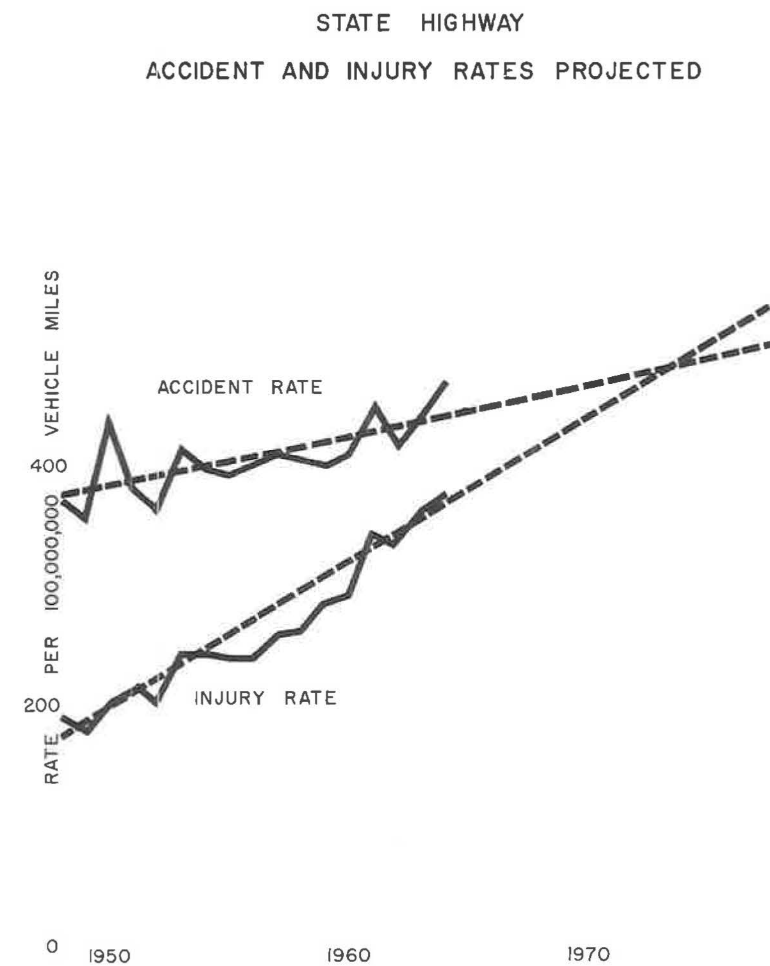
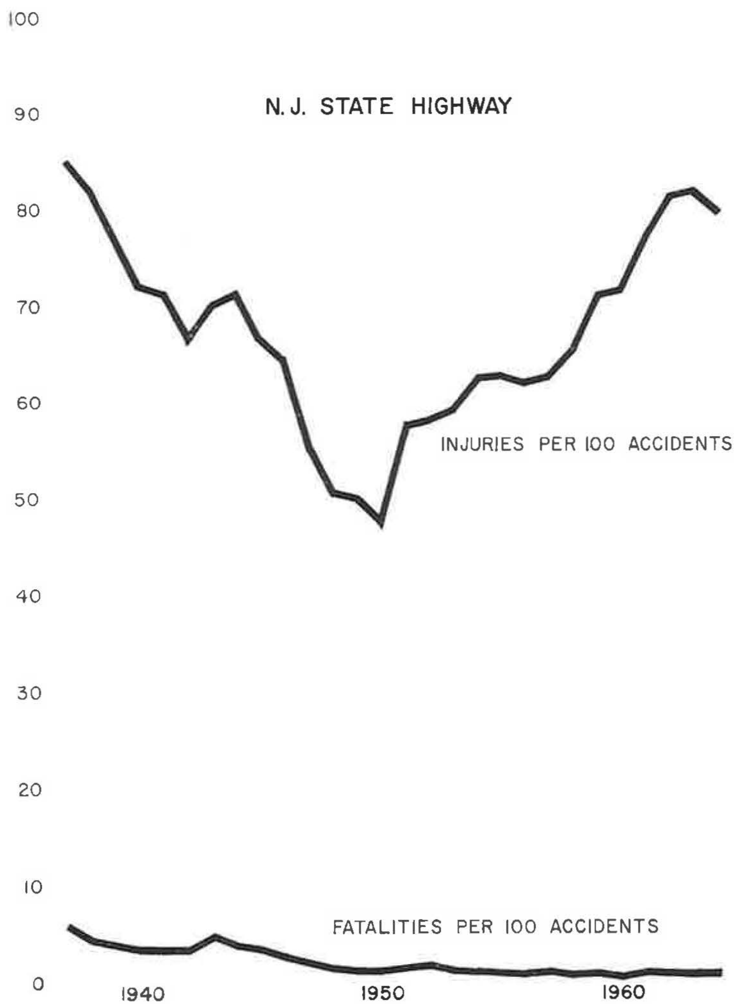


Figure 11.



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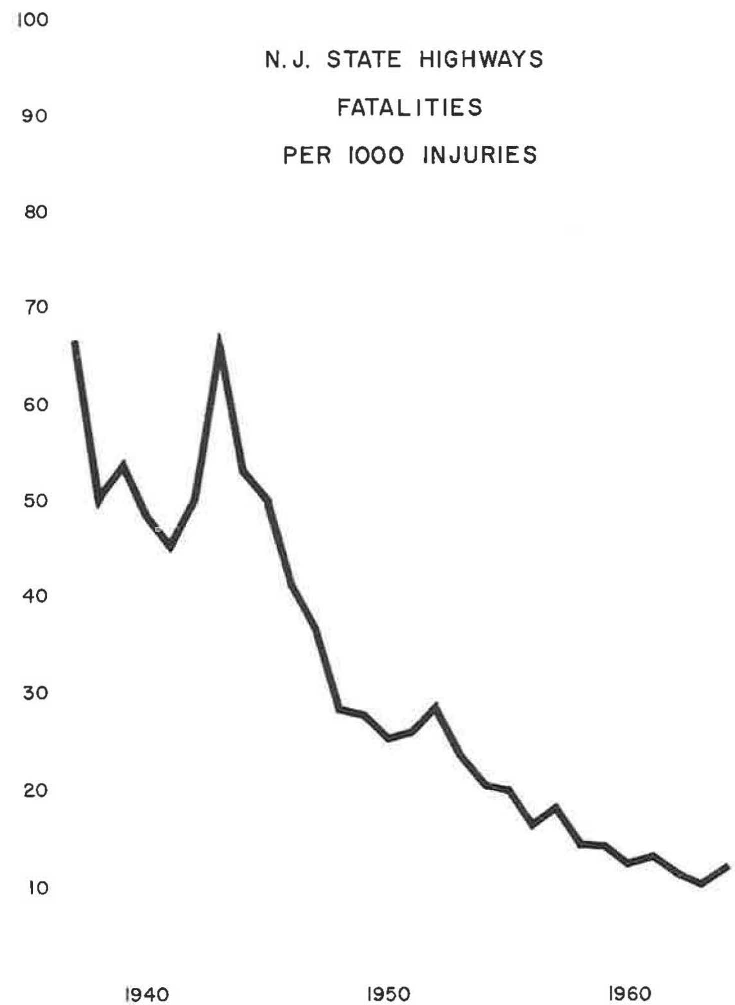

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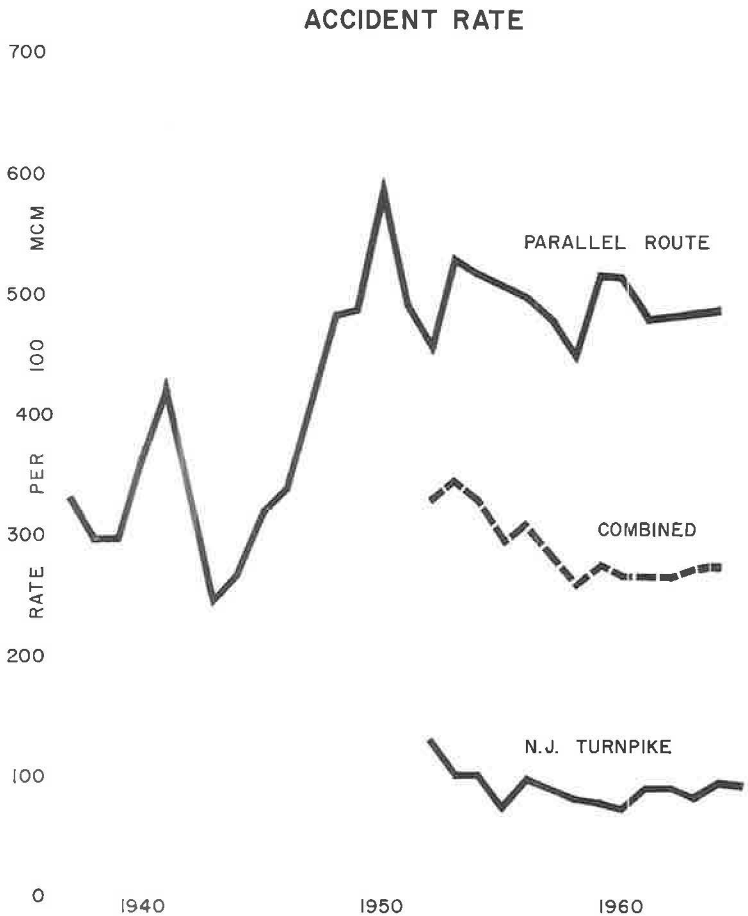

Figure 14.

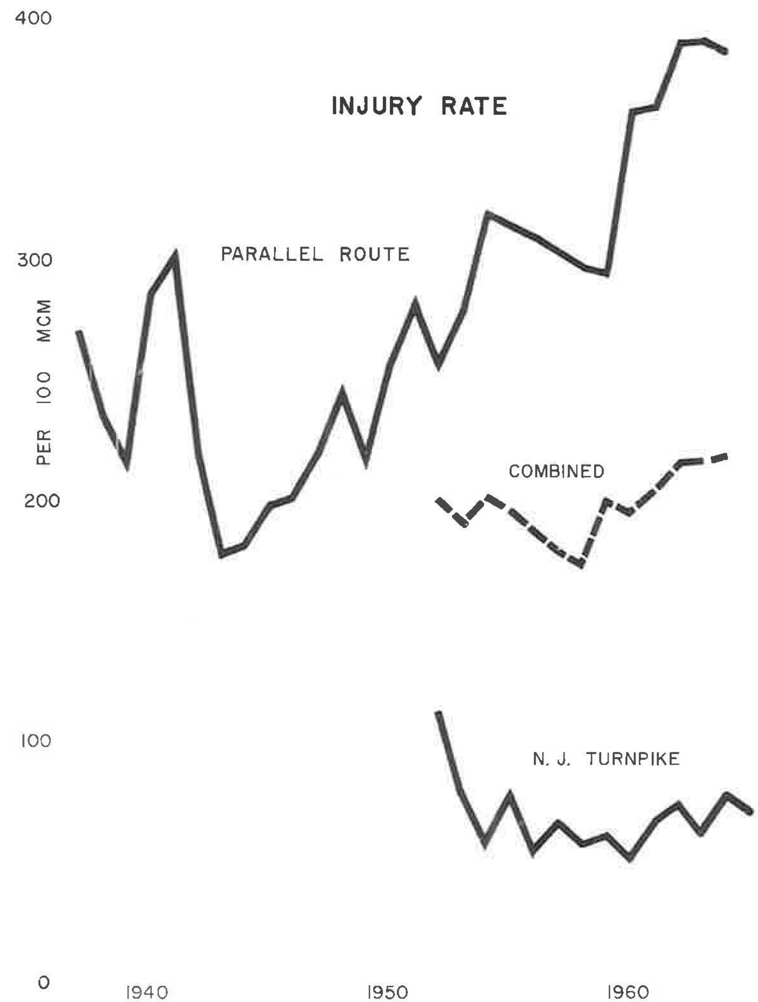

Figure 15.

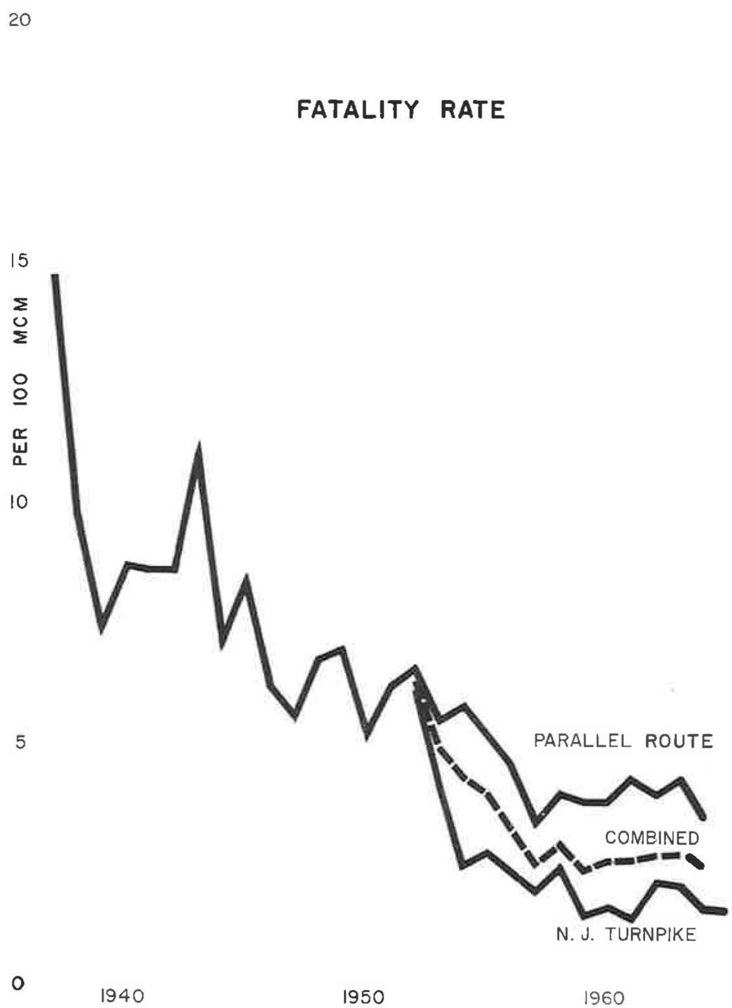

Figure 16.

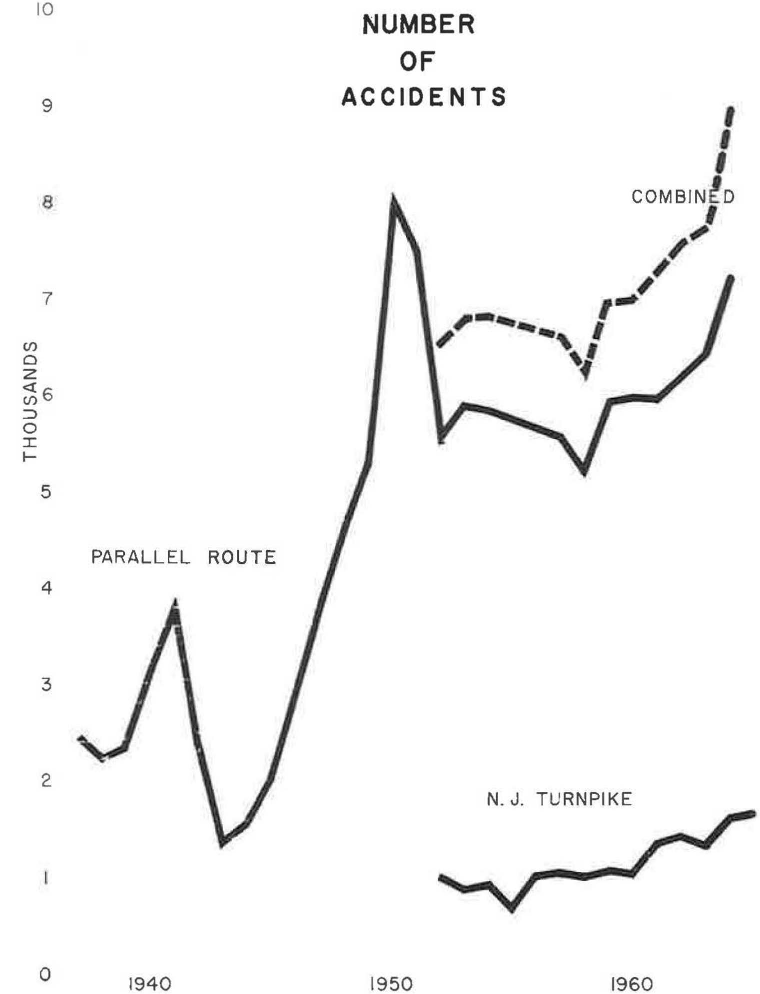

Figure 17.

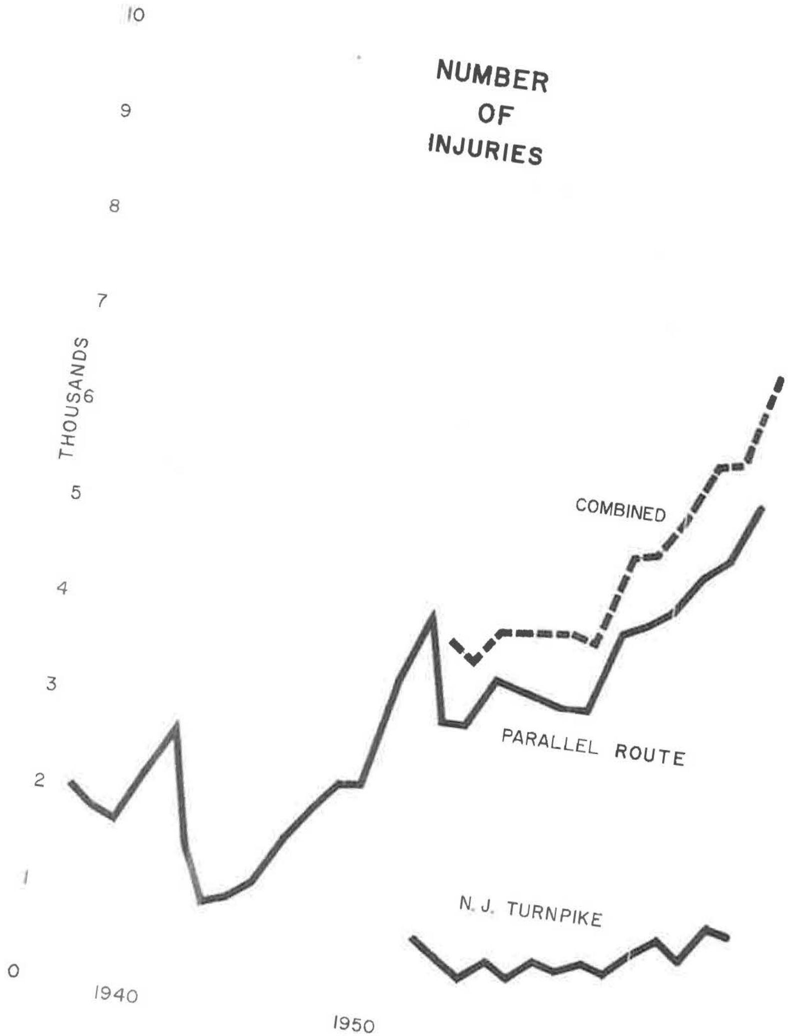

Figure 18.

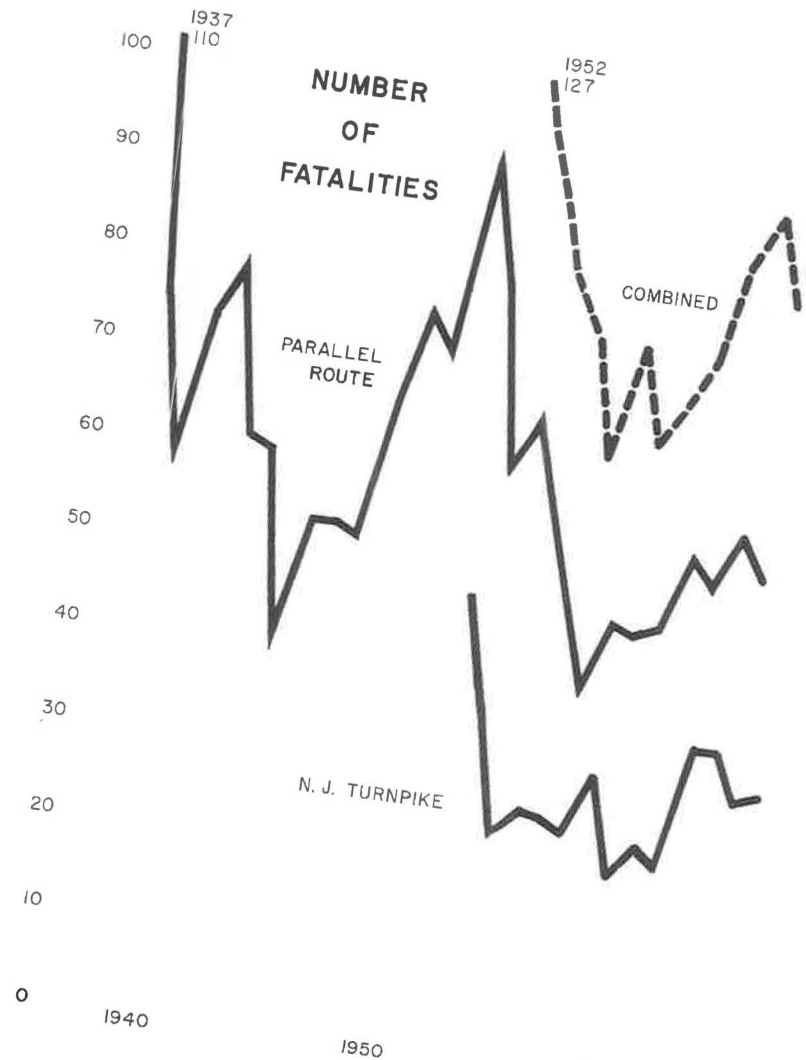

Figure 19.

not show reduction resulting from the turnpike construction. In fact, the injury rate has continued to grow at a fast rate. Figure 16 shows that the fatality rate has increased since 1957. By comparing the rates on the parallel route with that on the state highway system, one realizes the construction of the turnpike has not caused a reduction in these rates on the parallel route because the parallel route rates stayed similar to the rates on the state highway system. The accident, injury and fatality rates are a very good tool for studying accident experience, but the real ambition is to reduce the number of accidents, the number of injuries and the number of fatalities.

The New Jersey Turnpike was under construction in 1951. The number of accidents on the two facilities combined, i. e., the New Jersey Turnpike and the parallel route, has continued to increase. In 1965 the total number of accidents was higher than the total number of accidents in 1950 (Fig. 17). Figure 18 shows that the number of injuries on the combination of the two facilities has continued to rise, and Figure 19 shows that there were more fatalities in 1963 and 1964 on the combined routes than there were in 1950, the year before construction of the turnpike began.

CONCLUSIONS AND RECOMMENDATIONS

The solution proposed here is that, in order to reduce the accident experience, the state highway system should be converted to freeways. A system of freeways which does not displace the state highway system will not produce an overall reduction in accidents, injuries and fatalities, but, if the state highway system is converted to freeways, the reduction expected in accidents and injuries could be around 75 percent and the reduction in fatalities could be around 65 percent.

Conversion to freeways would consist of acquiring right-of-way along the existing state highway system so that the freeway could be located in the existing right-of-way or so close to it that only the short-distance local traffic would remain on the old road. The freeway should be no more than ½ mi from the old facility and parallel to it. In some cases an elevated freeway within the right-of-way would be cheaper than acquiring property frontage rights. In other cases property could be acquired on one side of the existing highway to provide for a service road to accommodate the property frontage. The sections of highways which can be expected to have the greatest number of accidents, injuries and fatalities per mile per year would be the sections to be converted first. Some sections of highway where marginal friction is almost zero could be deferred for many years.

The New Jersey State highway system consists of almost 2, 000 mi. It has been estimated roughly, that to convert this system to freeways, it would cost 4½ billion dollars. If this were divided into equal yearly construction programs over a 20-yr period, it would amount to 225 million dollars a year for a construction program. This is not unreasonable, and can be justified in savings of accidents, injuries and fatalities, and other benefits.

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Interstate Highway Shoulder Use Study In South Dakota

MAX R. CHEESEMAN and WILLIAM T. VOSS, South Dakota Department of Highways, Pierre

The Interstate Highway Shoulder Use Study was initiated in May 1963 to collect factual data on the use of shoulder areas for parking purposes on a controlled-access highway facility. Knowledge of shoulder usage can be utilized by the highway engineer when selecting a roadway design and when planning the size and distribution of safety rest areas along the highway. The information is also of interest to enforcement groups who are responsible for the maintenance and control of traffic on the Interstate Highway System.

•THE HIGHWAY study section was a 19-mi segment of I 90 and I 190 located near Rapid City, South Dakota, which was opened to traffic in October 1960. Average daily traffic during 1963, the year of the study, was 4, 600 vehicles. Due to seasonal variation, the traffic encountered during the field operations of the study was in excess of this volume.

I 90 is a four-lane divided facility with concrete driving surfaces. The asphalt shoulders are 10 ft wide on the right side and six ft wide on the left. The average width of the grassy depressed median is about 50 ft. The posted maximum speed limit during the study period was 70 mph for cars and 60 mph for trucks. There was no minimum speed posted.

I 190 has the same design as I 90 except for a 12-ft wide raised median and a 2-ft shoulder on the left side.

Access to the study section is provided by 10 interchanges of various designs (Fig. 1). There were no rest areas along the study section and no parking restrictions posted on the mainline during the study period. Also, there were no roadside attractions along the highway or physical features of the highway that would appear to cause abnormal shoulder parking or grouping of shoulder users.

Since the study section was opened to traffic in October 1960, accident records were available for analysis for the period 1961 to 1963. A total of 62 accidents was reported during the 3-yr period and three, or about 5 percent, involved shoulder users.

STUDY PROCEDURES

The field work was started July 9, 1963, in conjunction with a phase of Interstate System Accident Study No. 2, and finished October 5, 1963. Shoulder interviews were conducted during the 12-hr period from 6 a. m. to 6 p. m. each day of the week, every other week during the study period. In all, there were 47 days of interview data collection. During the weeks that interviews were not taken, vehicle classification counts were taken at selected points along the study section.

The shoulder usage interview data were collected by the drivers of an unmarked departmental car who continuously patrolled the study section during the study period.

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 45th Annual Meeting.

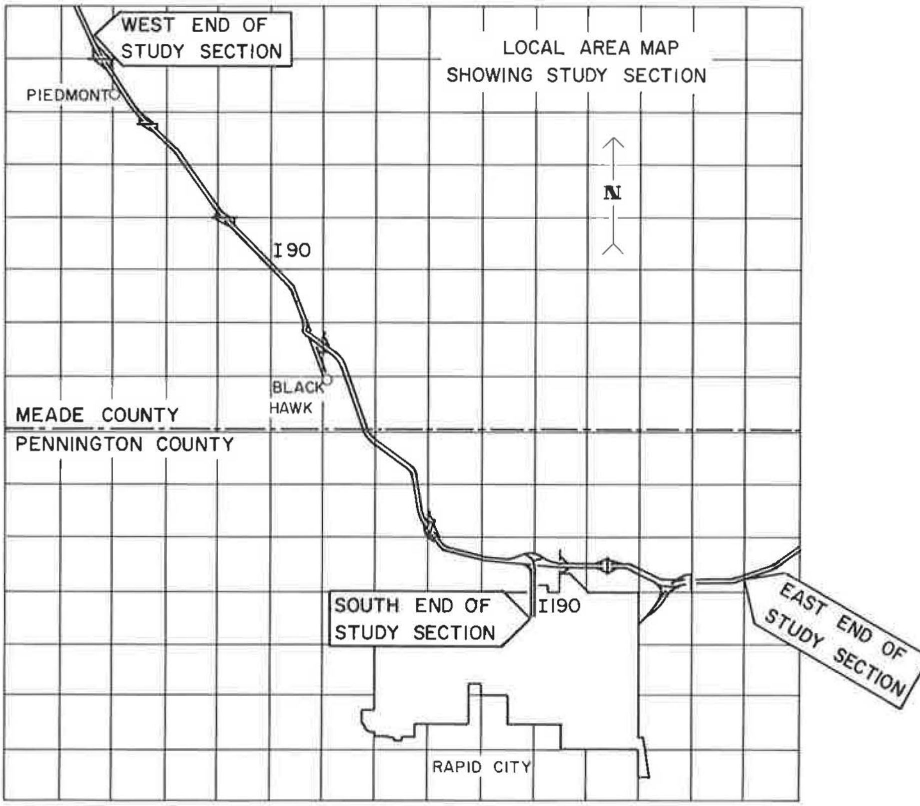

Figure 1. Interchanges along I 90 and I 191.

All motorists found parking on the shoulders in the direction of travel, except those involved in an accident, were asked to furnish information necessary to complete the questionaire (Fig. 2). Tabulation and analysis of the data contained in a total of 362 completed questionnaires are given later.

SHOULDER STOPS

Purpose

Table 1 gives the number and percentage distribution of observed shoulder stops, both voluntary and involuntary, classified by purpose of stop.

Approximately 61 percent of total shoulder stops observed were "voluntary." Voluntary stops were further subdivided into three broad classes as follows: (a) rest and leisure, (b) business and (c) other purposes. Fifty percent of the voluntary stops were for rest or leisure, 2 percent were for business and 48 percent for other purposes. "Checking maps" and "inspecting vehicle or load" are two predominant reasons for shoulder parking in the voluntary category (Table 1). The frequency of stops for these purposes can be explained by the large volume of tourist and truck traffic found on the Interstate highway during the survey period.

About 39 percent of the total shoulder stops were "involuntary." Mechanical failures were listed as the purpose of stop for 56 percent of the involuntary stops. Of the remaining involuntary stops, 23 percent were due to flat tires, 19 percent were caused by depleted fuel supply and 2 percent were due to other reasons. Table 2 gives the

Observer ______________________ Date ______________________

(1) TIME: a. Observed ________
b. How long have you been here? ________
☐ a.m. c. How much longer will you stay? ________
☐ p.m.
d. Actual departure time, if observed ________

(2) STOP DURATION:
a. Departed ______
b. Stopped ______
c. Length of Stay ______
(Office Use Only)

(3) DIRECTION OF TRAVEL: ☐ Northbound ☐ Southbound ☐ Eastbound ☐ Westbound

(4) STOP LOCATION: Delineator No. ________
Vehicle stopped:
On Median Shoulder ☐
On Outside Shoulder ☐
Distance parked from pavement edge ______ft. ______in.

(5) VEHICLE TYPE:
Passenger ☐
Truck and Bus ☐

(6) REGISTRATION:
State ☐
Out of State ☐
Unidentified ☐

(7) IDENTIFICATION ____________

(8) TRIP PURPOSE:
☐ Business
☐ Driving to or from work
☐ Vacation
☐ Recreation
☐ Social
☐ Shopping
☐ Other ________ (Explain)

(9) LAST STOP:
a. Time ______
b. Location ______
c. Estimated Elapsed Time ______

(10) NUMBER OF OCCUPANTS: ______

(11) TYPE AND PURPOSE OF STOP:

Voluntary

Rest and Leisure
☐ Rest or Sleep
☐ Checking Map
☐ Changing Drivers
☐ Eating in Vehicle
☐ Car Sickness
☐ Recreation (Picnic, Fishing, Etc.)
☐ Visiting
☐ Latrine
☐ Other

Business
☐ Discharging or Picking up Pass. (Bus Only)
☐ Inspecting Utilities
☐ Inspecting Farm and Crops
☐ Inspecting Industry
☐ Other ____________ (Describe)

Other Voluntary
☐ Assisting Vehicle
☐ Checking Vehicle or Load
☐ Minor Mechanical Trouble
☐ Police Enforcement Stop
☐ Police Assist. Stop
☐ Stopped by Police
☐ Other ____________

Involuntary
☐ Flat Tire
☐ Out of Gas
☐ Involved in Accident
☐ Mechanical Failure
☐ Other ____ (Describe)

Distress Signal Used: Yes ☐ No ☐ Knows About Distress Signal: Yes ☐ No ☐

Comments: ______________________________________

Note: This form to be completed for each Shoulder Stop Interview.

Figure 2. South Dakota Department of Highways Shoulder Stop Interview Form.

number and percentage distribution of causal factors of the involuntary stops of passenger and commercial vehicles. Due to the small sample of trucks and buses, the percentage differences determined for passenger and commercial vehicles may not be significant; however, they do seem to be logical.

Figure 3 shows the distribution of all observed shoulder stops by vehicle type and purpose of stop.

Distance Between Observed and Previous Stop

Figure 4, a cumulative frequency distribution diagram, shows the distance in miles driven between the previous stop and the observed shoulder stop, classified by vehicle

TABLE 1

NUMBER OF SHOULDER STOPS CLASSIFIED BY TYPE AND PURPOSE OF STOP

Type	Purpose of Stop	No.	Percent of Class	Percent of Total
Involuntary	Flat tire	33	23	9
	Out of gas	28	19	7
	Mechanical failure	80	56	22
	Involved in an accident	Not Interviewed		
	Other	3	2	1
	Subtotal	144	100	39
Voluntary	Rest and leisure			
	Rest or sleep	12	11	3
	Checkimg map	54	50	15
	Changing drivers	13	12	4
	Eating in vehicle	8	7	2
	Car sickness	3	3	1
	Recreation	0	0	0
	Visiting	7	7	2
	Latrine	6	5	2
	Other rest or leisure	6	5	2
	Subtotal	109	100	31
	Business			
	Discharging or picking up passengers (buses only)	0	0	0
	Inspecting utilities	4	80	1
	Inspecting farm or crops	0	0	0
	Inspecting industry	0	0	0
	Other business	1	20	0
	Subtotal	5	100	1
	Other voluntary			
	Assisting another vehicle	8	8	2
	Checking vehicle or load	46	44	13
	Minor mechanical trouble	35	34	10
	Police enforcement stop[a]	Not Interviewed		
	Police assisting stop[a]	Not Interviewed		
	Stopped by police[b]	1	1	0
	Unclassified	14	13	4
	Subtotal	104	100	29
	Total	362	—	100

[a]Interviewers were instructed not to interview these stops.
[b]Interviewed after police left.

TABLE 2

REASONS FOR INVOLUNTARY STOPS OF VEHICLES OBSERVED PARKING ON HIGHWAY SHOULDERS

Reason	Passenger Vehicles		Trucks and Buses		All Vehicles	
	No.	%	No.	%	No.	%
Flat tire	27	24	6	17	33	23
Out of gas	23	21	5	15	28	19
Involved in accident	Not Interviewed					
Mechanical failure	59	54	21	62	80	56
Other	1	1	2	6	3	2
Total	110	100	34	100	144	100

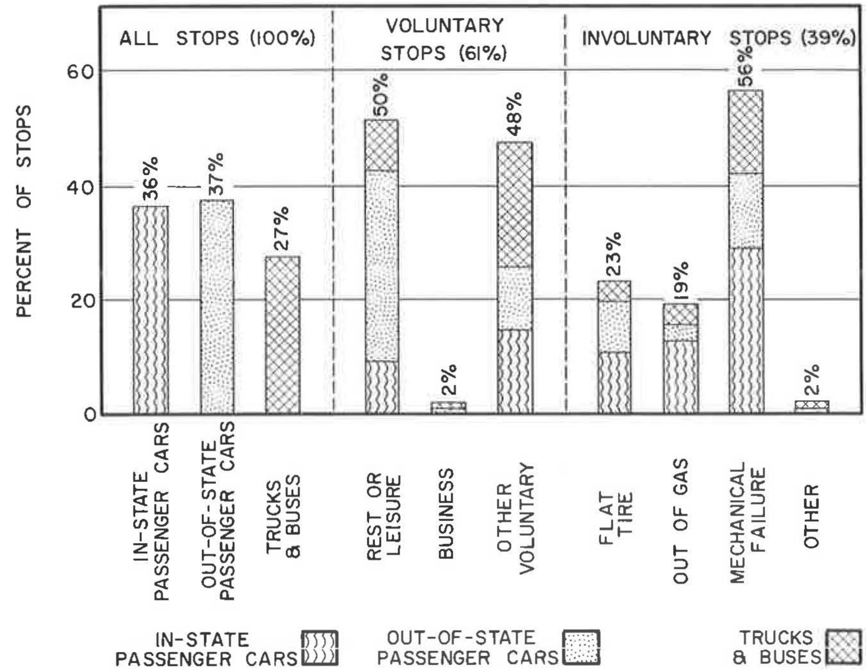

Figure 3. Distribution of shoulder stops by vehicle type and purpose of stop.

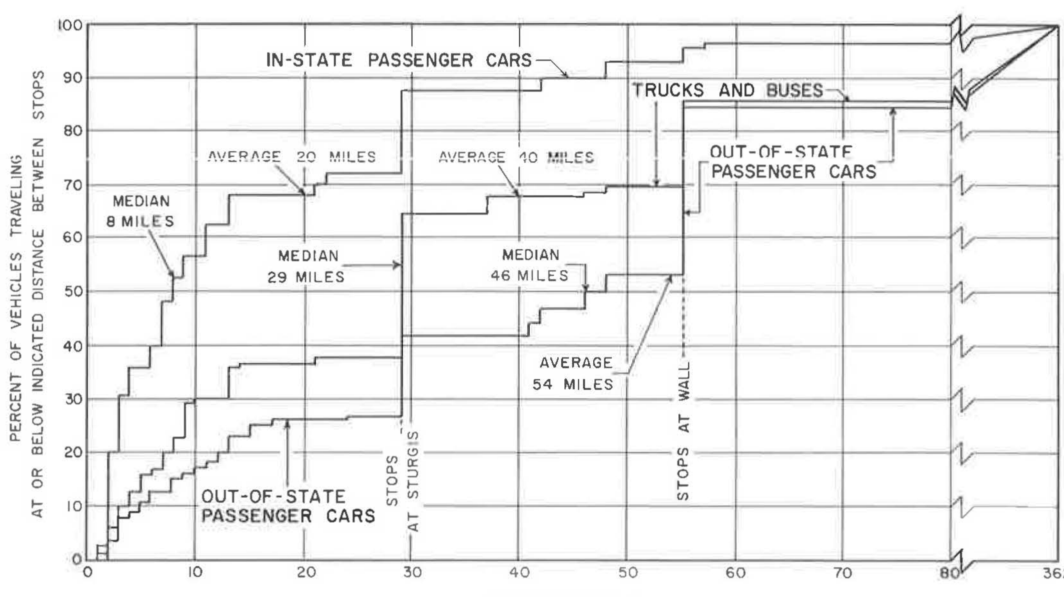

Figure 4. Distance interval between observed shoulder stop and previous stop, classified by vehicle type.

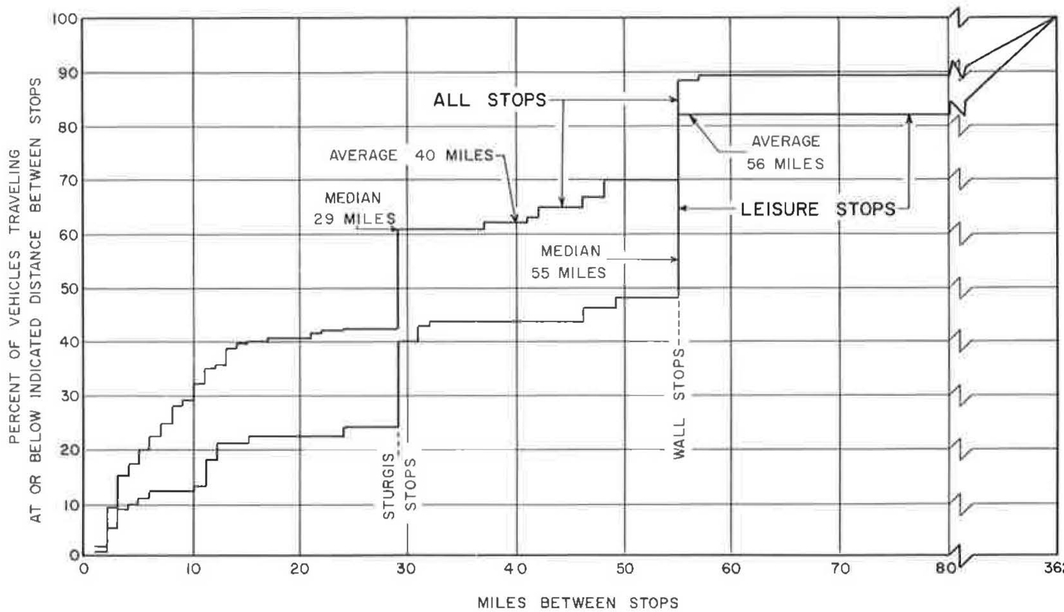

Figure 5. Distance interval between observed shoulder stop and previous stop, classified as leisure and all stops.

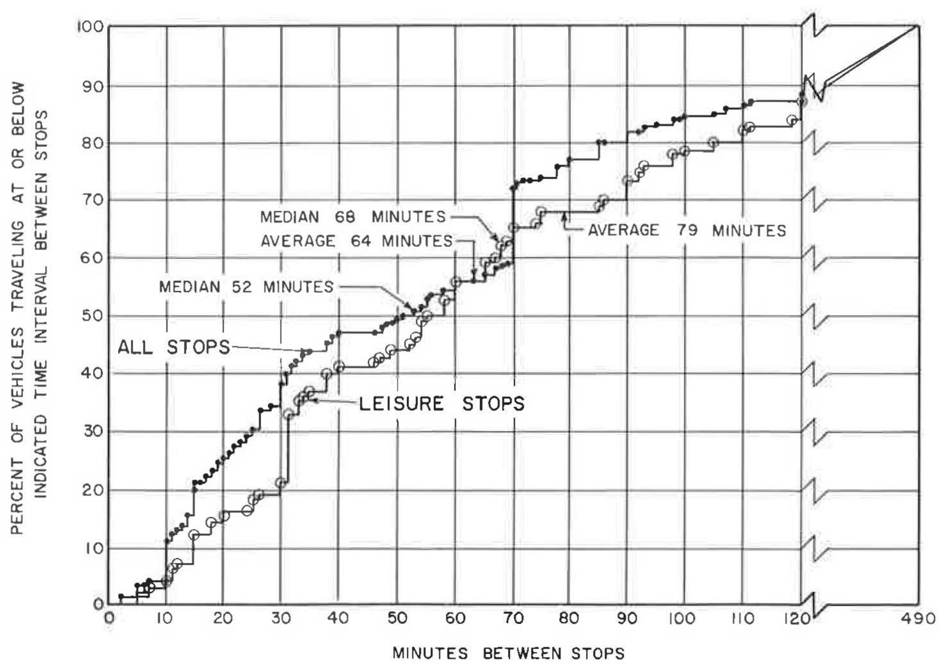

Figure 6. Time interval between observed shoulder stop and previous stop, classified as leisure and all stops.

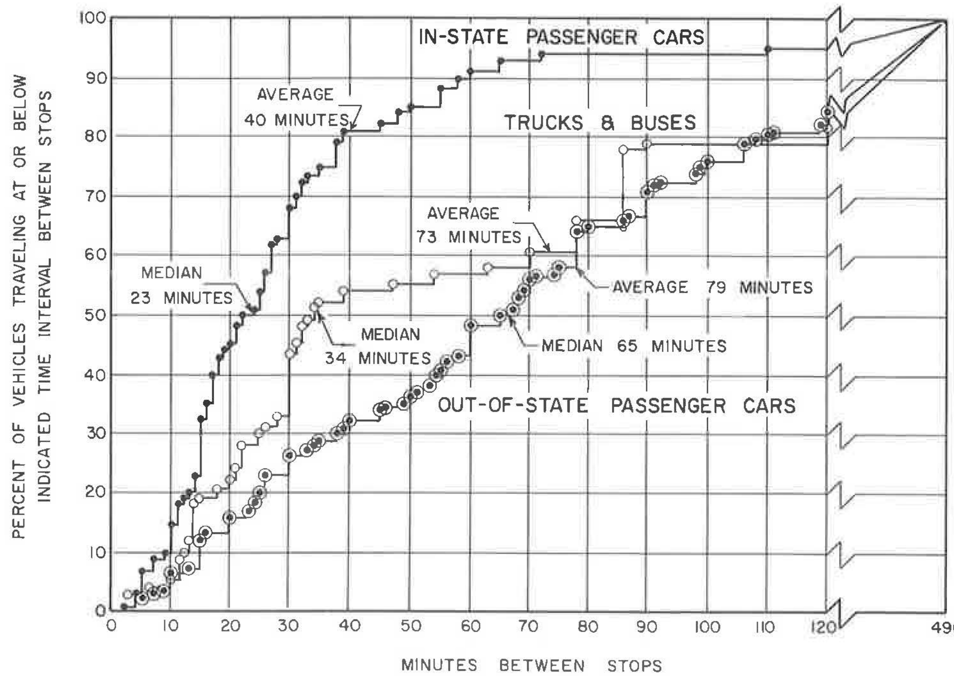

Figure 7. Time interval between observed shoulder stop and previous stop by vehicle type.

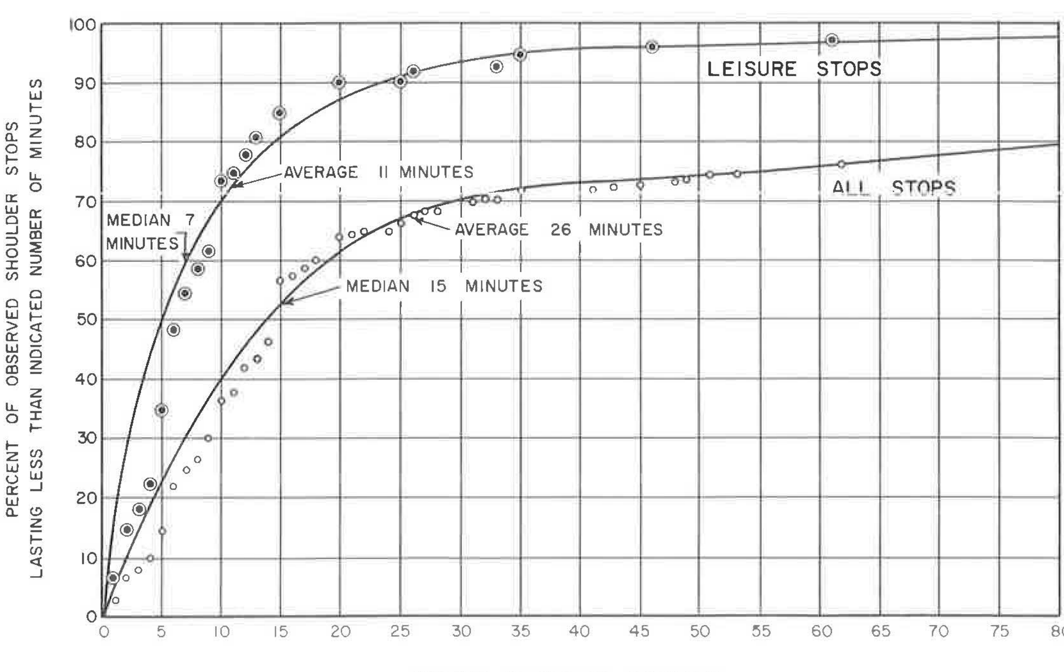

Figure 8. Percent of observed shoulder stops lasting less than indicated number of minutes classified as all stops and leisure stops.

TABLE 3

NUMBER OF OBSERVED SHOULDER STOPS CLASSIFIED BY HOUR OF OCCURRENCE AND TYPE OF STOP

Hour	Involuntary Shoulder Stops	Voluntary Shoulder Stops				All Shoulder Stops
		Rest & Leisure	Business	Other	Total	
6:00- 7:00 a. m.	22	5	0	4	9	31
7:00- 8:00 a. m.	18	6	0	5	11	29
8:00- 9:00 a. m.	9	7	0	8	15	24
9:00-10:00 a. m.	12	6	1	12	19	31
10:00-11:00 a. m.	7	13	0	9	22	29
11:00-12:00 a. m.	16	14	0	5	19	35
12:00- 1:00 p. m.	12	13	0	5	18	30
1:00- 2:00 p. m.	9	4	0	3	7	16
2:00- 3:00 p. m.	8	12	1	10	23	31
3:00- 4:00 p. m.	14	7	1	15	23	37
4:00- 5:00 p. m.	8	16	2	18	36	44
5:00- 6:00 p. m.	9	6	0	10	16	25
Total	144	109	5	104	218	362

TABLE 4

PERCENTAGE DISTRIBUTION OF OBSERVED SHOULDER STOPS BY HOUR OF OCCURRENCE AND TYPE OF STOP

Hour	Involuntary Shoulder Stops	Voluntary Shoulder Stops				All Shoulder Stops
		Rest & Leisure	Business	Other	Total	
6:00- 7:00 a. m.	15	5	—	4	4	9
7:00- 8:00 a. m.	13	5	—	5	5	8
8:00- 9:00 a. m.	6	6	—	8	7	6
9:00-10:00 a. m.	8	6	20	11	9	9
10:00-11:00 a. m.	5	12	—	9	10	8
11:00-12:00 a. m.	11	13	—	5	9	10
12:00- 1:00 p. m.	8	12	—	5	8	8
1:00- 2:00 p. m.	6	4	—	3	3	4
2:00- 3:00 p. m.	6	11	20	9	11	9
3:00- 4:00 p. m.	10	6	20	14	11	10
4:00- 5:00 p. m.	6	15	40	17	16	12
5:00- 6:00 p. m.	6	5	—	10	7	7
Total	100	100	100	100	100	100

type and registration. Out-of-state passenger cars traveled the greatest distance, or an average of 54 mi, between stops. Trucks and buses logged 40 mi between stops and in-state passenger cars traveled only 20 mi between stops. The average travel distance for all vehicles between stops (Fig. 5) was 40 mi, exactly twice the distance as for in-state passenger cars.

Figure 5 also provides a comparison of distance interval for total shoulder stops as opposed to shoulder stops of a leisure nature. The average travel distance between stops of those observed and classified as leisure was 56 mi, or 16 mi longer than the average of all shoulder stops. Distance intervals for leisure stops and out-of-state passenger car stops were almost identical at 56 and 54 mi, respectively.

Time Interval Between Observed and Previous Stop

Figures 6 and 7 are quite similar to Figures 4 and 5 except that time interval, rather than distance interval, is considered for analysis. The average time between

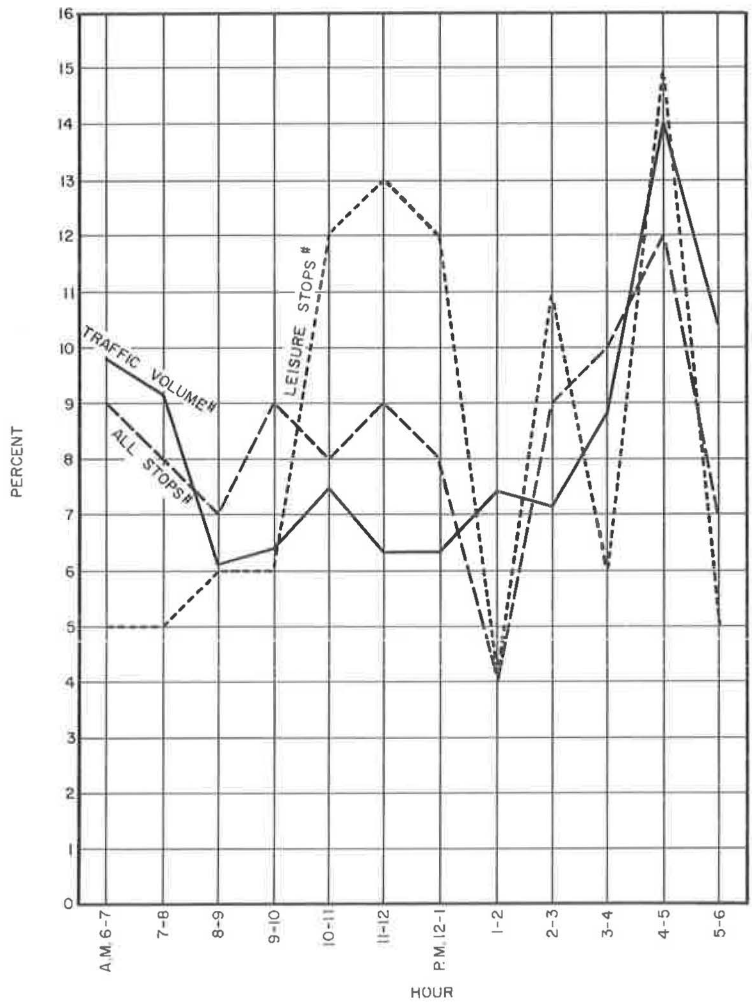

Figure 9. All stops and leisure stops compared to traffic volume on an hourly basis.

TABLE 5

PURPOSE OF TRIP AS GIVEN BY DRIVERS INTERVIEWED WHILE PARKING ON HIGHWAY SHOULDER

Purpose	Passenger Vehicle				Trucks & Buses		All Vehicles	
	In-State		Out-of-State					
	No.	%	No.	%	No.	%	No.	%
Business	30	23	11	8	76	76	117	32
Driving to or from work	19	15	4	3	10	10	33	9
Vacation	1	1	98	74	1	1	100	28
Recreation	10	8	1	1	0	0	11	3
Social	7	5	5	4	0	0	12	3
Shopping	6	5	0	0	0	0	6	2
Other	3	2	1	1	4	4	8	2
Unknown	54	41	12	9	9	9	75	21
Total	130	100	132	100	100	100	362	100

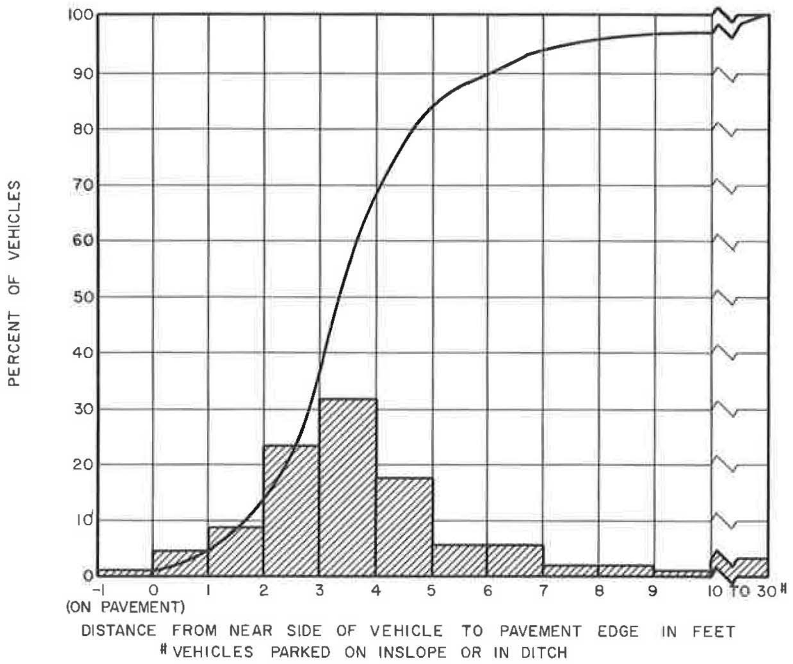

Figure 10. Percentage distribution of vehicles parked on highway shoulders by distance from pavement edge.

stops for in-state passenger cars was 40 min; for out-of-state passenger cars, 77 min; for trucks and buses, 73 min; for all leisure stops, 79 min; and for total observed shoulder stops, 64 min.

Duration of Observed Shoulder Stops

Figure 8 shows a comparison of the duration of time that vehicles were parked along the highway shoulder for total observed stops and those of a leisure nature. The average length of leisure stops is 11 min as compared to nearly 26 min for all stops.

About one-fourth of all stops were for a duration of 1 hr or over. Ninety-six percent of these longer stops were of an "involuntary" nature. The shorter stops, or more specifically, stops with a duration of 8 min or less, were mostly "voluntary" in nature and accounted for 88 percent of the stops in that time grouping. Also, about 64 percent of all stops lasting 8 min or less were for "rest and leisure" purposes.

Hourly Distribution of Observed Shoulder Stops

Tables 3 and 4 give the distribution of observed shoulder stops by hourly periods according to purpose. Figure 9 shows that the distribution of stops for all purposes is quite similar to the hourly volumes of traffic using the study section. The hourly distribution of leisure stops does not conform as well with traffic volumes as all stops, especially during the hours of 6:00 a. m. to 8:00 a. m. and 10:00 a. m. to 2:00 p. m. Leisure stops are low in proportion to traffic volume during the early morning peak and extremely high during off-peak hours, 10:00 a. m. to 1:00 p. m. A sharp drop in leisure stops between 1:00 p. m. and 2:00 p. m., when many trips might be started or resumed after the noon luncheon, is shown.

Trip Purpose of Motorists Making Shoulder Stops

The purpose of trip as given by the drivers interviewed while parked on highway shoulders is given in Table 5. Of the total trips, 32 percent were for the purpose of

TABLE 6

DISTRIBUTION OF TRAFFIC AND VEHICLE MILES BY VEHICLE TYPE AND DIRECTION OF TRAVEL

Direction	Miles	In-State Pass.		Out-of-State Pass		Truck or Bus		Total	
		Traf.	V. M.	Traf.	V. M.	Traf.	V. M.	Traf.	V. M.
Eastbound	19. 07	1, 127	21, 494	537	10, 250	235	4, 477	1, 899	36, 221
Westbound	18. 89	1, 297	24, 504	535	10, 112	234	4, 412	2, 066	39, 028
Total both directions		2, 424	45, 998	1, 072	20, 362	469	8, 889	3, 965	75, 249

TABLE 7

AVERAGE VEHICLE-MILES PER STOP BY VEHICLE TYPE AND PURPOSE OF STOP

Type	Voluntary Stops				Involuntary Stops	All Stops
	Rest & Leisure	Business	Other	Total		
Passenger:						
State	11, 900	0	10, 200	5, 500	12, 000	3, 800
Out-of-state	1, 400	159, 500	7, 900	1, 200	10, 600	1, 100
Subtotal	3, 600	519, 800	9, 300	2, 600	11, 600	2, 100
Truck or bus	2, 900	99, 500	1, 900	1, 000	4, 100	800
Total	3, 500	73, 700	6, 400	2, 200	9, 500	1, 800

business and 28 percent were listed as vacation travel. Purpose of trip was not obtained for 21 percent of the vehicles found parked during the period of the study.

As might be expected, 74 percent of the motorists operating passenger vehicles registered outside of the state were traveling on vacation. About 38 percent of the drivers of passenger vehicles registered in South Dakota were on business or driving to and from work. A high proportion, or 86 percent, of the truck trips were for work purposes.

Vehicle Parking Position on Highway Shoulders

Less than 1 percent of shoulder users parked on the left or median shoulder. It can be concluded that there is little need to use the median shoulder in view of the low traffic volume and the availability of the wide right shoulder.

Figure 10 gives the percentage distribution of vehicles parked on the shoulders according to the distance from the pavement edge. Only 10 percent of the vehicles were parked at a distance of 6 ft or more from the pavement edge, which is considered as a minimum hazard by the Committee on Shoulders and Medians. Thirty-eight percent of the shoulder users parked 3 ft or less from the pavement edge which is considered as an extreme hazard.

COMPUTATION OF VEHICLE-MILES PER SHOULDER STOP

The analysis of data to this point was based on observed shoulder stops. In order to compute the rate or vehicle-miles per stop it was necessary to develop a method to expand the number of observed stops to represent the actual number of shoulder stops occurring on the study section.

To obtain the necessary expansion factors, the average time (53 min) to travel the study course was divided by the time a vehicle was found to be stopped along the shoulder. This procedure was based on the theory that the probability of a shoulder stop being observed is equal to the time the vehicle was stopped divided by the time it would take the observer to traverse the study section. Since the time in which vehicles were stopped on the shoulder was stratified into various time groups, the midpoint of each time group was considered an accurate approximation of the mean stop time for that group. The number of observed stops for each time group was then multiplied by the corresponding expansion factor to obtain the total number of actual stops. As a result of this adjustment, the original 362 observed stops were expanded to a total of 1,983 actual stops.

Average traffic volumes and vehicle-miles of travel by vehicle type during the hours of the study period used in the determination of the rate or vehicle-miles per stop are given in Table 6.

Table 7 gives the rate or vehicle-miles per stop as determined for the various types of vehicles according to shoulder stop purpose.

A shoulder stop was made by an in-state passenger vehicle for each 3,800 miles of travel compared to 1,100 miles for a passenger vehicle registered outside of the state. The average passenger vehicle using the study section made a shoulder stop each 2,100 miles. Trucks or buses traveled about 800 mi before each shoulder stop occurrence.

Vehicle-miles per stop for voluntary purposes ranged from 1,000 for trucks to 5,500 for in-state passenger vehicles. Out-of-state passenger vehicles made a voluntary stop each 1,200 miles.

Involuntary stops by trucks were much more frequent than those experienced by passenger vehicles. An involuntary stop was made each 4,100 miles compared to 11,600 for the passenger vehicle.

SUMMARY AND CONCLUSIONS

It was determined from this study of a typical section of Interstate highway that one motor vehicle parks on the road shoulder for each 1,800 vehicle-miles of travel. A large majority of these motorists utilized shoulder areas for reasons other than those of an emergency nature. Provisions for properly spaced, adequately designed and well marked safety rest areas could reduce the volumes of shoulder users and minimize the chances for vehicle collisions.

That accidents do occur as a result of shoulder parking is verified by accident analysis which indicated that about 5 percent of all accidents occurring during the 3-yr period involved shoulder parkers. The data collected in the shoulder study indicated that about 38 percent of the shoulder users parked 3 ft or less from the pavement edge which is considered an extreme hazard. This factor may be contributing to the accident frequency rate of the highway section.

Involuntary shoulder stops caused by mechanical failure or depleted fuel supply pose a special problem. It was learned that these two items account for 75 percent of the involuntary stops. Some provisions will have to be made to promptly service or remove these "hazards" from the controlled-access facility.

The Bureau of Public Roads has authorized a rest area for about every 35 mi of Interstate highway. When questioned as to the distance between stops, the average shoulder parker indicated a distance of 40 mi. The small difference between the two figures tends to substantiate the 35-mi spacing of rest areas when water supplies and terrain features will allow their construction.

Information contained within this report will provide a basis for planning rest area facilities adequate for anticipated traffic volumes of varying characteristics when considering highway improvements, especially those improvements with full control of access.

The shoulder stop rates give some insight as to the extent of policing necessary as new sections of Interstate highway are opened to traffic. The effects of increasing traffic volumes on the existing sections of controlled-access highway might also be predicted from the use of the same material.