HIGHWAY RESEARCH BOARD
Bulletin 303

Motor Vehicle Speed, Volume, Weight and Travel Times

National Academy of Sciences—
National Research Council

publication 931

The opinions and conclusions expressed in this publication are those of the authors and not necessarily those of the Highway Research Board

N.R.C. HIGHWAY RESEARCH BOARD

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Presented at the

40th ANNUAL MEETING

January 9-13, 1961

National Academy of Sciences—
National Research Council
Washington, D.C.
1961

$2.40

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Effect of Speed Limit Signs on Speed on Suburban Arterial Streets

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Speed control has been identified as one of the most important tools in attempting to counteract excessively fast driving and to reduce accidents. At the same time, speed control also has been recognized as a difficult and controversial issue due to the fact that criteria for establishing speed zones do not have the same degree of acceptability as have other traffic controls; for example, no-passing zones or traffic signals. Numerous speed studies have been conducted throughout the United States; very few, however, deal with speed patterns within the fringe area between the genuine rural and genuine urban area, generally termed the suburban area.

This paper reports the results of a study at five suburban areas of the combined effect of various speed limits and roadside development on driver speed patterns. Three of the locations concerned were two lane U.S. highways traversing suburban development of predominantly residential, commercial, and industrial type, respectively. One location was a four-lane, partially divided highway recently reconstructed from two lanes to four lanes. No speed limits had been posted along this highway after the reconstruction had been completed. Consequently, this highway was an excellent location for studying the effects of various posted speed limits, making possible comparisons between unrestricted and restricted speeds. Only free moving passenger vehicles traveling on dry pavement under optimum day and night visibility conditions were considered. Speed patterns for local and non-local vehicles were analyzed separately; local vehicles being those officially registered in the actual county as verified by the prefix on the license plate. Spot speeds were recorded, using an Electro-matic radar speed meter, at various points at each location, during day and night, before and after speed limit signs had been posted. Altogether, 31,573 vehicles were recorded, of which 20,552 were during the day and 11,021 were during the night.

The study revealed that the drivers paid little, if any, attention to posted speed limits. They seemed to choose a speed which they themselves considered appropriate for prevailing conditions. Non-local vehicles traveled significantly faster than local vehicles in many instances. Noticeable differences between day speeds and night speeds were also recorded at some sites. Variabilities in the speed distribution at a few individual stations for the same sign and road conditions were unexpectedly high, thus somewhat challenging the accuracy of fixed-point speed measurements.

The influence on the speed pattern of various types of roadside development was difficult to distinguish intelligibly, and was evidenced by the revelation of distinct interaction effects between the diverse factors. The limited number of locations, adequate for the roadside development study, also contributed to making difficult an ultimate conclusion.

● FOR YEARS philosophers have argued that speed, per se, is not destructive. The elliptical path traveled by the earth with a speed exceeding 66,000 mph in its annual trip around the sun is actually a highly advanced freeway. There are, however, as yet, no appreciable conflicts or frictions along this freeway in terms of crossroads or other vehicles.

To a mathematician, physicist, or astronomer, speed may simply mean the ratio of distance to time; to a traffic engineer speed means nothing, but in conjunction with terms such as mean speed, speed percentiles, speed zones, and other measurements it has significant connotations.

Man always has strived and always will strive for attaining faster and faster speeds on the land, on the sea, and in the air. The problem, therefore, is not only to develop vehicles which are able to travel at high speeds, but also to develop additional devices which will assist man to safely and efficiently control and use the speeds attained. McMonagle (1) has stated the same thing this way: "Speed is one of the great essential benefits which make highway transportation indispensable in modern America. It must be provided for and protected."

What speed is safe? Drastically expressed, the only safe speed is 0 mph (2,3), for accidents occur at all speeds. However, higher speeds do increase the chances of exposure to situations and the rapidity at which these develop may reduce the ability of a driver to react properly and may lead to more accidents. It also is generally true that speed is a factor in the severity of an accident (4).

Ever since the early growth in the use of motor vehicles, special attention has been given the problem of safe speed. In 1925 Dickinson and Marvin (5) defined safe speed as "such that a driver will be able to stop his vehicle within the distance ahead that is certain to be free from any obstruction." This is certainly still true today. What is then a good driver? Johnston (6) defines him as "one who starting on a journey regardless of the roadway, weather, vehicle, or traffic arrives at his destination without having any trouble himself or without causing anyone else any trouble." Is this a driver who always drives at a safe speed? A research study in Pennsylvania (7) revealed that the speeds of drivers with accident records were only slightly higher than those for drivers without accident records.

Yet, speed is often advocated as the greatest contributing cause to accidents. Thorough studies, however, have indicated that this may not be entirely true. A study in Minnesota of 40,000 accidents (2) in which data as to vehicle speeds were stated, showed that if every accident in which speed was the only violation could have been prevented the number of accidents would have been reduced less than 10 percent, and if all accidents could have been prevented in which speed was the primary cause of the accident the reduction in number of accidents would not have been much greater. Actually nearly 75 percent of the accidents involved some violation other than a speed violation. In 1955 the Chicago Park District ascribed speed as the principal cause of only nine percent of automobile accidents (8).

It is not surprising, therefore, that speed control is one of the most difficult and controversial problems of traffic operations. Criteria for establishing speed zones do not have the degree of acceptability as have such other traffic controls as no-passing zones or traffic signals. Speed control is difficult because variations in driving behavior between individuals complicate the establishment of adequate, reasonable, and uniform warrants based on objective speed surveys. Speed control is controversial because of the divergence of opinion among engineers, enforcement officers, the motorists, and the people living along the highway as to the proper speed and the appropriate methods for controlling speeds.

Speed control, however, is important because properly done it may facilitate movement at uniform speed and thereby contribute to improved capacity and safety. It may also assist the motorist in selecting speeds that are safe and that permit him to obtain the maximum utility, economy, and convenience from his vehicle and the road. Speed zoning has therefore developed and is defined as the establishment of reasonable speed limits, based on engineering study, for sections of street or highway where any general statewide legislative speed limits do not fit the road and traffic conditions (9). Speed limits for such speed zones are established for favorable weather and traffic conditions with further reduction due to inclement conditions regarded as the responsibility of the driver.

Approaching an urban area, speed zones generally begin in the fringe areas — usually termed the suburban — in which roadside development of all kinds such as residences, filling stations, drive-in-theaters, cafes, and trailer courts may occur. This transition zone between rural and urban development is also often the transition zone for speed limits. Should there be a series of speed limit signs, gradually limiting the speed from the statewide limit in the rural area to the limit used in the urban area? What warrants should be used and where should signs be placed? Is the type of roadside development in the area a factor to be considered in signing the area? The answers to these questions were the purposes of a research project which the authors conducted and are the subject of this paper.

TABLE 1

SPEED STUDY LOCATIONS

Location	Urban Area	Highway	Direction from Which Vehicles Approach the Urban Area	Direction of Study	ADT
A	West Lafayette	US 52	North	Southbound & northbound	4,600-5,500
B	West Lafayette	Ind. 43	North	Southbound	2,800-4,300
C	Logansport	US 35	South	Northbound	3,500-4,100
D	Peru	US 31	North	Southbound	4,100-4,300
E	Wabash	US 24	East	Westbound	4,300-4,500

STUDY LOCATIONS

Five locations, given in Table 1 and shown in Figure 1, were selected as study sites. These locations generally met the following pre-set requirements:

1. The highway was straight and level so that the motorist could easily perceive the change from rural to suburban to urban area.
2. Posted speed limits if any, were not overly restrictive, and the limits were placed in reasonable decreasing order when approaching from the rural area.
3. No signals, stop signs or warning signs which might have an adverse effect on the speed pattern were present.
4. The roadside development was predominately of one type — residential, commercial, or industrial — on each section.

One of the five locations, referred to as Location A, was chosen for a comprehensive speed limit signing study due to the fact that the highway had been recently reconstructed from a two-lane to a four-lane facility and no speed limits had as yet been posted.

This highway in the rural area is a four-lane divided facility with a 30-ft wide grass median which narrows to a 5-ft concrete median as it enters the city limits (Fig. 2). The lane width is 12 ft. On one side of the highway are golf courses, on the other, some development which is not closely situated to the highway. The effect of this development on speed was subsequently found to be negligible. The highway inside the city has four undivided lanes, each 12 ft wide, and it runs through a developed residential area (Fig. 3). The test section of this highway was approximately 9,000 ft long with 3,800 ft of this distance primarily undeveloped or lightly developed and 5,200 ft highly developed as an urban residential area. The reconstruction of this highway was completed in the fall of 1958. From that date until June 1959 no speed zoning was in effect. Speed surveys were extensively performed during this period of time, especially in the inbound direction. At seven sites a radar speed meter was concealed and used to measure the speed of free-moving passenger vehicles. Vehicles were also classified into local and non-local vehicles with a local vehicle being one registered in the county

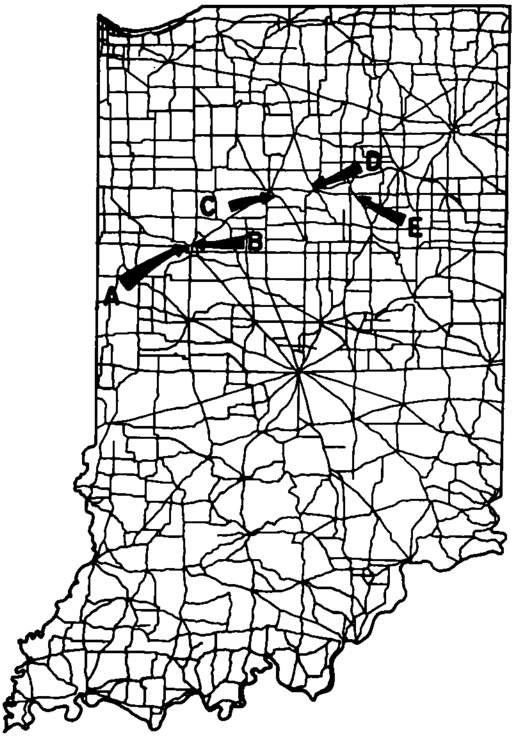

Figure 1. Study locations.

where the road was located. This was done because it was felt that local vehicles being familiar with the environment might be less sensitive to changes in posted speed limits than non-local vehicles. The effect of speed limits during day and night hours was also separately analyzed.

For more than one-half year this highway operated without posted speed limits and it was under these conditions that speed distributions for weekday operation were first obtained at the seven sites. It was from these comprehensive data that speed limits along the highway were determined and put into effect. The numerical values for the speed limits and their locations along the highway coincided as practically as possible with the 85th percentile speed found under no-sign conditions. Signs placed on the section included speed zone ahead and 50-, 40-, 35- and 30-mph speed limits. Only one sign at a time was posted starting with the higher speed limit, and the effect on the entire section was studied for several weeks before a new sign was added. This procedure continued until the complete signing plan (Table 2) had been completed.

Locations B, C, D, and E, the other four locations of this study, are similar in that they are the transition sections from rural to urban of major highways. Each of these locations was speed zoned prior to the study by the state highway department with the intent of reducing vehicle speeds from the 65-mph rural limit to a 30-mph urban limit. Location B was characterized by little development along it with none on one side of the highways due to a paralleling and nearby river. The development along Location C was almost entirely residential, that along Location D was primarily commercial and that along Location E primarily industrial.

Speed at Locations B, C, D, and E were taken at preselected sites throughout the suburban area in sufficient quantity to present a clear picture of the speed pattern for inbound vehicles in each zone. Speeds were taken in such a manner that the motorists were not aware that they were being checked. This was accomplished by concealing all personnel and the radar meter from the motorists view.

All speeds were taken at all locations under non-peak volume conditions and for each site this was when volume was less than the capacity of the road section. Speeds were taken of passenger cars only. Enforcement during the period of the study was not increased or decreased. Local authorities were requested to provide normal enforcement at the study locations.

TABLE 2

SPEED SIGNING PLAN AT LOCATION A

Area	Speed Sign Installed
Rural	65 mph, state limit
Approach to urban	Speed zone ahead
Residential development	Begin 50
Increasing in intensity	Begin 40
↓	40
	40
	Begin 35
	Begin 30

ANALYSIS OF SPEED PARAMETERS

The field data were analyzed to yield values of speed parameters for each station at each site by speed limit in effect at that station, residence of vehicle (local or non-local) and time of study (day or night). The parameters calculated were a measure of two characteristics of the speed distribution: the magnitude and the dispersion. The parameters of magnitude obtained were the mean speed, the median (or 50th percentile) speed, and the 15th and 85th percentile speed; those of dispersion were the variance (or square of the standard deviation), the speed differential, and the pace.

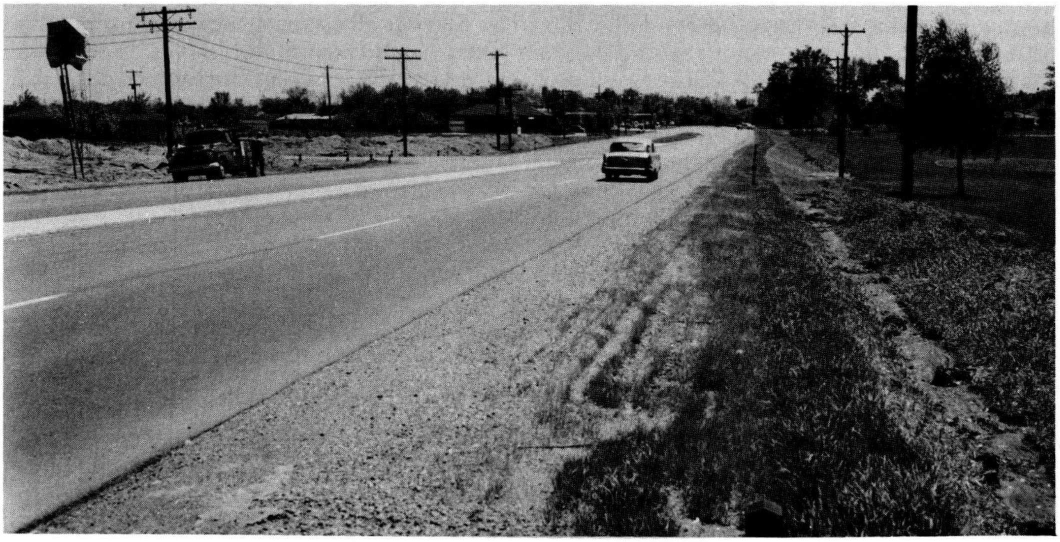

Figure 2. The sparse land development in the 50-mph zone of Location A is clearly evident.

Figure 3. The almost complete residential development in the 40-mph zone of Location A is indicated by the residential drives, mailboxes, and residences.

These parameters were then analyzed by statistical methods to determine if differences in parameter values were significant for the different conditions and the source of such variation.

Location A

The effect of speed limit signs on speed in a suburban area as found in the comprehensive study at Location A is typically illustrated by the effect on the 85th percentile speed in Figures 4 to 7. These figures are for local vehicles, non-local vehicles, day, and night conditions, respectively, and for the after condition the data shown were obtained with the full speed limit signing plan (Table 2) in place.

The 85th percentile speeds of local vehicles were slightly higher after posting speed limit signs than they were before (Fig. 4). The increase is slight, however, and may be because of the removal of the uncertainity from some local motorists as to whether a lower speed limit, which had existed on the road before its reconstruction, still was in effect even though no signs were present. On the other hand, non-local drivers appeared to change their speeds very little after the posting of the limits (Fig. 5). Although there were some slight mathematical differences between the before-and-after 85th percentile speeds in the respective cases shown, there were no statistically significant differences at a five percent significance level.

Analysis of other parameters — average speed, 15th and 50th percentile speed, and the 10 mph pace — also revealed that in general they were not affected by the placing of speed limit signs.

Two well-known speed characteristics were also noted in this study. Speeds were, at most locations, significantly lower at night than during the day, but only by a small amount. Local vehicles were found to travel significantly slower than non-local vehicles but again by only a small amount and only at some stations.

At one site additional signing was studied. Data were obtained there for four conditions of posted speed limit, no posted limit, 40 mph, 35 mph, and 30 mph. The effect on the various parameters is given in Table 3.

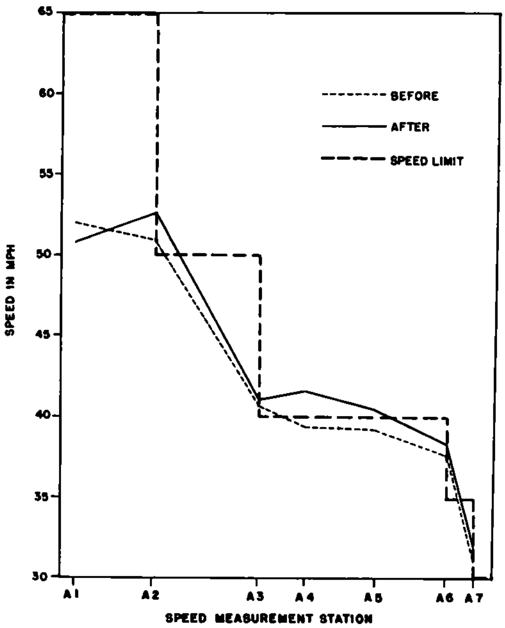

Figure 4. 85th percentile speed before and after posting of speed limits (local vehicles).

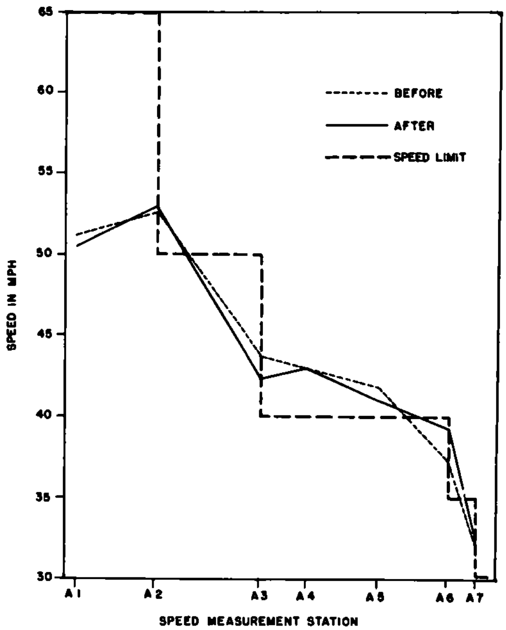

Figure 5. 85th percentile speed before and after posting of speed limits (non-local vehicles).

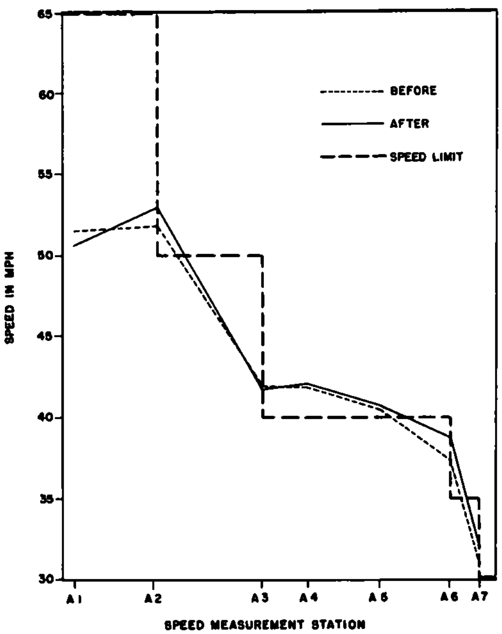

Figure 6. 85th percentile speed before and after posting of speed limits (day condition).

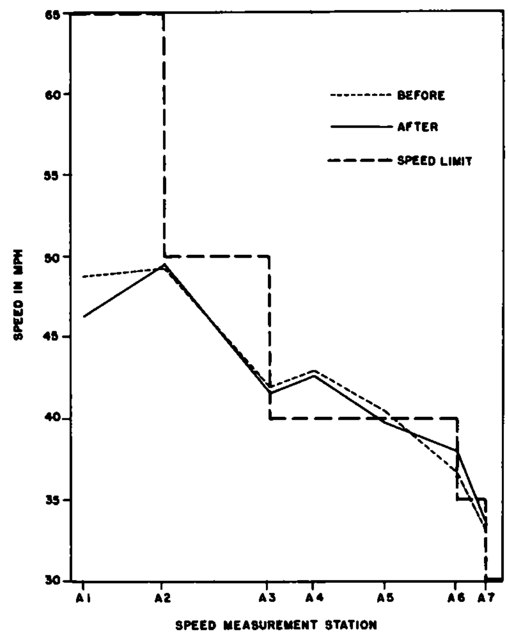

Figure 7. 85th percentile speed before and after posting of speed limits (night condition).

Although the results at this one site cannot be classed as conclusive, the results indicate the small changes which one might expect in speed parameters by speed zoning. The 85th percentile speed, for example, was found to be virtually the same for the posted 40-, 35-, and 30-mph limits; in other words, the numerical value on the speed limit sign did not have any effect on this parameter at this site. The lower speed limit sign did give a larger percentage in the pace but the pace occurred at a higher range. The possibility of efficient enforcement was also noted. It must be characterized as impossible to apprehend more than one-third of the drivers because of speeding 5 mph over the speed limit (the situation with 30-mph sign); in fact, reputable enforcement officials have stated that apprehension of more than one percent of the drivers would be difficult. This was the approximate percentage in violation more than 5 mph at the posted 40-mph limit at this site.

Locations B, C, D, and E

The results found at the four other locations were similar. Location B, which had little development along it, was found to have a decreasing 85th percentile speed as one approached closer and closer to the urban area, but this speed was found to have little relation to the posted limit at any specific location. This was true because the signs had not been placed as the result of a speed zone study.

Speeds at sites on Locations C, D and E, each one displaying a predominant type of development, were statistically tested against each other to determine whether or not there was any significant differences in speed pattern by type of development. The results revealed significant differences in speed between comparable locations (except for type of development) including significant interactions. The results indicated that type of development may significantly affect actual speeds but the factors which could have affected the speed were so many that it was not possible to come to definite conclusions regarding this effect or to place any quantitative values on it. It was again found, however, that non-local vehicles at all locations traveled noticeably faster than local vehicles. It was also found that night driving speeds generally were significantly

TABLE 3

SPEED PARAMETERS FOR PASSENGER CARS AT ONE SITE WITH DIFFERENT POSTED SPEED LIMITS

	Speed Limit Posted			
Speed Parameter	None	40 mph	35 mph	30 mph
Mean speed (mph)	31.99	33.48	33.91	34.31
85th percentile speed (mph)	37.38	38.44	38.42	38.49
Speed differential (mph)	9.77	9.83	9.09	8.47
10-mph pace	26-35	28-37	28-37	30-39
Percent in pace	73.0	72.8	74.6	78.9
Standard deviation (mph)	4.65	4.83	4.70	4.26
Percent of vehicles exceeding the speed limit by 5 mph or more	0	1.6	10.1	37.8

lower than day speeds at the 5 percent level but by different amounts and the differences were not always significant. Little relationship between the posted speed limit and the 85th percentile speed at that point was noted at any of the locations.

CONCLUSIONS

This study revealed that drivers, in general, do not drive according to posted speed limit signs. Under normal enforcement conditions most of them select a speed which they consider proper, reasonable, and safe for conditions prevalent, regardless of regulations. Moreover, it was shown that when speed limits are determined from the 85th percentile speed, traveled speeds are not materially affected. The fear that the establishment of a speed limit which appears relatively high to some will create a new speed pattern with faster speeds appears to have no foundation.

The status of speed limit signs in this area of Indiana appears to be that most drivers will obey a posted speed limit if, and only if, the sign is properly posted and the numerical limit is that which they would travel without signs. Such signing, however, does permit realistic enforcement of speed limits, and the use of speed limits as an enforcement tool appears to be, in the absence of having an effect on speed, a primary use of speed zoning, at least until better respect for speed limit signs has again been established with the motorists.

The study of the effect of roadside development on speed did not produce conclusive results, but the many significant differences noted in the speeds indicated a strong possibility that there is a significant effect on speed by type of development.

It was also clearly evident that the relationship between speed and roadside development is extremely complex. More detailed studies of this effect would be extremely valuable.

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Effects on Speed and Accidents of Improved Delineation at Three Hazardous Locations

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Delineation has been recognized as a useful tool in attempting to counteract the decrease in visibility attendant with the hours of darkness. In some circumstances it can be used as an inexpensive substitute for illumination. Little work has been done, however, in regard to the relationship between visibility, speed, and safety.

This paper reports the results of a study at three distinctive locations of the combined effect of various forms of delineation on driver speed patterns and on accidents. The locations concerned were a narrow bridge, a hazardous intersection, and an adequate intersection, all in rural areas. The various forms of delineation under consideration were roadside reflectors, pavement edge lines, signing, and, in one case, channelizing islands. Only free-moving passenger cars traveling on dry pavement under optimum day or night visibility conditions were considered in the speed study. In the case of one location heavy commercial vehicles were also recorded; however, speed patterns for cars and trucks were analyzed separately. Spot speeds were recorded, using Electro-matic radar speedmeters, at various points at each location during the day, at night, and again at night after the additional delineation had been installed. From 30 (for heavy trucks) to 300 or more observations, depending on the estimated variability of the speeds and prevailing traffic volumes, were made for each movement.

Accidents were also analyzed for a like period before and after placing the delineation, with proper delineation maintained during the after period.

The results were variable, but indicated a slight increase in speed and a slight reduction in accidents after the delineation.

THE MAJOR LIMITATIONS on the speed of a vehicle are those imposed by the driver, by limitations of the vehicle, or by legal statute. There are many locations, however, where the characteristics of the highway do, or should, impose an additional limit on speed, particularly at night when visibility is sharply decreased.

It has been shown by many researchers that most drivers vary their driving characteristics according to existing conditions rather than according to traffic controls. It follows that motorists must be made aware of highway and traffic conditions before they can react to them. Although warning signs, advisory speeds and other information are posted at many hazardous locations, the information on conditions ahead may not be adequately supplied to the motorist at all times.

Where there is doubt as to the nature of conditions ahead, caution would dictate a policy for the driver of going slow. Most drivers, however, prefer to drive at a uniform speed, and when they do not know of conditions ahead, continue at a speed too great for conditions, not realizing the hazard until too late. Such situations are especially prevalant at night when visibility is restricted and motorists are traveling at a high speed. The hazard of the situation is further increased by the large speed differential which exists between the overly cautious motorist and the incautious one.

One remedy for this situation would be to attempt to reproduce daylight conditions by means of highway illumination, as in urban areas. The cost of providing adequate illumination for a large extent of the rural highway system is, however, prohibitive. Some other means of enabling the driver to "see ahead", therefore, needs to be provided at many locations.

A method which has found considerable favor is the use of roadside reflectors, pavement edge lines, and other media as aids to the discernment of the course of the roadway. These devices serve not to illuminate the roadway but to outline or delineate it.

It has been found that the installation of illumination at high accident intersections has reduced the night accident rate at many of these locations. In addition to enhancing visibility, lighting may serve as a warning indication, thus alerting the driver to critical areas. The distant lamps at an illuminated intersection may also indicate to the approaching motorist the alinement of the roadway ahead.

Delineation also provides advance warning and an indication of the roadway and contributes to motorist recognition of conditions ahead.

SPEED-VISIBILITY STUDIES

This study was the first of several which are planned by the Joint Highway Research Project at Purdue University to investigate the relationship between visibility and speed at roadway friction points. This initial study was concerned with the effect of delineation on speed patterns and accidents at three hazardous or potentially hazardous rural locations: a narrow bridge, an inadequate intersection, and an adequate intersection. Illumination was eventually planned for these locations and a comprehensive study in the area of speed and visibility was developed for the period prior to and subsequent to the date of illumination.

DESCRIPTION OF STUDY

Variables

The various forms of delineation used in this study were roadside reflectors (delineators), pavement edge lines, signs, and, in one case, channelizing islands.

The types of vehicles studied were passenger cars and, at one location only, heavy trucks (primarily semi-trailers).

Only free-moving through vehicles were recorded at each location, a free-moving vehicle defined for the purposes of this study as one whose speed was not affected by the immediate presence of other vehicles in its path. In accordance with this definition, only the lead vehicle in a platoon was recorded. The minimum allowable headway for a following vehicle to have been considered free moving was about five seconds. Vehicles in the act of passing, or initiating or completing a pass were also not included in the sample. Similarly, vehicles which were not traveling at a free-moving speed because they had just turned onto, or were about to turn off, the through roadway were not included.

The major variable under investigation for the study reported in this paper was the degree of delineation. Accordingly, speeds were measured under three conditions: during daylight hours with existing delineation (considered to be the condition of optimum visibility), at night with the existing delineation, and again at night after additional delineation had been placed (the condition of optimum practical delineation). These three phases of the study were known, respectively, as the day, night-before, and night-after phases.

To further reduce the number of variables, data were collected on week days only and, as far as possible, at the same time each day. Data collection took place only when the pavement was dry, and under optimum atmospheric conditions (that is, absence of fog or haze). For the before phases, measurements were taken in the afternoon with data collection ceasing not later than one hour before sunset and at night after darkness was complete. The before phases were conducted during the months of July and August

1958. Optimum delineation was then placed and speed data for the after phases were collected in September and October 1958.

Equipment

Spot speeds were recorded by means of radar speed meters at various points along the roadway legs approaching the study locations. The radar meters were placed well back from the pavement so that the effect of their presence would be negligible. A result of this precaution was that the radar meter indicator readings represented only a component of the true vehicle speed. To correct for this factor, for calibration corrections, and for errors due to continuously changing voltage, a control car was constantly used. Comparisons between the control car's accurate speed and the corresponding meter indication yielded corrections for the remainder of the data.

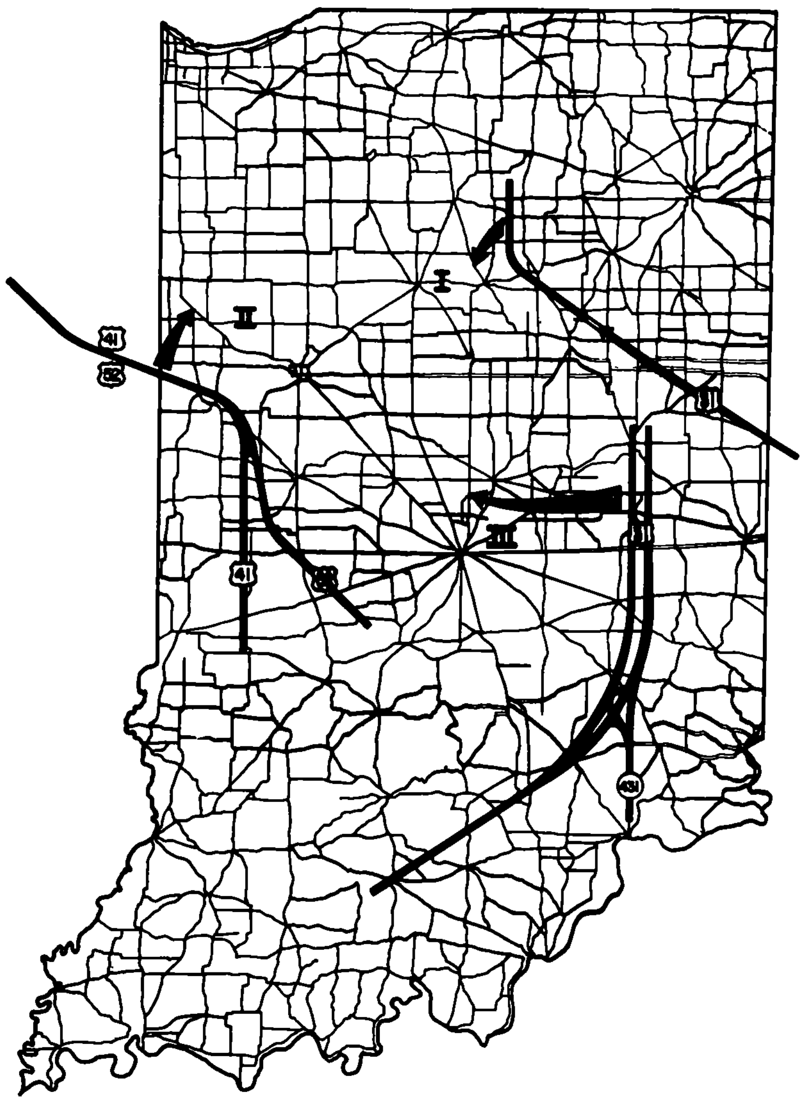

Figure 1. Study locations.

Speed Measurement Stations

Those points at which speeds were measured were termed stations and were situated in predetermined zones. On each roadway leg connecting with a study location, one station was operated at a point far enough removed from the immediate site of the study location so that motorists at that point could not see the changes in delineation. The purpose of these stations was to obtain data on the normal (open-highway) speeds of vehicles approaching or leaving the locations. These were termed, respectively, open-

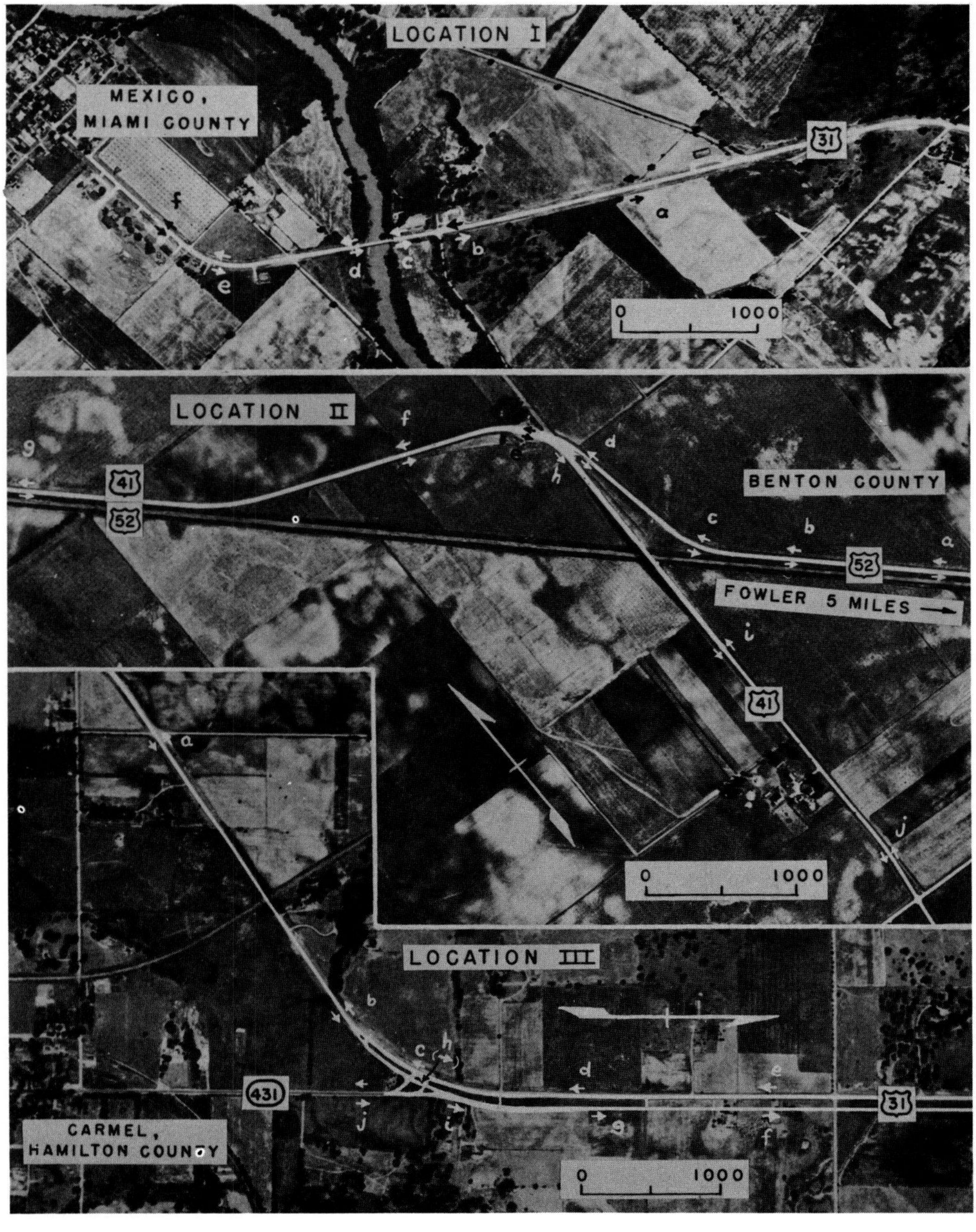

Figure 2. Study locations showing speed measurement stations.

highway approach and open-highway leaving stations in zones of the same name. The open-highway approach stations were also control stations as vehicle speeds at these points should not have been affected by any changes in delineation.

Other stations were operated in the approach and recovery zones 500 to 1,000 feet on either side of the study location. Finally, one or more stations were operated at the entrance, exit or center (whichever was most feasible) of each specific study loca-

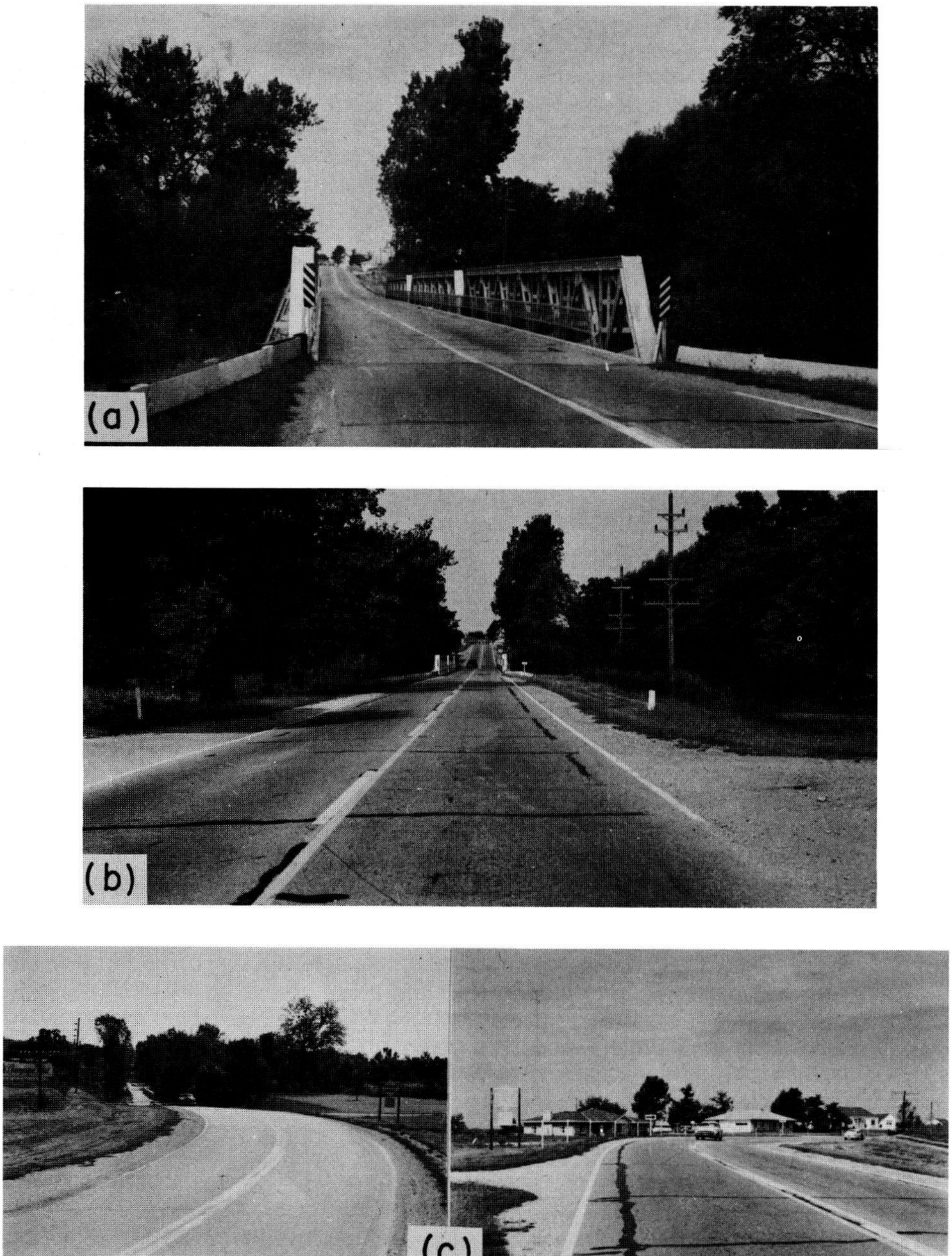

Figure 3. Location I: (a) entrance to bridge from south, (b) approach to bridge from south, and (c) curve north of bridge.

tion. These were grouped together for the analysis and are referred to as "critical" stations.

In consideration of a possible future illumination study, stations in the approach and recovery zones were situated so as to coincide with the probably limits of future illumination.

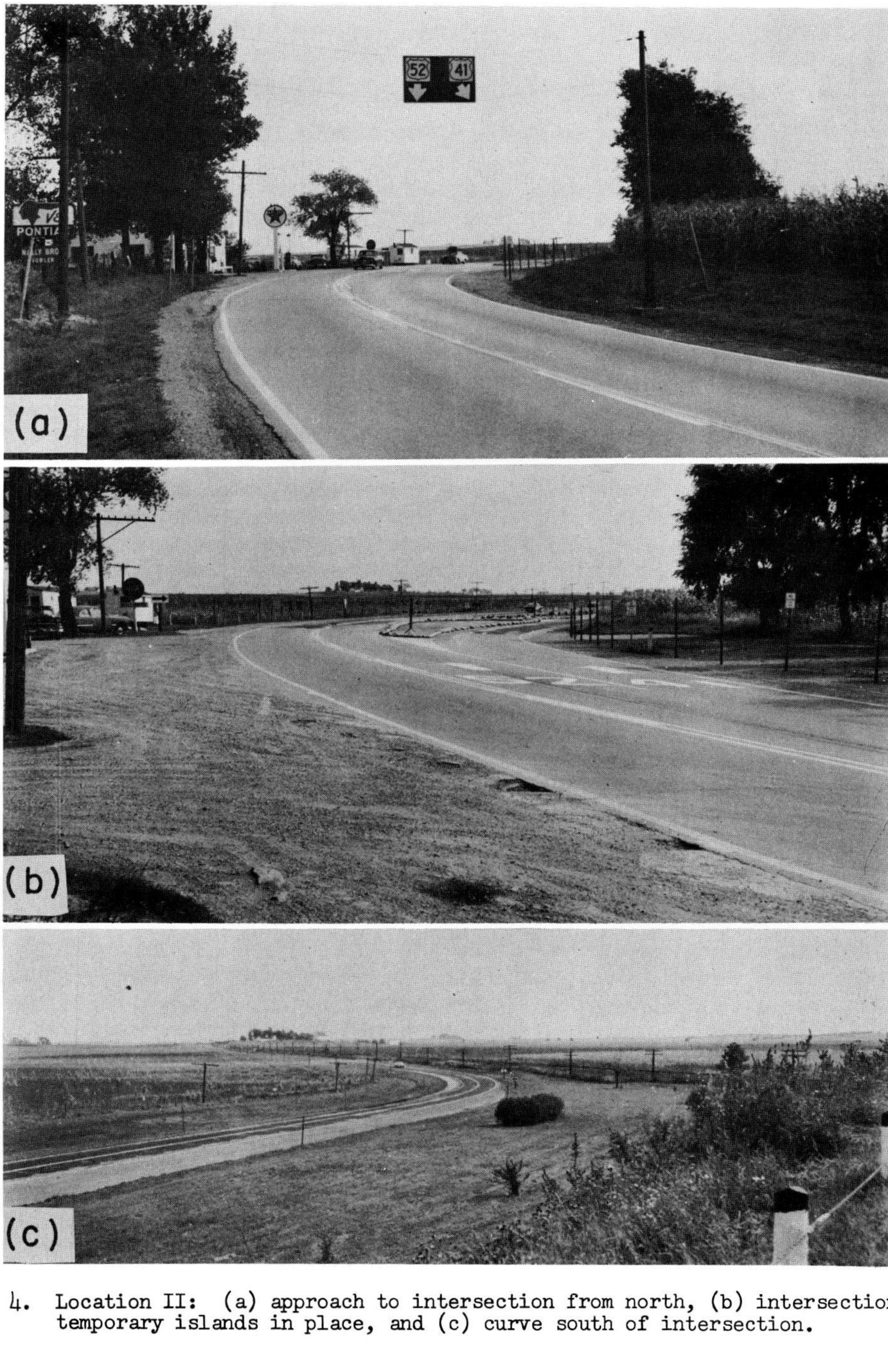

Figure 4. Location II: (a) approach to intersection from north, (b) intersection with temporary islands in place, and (c) curve south of intersection.

STUDY LOCATIONS

Figure 1 shows the outlines of the three study locations in relation to Indiana's highway system. Aerial views of the three study locations are shown in Figure 2. The lower case letters identify the speed measurement stations and the arrows indicate the directions in which speeds were measured.

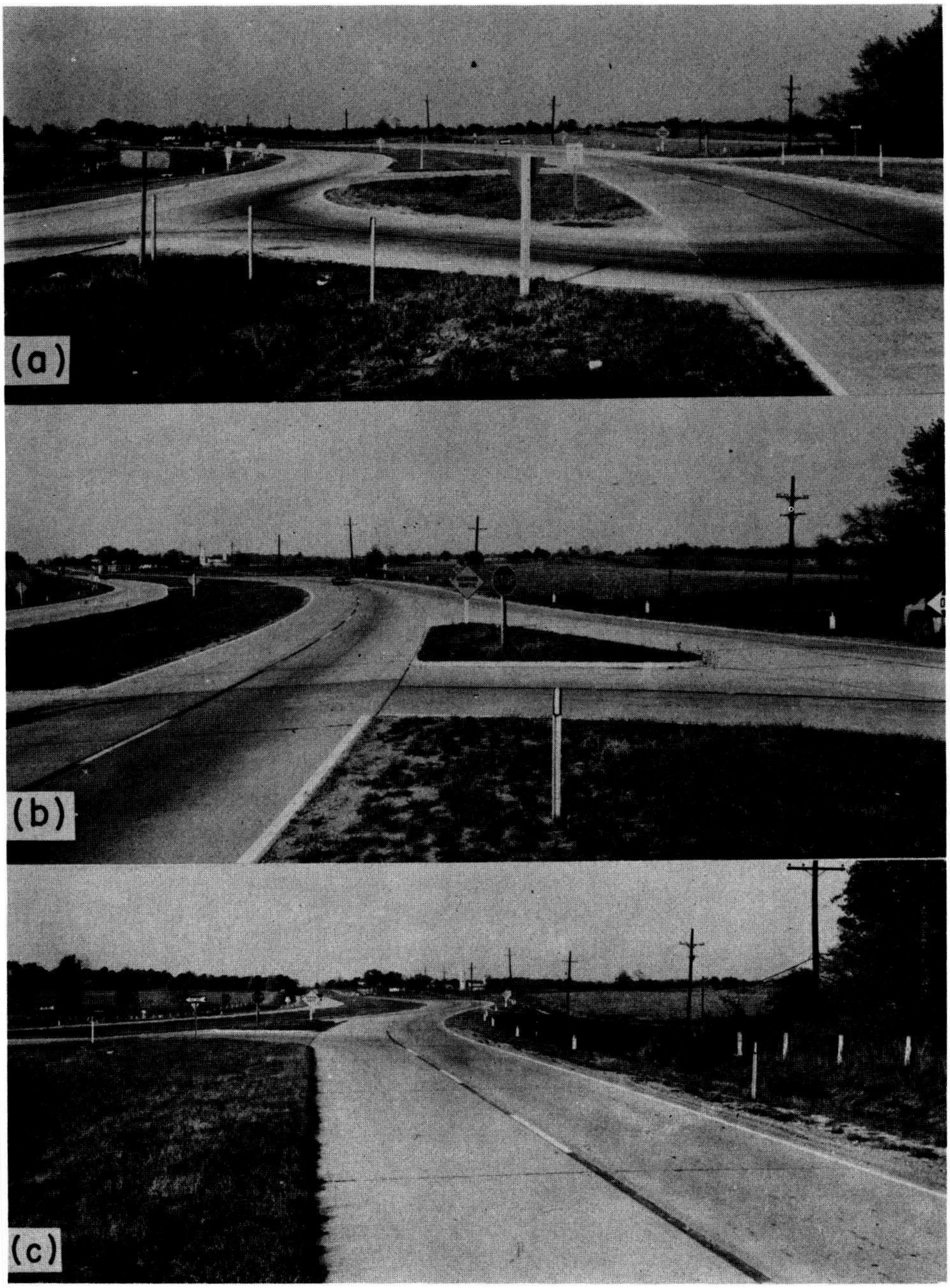

Figure 5. Location III: (a) center of intersection, (b) merging area for traffic traveling north, and (c) approach to intersection on State Route 431.

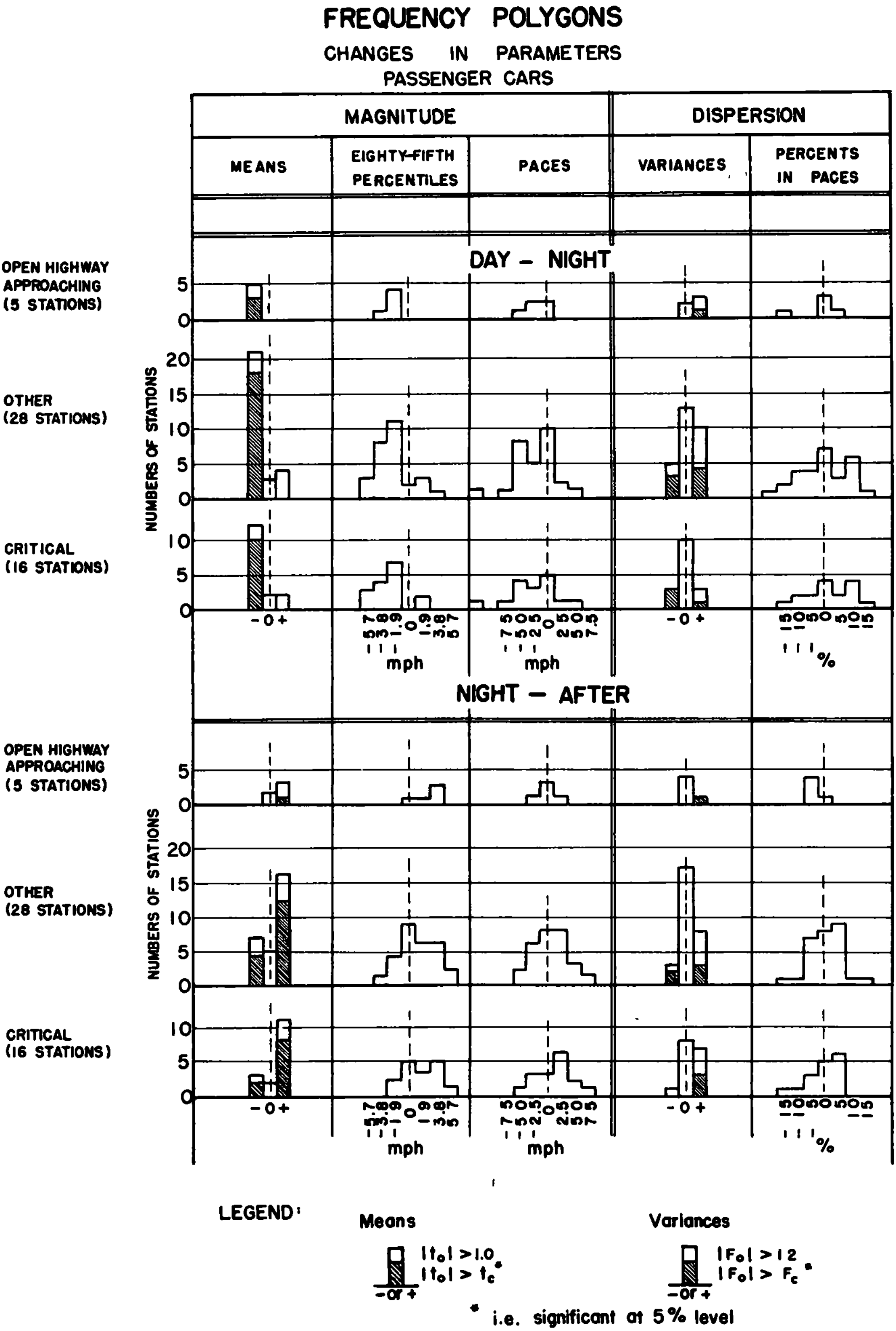

Figure 6. Frequency diagrams for passenger cars.

Location I

Location I is a narrow bridge on a 2-lane section of US 31. The bridge is 256 feet long and 3 feet narrower than the adjacent roadway. The size of the trusses and their proximity to the roadway accentuate the feeling of constriction. Beyond the bridge in one direction is a 5 percent grade at the crest of which is a horizontal curve. The new delineation installed at this location consisted of edge lines throughout the area and supplemental delineators on the curve. No-passing barrier lines on the bridge approaches were also extended and the bridge centerline and curbs were painted yellow. Figure 3 is a view of the approaches to the bridge.

Location II

Location II is the southern junction of US 41 and 52, both of which are 2-lane highways in this area. Both routes are relatively straight in alinement in advance of the intersection, but at this point US 52 makes a semicircular arc so as to connect in a Y intersection with US 41 as it descends from a railroad overpass. Travel through this section is complicated by the presence of fairly sharp curves to the north and south and at the intersection itself. Further hazard arises from the fact that the two roadways merge at a very small angle resulting in a wide expanse of paved area at the junction. Because of this large open area and poor sight distance on the inside of the curve at the intersection, southbound drivers had difficulty in locating their proper route.

Edge lines were placed throughout this area and delineators were placed on the curves and on the approaches to the intersection. Directional and warning signing was also improved. Figure 4 (b) is a view, facing south, of the center of the intersection curve showing the channelizing islands which were added at the intersection to reduce the open area and to outline the traffic ways for the two routes.

It was only at Location II that trucks were present in sufficient numbers to permit including them in the study.

Location III

Location III is the junction of US 31, a four-lane divided highway, and State Route 431. US 31 becomes a two-lane highway 800 feet southwest of the intersection. The intersection is a recently constructed, channelized, bulb-type, Y intersection and is considered to be a good design. Although this intersection is also on a curve, the curvature of the through roadways is about 3 and 4 degrees and sight distances are adequate. Delineation changes at this location consisted primarily of the addition of edge lines and delineators. Figure 5 shows three views of this intersection.

ANALYSIS

For each station by direction of travel and condition of delineation, the mean speed and the variance of the spot speed distribution were calculated.

Statistical tests of significance were performed on the observed changes in mean speeds and variances at each station between phases of the study. Frequency polygons for qualitative observation were also constructed for stations in different types of zones. These frequency polygons for the speed parameters of passenger cars are shown in Figure 6. The classes for the means and variances labeled -, 0, + indicate a decrease, no change, or increase, respectively. The shaded portions for the means and variances denote the number of stations where the statistical test indicated evidence of an actual change in the parameter at the five percent level of significance. The unshaded portions of the - and + classes indicate the number of stations where the absolute value of the t statistic used in the test on the difference of means was 1.00 or more and the absolute value of the F statistic used in testing the variance was 1.2 or more. This was incorporated into the analysis as a further aid in spotting any trends. The criteria chosen correspond to a lower level of significance (roughly 30 percent).

The observed t statistics used in testing the differences between means were also regarded as random variables and the average for each of the types or combinations of zones calculated. The first series of tests along these lines was designed to determine, to a specified probability, whether $\bar{t}_0$ (the mean t statistic for all stations in a zone class) differed from zero for the individual zone; that is, to determine whether there was any statistical significance to the apparent changes in mean speeds for each zone between conditions of the study. (A second series of tests along these lines is noted in the section on "Results of Speed Study".)

The 85th percentile speeds, pace and percent in pace were also graphically illustrated by frequency polygons (Fig. 6). In Figure 6 the changes from the day to night-before and night-before to night-after conditions are grouped on the basis of the change experienced. For example, a change in the pace from 36-46 mph to 34-44 mph was a change of minus 2 mph in the pace and would have been included as one of the stations plotted under the minus 2.5 grouping. Similarly a change in the percent in the pace from 55 to 65 percent was recorded as a plus ten percent change and would be plotted under plus ten percent, while an increase in the 85th percentile speed at a station from 46.0 to 46.3 mph, a plus 0.3 mph, would have placed that station in the zero-mph change group. All groups are identified by the midpoint value of the change for that group. These polygons were prepared to determine the character of the changes in these characteristics for each of the phases of the study.

Figure 6 shows, by number of stations, the frequency polygons of changes which occurred in the parameters for passenger cars from day to night, and from night-before to night-after, at stations in the open-highway approach zones; all other stations (the approach, recovery and critical zone stations) and the critical zone stations alone.

Speed profiles of mean speeds at each location were also drawn and are shown in Figures 7, 8, 9, and 10.

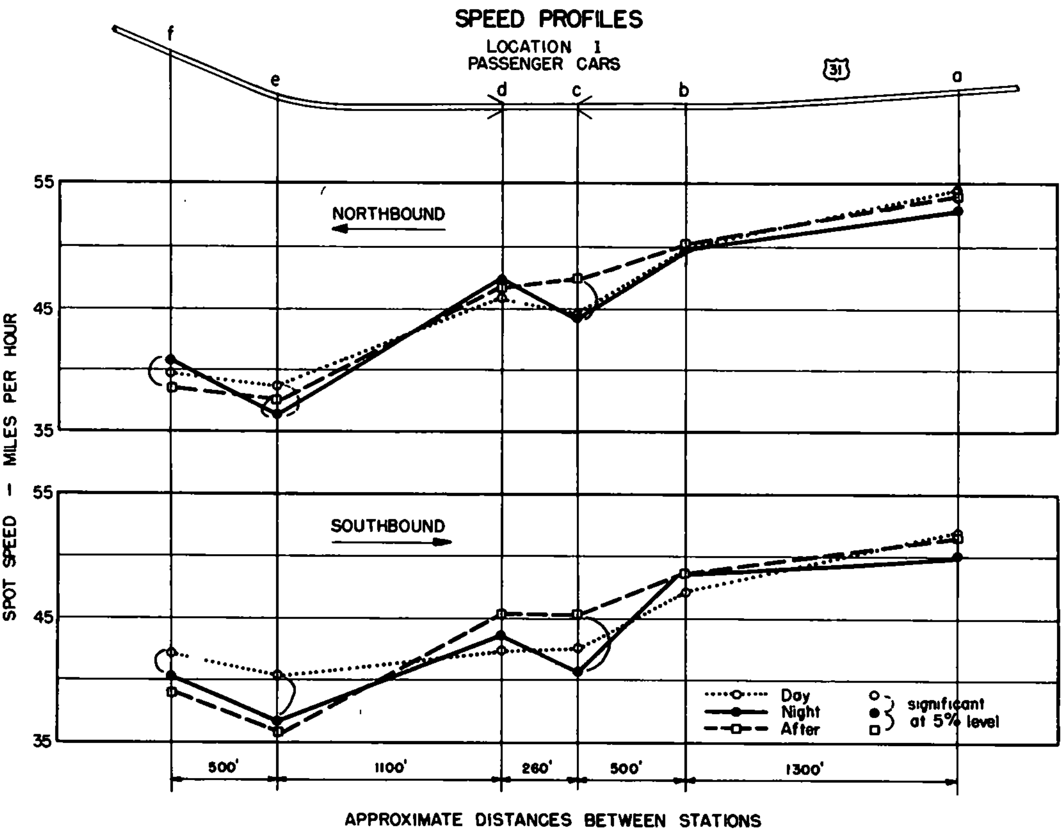

Figure 7. Speed profiles — Location I, passenger cars.

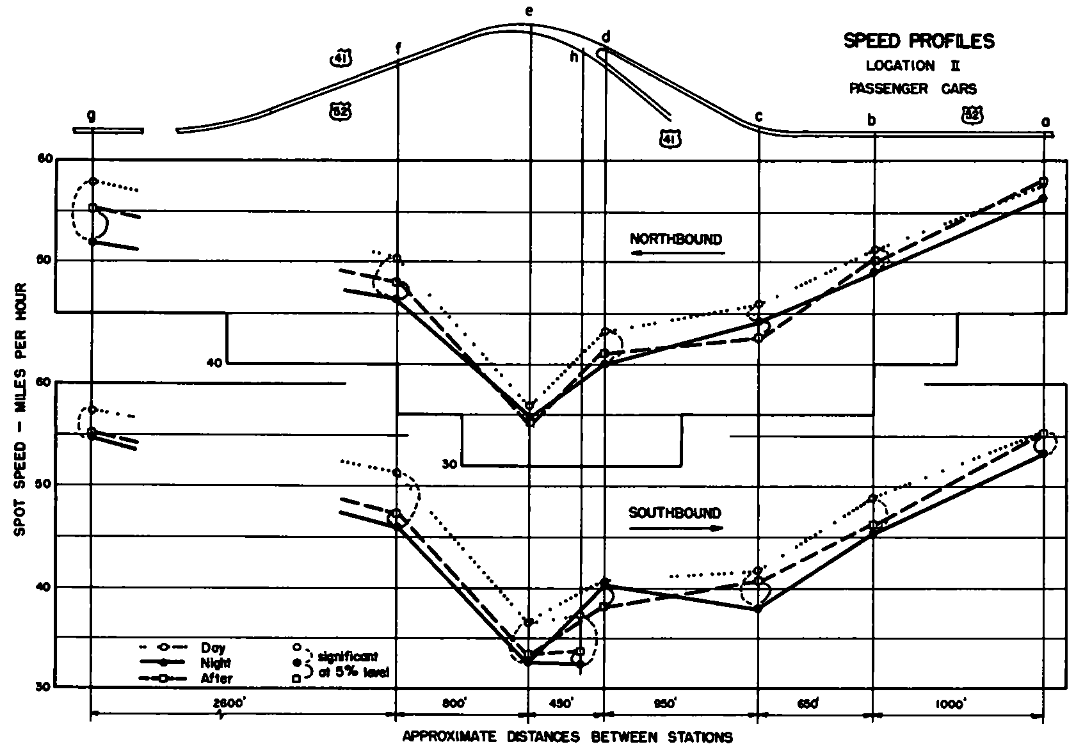

Figure 8. Speed profiles — Location II, passenger cars.

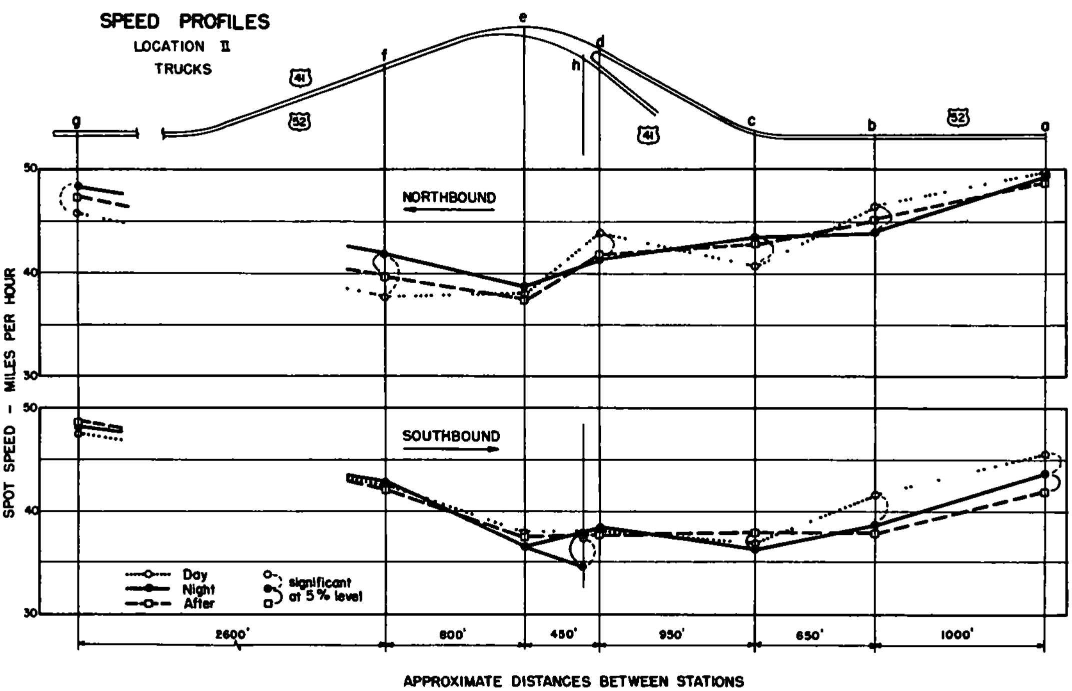

Figure 9. Speed profiles — Location II, trucks.

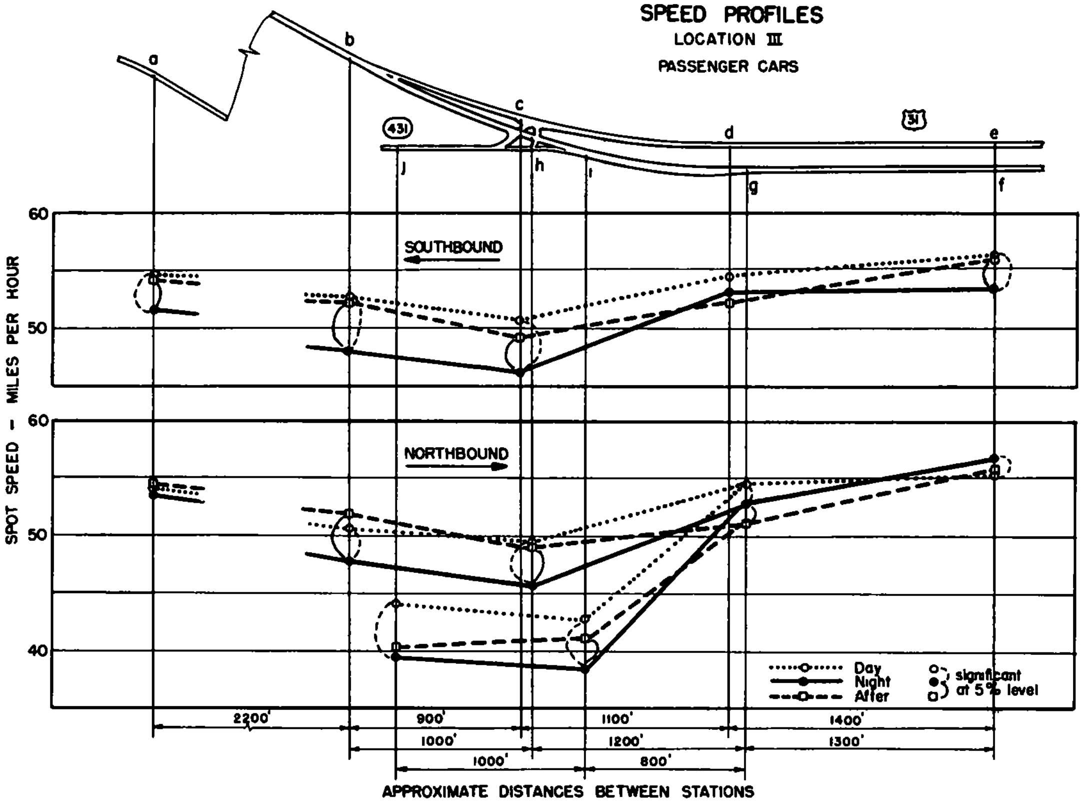

Figure 10. Speed profiles — Location III, passenger cars.

RESULTS OF SPEED STUDY

Inspection of the frequency polygons indicated that mean speeds of passenger cars tended to be less at night than during the day, with the decreases ranging from 1 to 5 mph. Statistical tests on the zones indicated that, at the five percent level of significance, there was sufficient evidence to conclude that night mean speeds were lower than day mean speeds for all zones except the recovery and open-highway-leaving zones. (The test on the latter was significant at the ten percent level).

The effect of darkness on truck speeds, however, was not consistent, with increases as well as decreases observed.

With the added delineation in place, mean speeds of passenger cars showed a tendency to increase slightly at the critical zone but were inconsistent in the other zones. Statistical tests on the zones yielded sufficient evidence at the five percent level to conclude that mean speeds were slightly higher with the added delineation than without it for all zones except the approach and recovery zones.

Truck mean speeds were not affected by the added delineation.

There also did not appear to be any significant effect on the variances of spot speed distribution due to the day, night, and added delineation factors. The results of statistical tests on the variance of the combined truck and passenger car spot speed distributions at the location where both trucks and cars were studied also showed no significant effect.

Observation of the frequency polygons indicated a slight decrease in the 85th percentile speed and a slight decrease in the pace for the day to night-before condition and a slight increase in the 85th percentile speed for the night-before to night-after condition. Observation also indicated no important changes in the pace for the night-before to night-after condition and in the percent in the pace for any conditions.

Inspection of the speed profiles (Fig. 7, 8, 9, and 10) showed no consistent effect of delineation on acceleration or deceleration rates through the locations. In many places, acceleration and deceleration were apparently delayed or dimished in rate after the addition of delineation. However, in those cases where the delineation resulted in a decrease in speed at a critical feature it was because the opposite was true — the deceleration rate, and often the acceleration rate beyond the feature, was greater.

There was evidence, however, of an increase in speed at the open-highway approach zones where the added delineation should not have had an effect. Because this could have been due to a normal increase in speed during the period of the study, it was desired to determine whether there were any differences in degree between the speed changes at each type of zone. Additional statistical analysis was made of these differences, but the results were not illuminating, possibly because of the small differences being tested or the small number of stations in each class of zone.

It is suggested, however, that there may have been a slight change in the speed characteristics of the traffic being sampled between the times that the before-and-after field studies were conducted. This suspicion is strengthened by the results of Shumate and Crowther's study (18). Even if this possibility is discounted, it appears that the additional delineation tended to increase night speeds only slightly at some stations and in an amount that was not of practical significance.

ACCIDENT STUDY

Accident records for the three study locations were studied, for the 20-month period January 1, 1957, to August 31, 1958, prior to the addition of delineation and compared with a similar 20-month period January 1, 1959 to August 31, 1960. These two periods encompass the same variety of seasons.

A summary of the number of accidents for the two 20-month periods is given in Table 1. At Location I, accidents are only shown for the before delineation period as conditions at the location changed during 1960 when construction began on a new bridge to replace the existing narrow bridge. This, of course, is a better solution to this problem than delineation or illumination and was welcomed by everyone.

TABLE 1

COMPARISON OF ACCIDENTS

Location Number	Number of Reported Accidents: 20-month Before Period	Number of Reported Accidents: 20-month After Period
I	11	-[a]
II	23	16
III	8	7

[a]Location removed from study due to construction of new bridge which replaced the hazardous narrow bridge, the subject of study.

Note: 20-month before period—January 1, 1957 to August 31, 1958
20-month after period—January 1, 1959 to August 31, 1960

At Location II there were 23 accidents in the before 20-month period, 9 of them at night. The accidents were mainly concentrated at the northern end of the intersection curve and at the junction. Seventy percent of all the accidents occurred under slippery pavement conditions. These data would seem to indicate that excessive speed for conditions played a large part in accident causation at this location.

In the 20-month period after delineation, 16 accidents occurred, of which eight were at night. Some reduction in accidents occurred in the former large open area, which is now in the "shadow" of the island, and at the northern end of the intersection curve. Fifty percent of the accidents occurred when the pavement was slippery.

Of the nine night accidents in the before period, three resulted from failure to negotiate the north or south curves (under dry pavement conditions). The other six involved vehicles southbound on the intersection curve.

All eight night accidents in the after period occurred at the island channelizing northbound vehicles on US 41 into the intersection. Of these, four were involved in collisions with cross traffic as they attempted to enter the through highway. The other four came to grief at or in advance of the island.

In the before period there were eight accidents at Location III, four of them at night. However, there was no pattern to these accidents of a type susceptible to correction by delineation. Seven accidents occurred in the after period with four of them at night. As in the before period there was no pattern to these accidents.

These accident data were too limited and too nearly similar for the before-and-after periods to permit the drawing of conclusions. They are presented in the hope that the accumulation of evidence of this type in this and other studies will provide an indication of the safety value of delineation.

CONCLUSIONS

With few exceptions, night passenger car average speeds were lower than daytime speeds, most of the differences being in the range of 1 to 5 mph.

With added delineation, night passenger car average speeds showed a tendency to be slightly higher, particularly at critical points such as at the bridge, at the centers of the intersections, or on sharp curves. Such speed increases were probably of little practical significance. The average change for all stations was less than 1½ mph.

The limited data for trucks showed no appreciable or consistent effects on speed due to conditions of visibility or delineation.

There did not appear to be any significant effect on the dispersion of speed due to conditions of visibility or delineation.

The results of this study suggest that the practical effects of delineation do not, at least in the case of the locations studied, manifest themselves in significant changes in speed patterns.

It is anticipated that an accident speed-illumination study will be performed at two of the sites reported on herein. Such a study should prove to be very profitable as it would compare speed patterns and accident rates under conditions of optimum delineation with speed patterns and accident rates under conditions of optimum lighting and delineation (inasmuch as some measure of delineation will be necessary even with illumination). The results of this planned accident-speed-illumination study when compared with the results of the study reported in this paper should provide important information on the relative value of optimum delineation and illumination at hazardous locations.

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Application of Mechanical Analysis to Traffic Recorder Data

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The Oregon State Highway Department in 1958 applied data processing equipment to the analysis of traffic count data obtained from mechanical recorders. The use of mechanical data processing equipment has certain advantages in terms of (a) versatility of the manipulation of the many facets of desired data, (b) ability to accommodate expanding counter coverage with minimal cost increases, and (c) economy derived from efficiency of operation and avoidance of training additional personnel when counter coverage is expanded. The data processing program presently employed, and having been tested quite extensively for the year 1959, provides all of the normal monthly and annual requirements of the Highway Planning Survey Program of the Bureau of Public Roads, and in addition thereto supplies the Department with its basic needs.

The conversion from manual to mechanical analyses also allowed a secondary research project to objectively evaluate the accuracy of ADT estimates. Thirteen of the 92 permanent mechanical recording station installations were selected to provide data for a pilot study to assess the effect of ADT estimates on (a) weekday adjustment factors based on the weekday starting at 12:00 noon versus weekday starting at 12:00 midnight, (b) accuracy of 24-hr versus 48-hr short counts, (c) average error for each weekday in a week, (d) adjustment by weekly rather than monthly seasonal factors, and (e) use of area control versus sectional control estimation procedures.

The results of using the data processing equipment were most gratifying in that the accuracy of the resulting data was enhanced, coupled with the ease of its development and time saved in analysis. Statistical analysis of the 13 stations used in the pilot study indicate that (a) ADT estimates, based on weekdays beginning at noon were significantly more accurate than those based on weekdays beginning at midnight, (b) ADT estimates obtained from 48-hr short counts were typically not significantly more accurate than those obtained from 24-hr short counts, (c) ADT estimates from coverage counts beginning early in the week and from coverage counts ending late in the week were less accurate than those taken from the more central portions of the week, (d) ADT estimates adjusted by weekly seasonal factors were more accurate than those adjusted by monthly seasonal factors, and (e) ADT estimates using the area control method were generally less accurate than ADT estimates using the sectional control method, but the variance was not serious.

● THE TREND in recent years in the highway field has been for the administrators to require more and more factual data on which to base their decisions. Increased knowledge in the use and capacity of transportation systems has required more and more factual data on which to base system actions, establish priorities, plan construction projects, and design the roadway and its appurtenances. Among the more important items generally considered essential for these actions are average daily traffic (ADT), design hour volumes (DHV), and percent commercial vehicles.

The extensive mileage of highways, roads and streets necessitates that traffic volume information, whether it be ADT, DHV, or other information, must be determined from sample studies, as it is physically and financially impractical to count each section of highway, road and street every hour of each day.

It has long been the practice for sample studies to obtain short duration counts at the locations where volume information is desired, and adjust the short duration count information to an ADT. The adjustment factors are normally derived from the relationship of short period counts to long period counts at select control locations. Control location information has generally been obtained from one of two sources, either recorders set for extended periods of not less than a week or from permanent traffic recorders installed for continuous operation.

To increase the accuracy of traffic count information in Oregon, the State Highway Department has, during the last few years, extended the number of control station locations with permanent traffic recorders, so that by the end of 1959 there were 92 permanent automatic traffic recorders in operation throughout the state. At five of these locations, traffic volumes are obtained by direction of travel.

The large number of permanent traffic recorders which record hourly traffic volumes for each day of the year produce almost one million individual traffic counts which must be summarized and analyzed in the course of normal operations. The manual procedure of using adding machines and desk calculators for this purpose required a rather extensive organization.

The rapid increase in the number of permanent traffic recorders created a situation which required an expansion of personnel to handle the summarization of the data. Furthermore, there was a large mass of data available for analysis which was very cumbersome to analyze with manual procedure. It, therefore, appeared that a large portion of the valuable information available from the data obtained from permanent recorders would be lost, and that the personnel of the department would have to be increased unless some mechanical method of processing the data could be devised.

It appeared that the use of electronic data processing equipment would not only facilitate the analysis of the data at hand which previously had not been used with manual procedures, but that it would also allow the summarization of the data quicker, more efficiently, and more accurately. Furthermore, additional permanent recorders would not increase the summarization costs in direct proportion to the amount of data added for summarization.

Recent developments in the recorder field indicate that it will not be very long before punch tape recorders will be practical and commonplace. The conversion of permanent recorder data summarization to mechanical processes lends itself to the use of these punch tape recorders. The combination of these two developments should reduce materially summarization costs.

Other developments in permanent traffic recorder data which have been under investigation, include the transmission of the detection signal to recorders centrally located. Oregon has experimented using a telephone interconnect system, but has found that even though the method is feasible, the cost makes it prohibitive.

It is the intent of this paper to report the findings in Oregon on the conversion from manual to mechanical processes for the summarization of the hourly traffic volume data obtained at permanent traffic recorders. These same procedures were reported to the Committee on Urban Volume Characteristics of the HRB Department of Traffic and Operations at its meeting in January 1959. The report is incorporated in this paper. Also included is an analysis of procedures used for converting coverage counts of short duration to average daily traffic (ADT). The conversion comparisons are based on data obtained from 13 permanent recorder locations selected for the special analysis.

MECHANICAL SUMMARIZATION OF TRAFFIC DATA

The conversion from manual to mechanical summarization of traffic data required initially the development of means of converting the information to the language of the electronic data processing equipment. It was, therefore, necessary to develop a punch card to suit the equipment in use in Oregon. Consideration was given in the development of this punch card to minimize the number of cards required. The final step for conversion to mechanical summarization of traffic volumes was the development of suitable procedures to provide the desired as well as required daily and monthly traffic volume summaries.

The permanent traffic recorders used in Oregon accumulate traffic volumes for an hour, and at the end of that time the cumulative volume and the time of day are printed on a standard adding machine tape (Fig. 1). This printed tape, with minor modifications, is used as the code sheet for punching the tabulating cards. It is necessary to add to the printed tapes a code for the location of the recorder and the time for which the counts were obtained (Fig. 1).

In the regular operations, two cards are necessary to record the 24-hourly volumes recorded for each day. A sample of the hourly volume card is shown at the top of Figure 2. The first card of each pair for a given day contains the AM volumes, and the second card for each pair contains the PM volumes. The segregation of hourly volumes

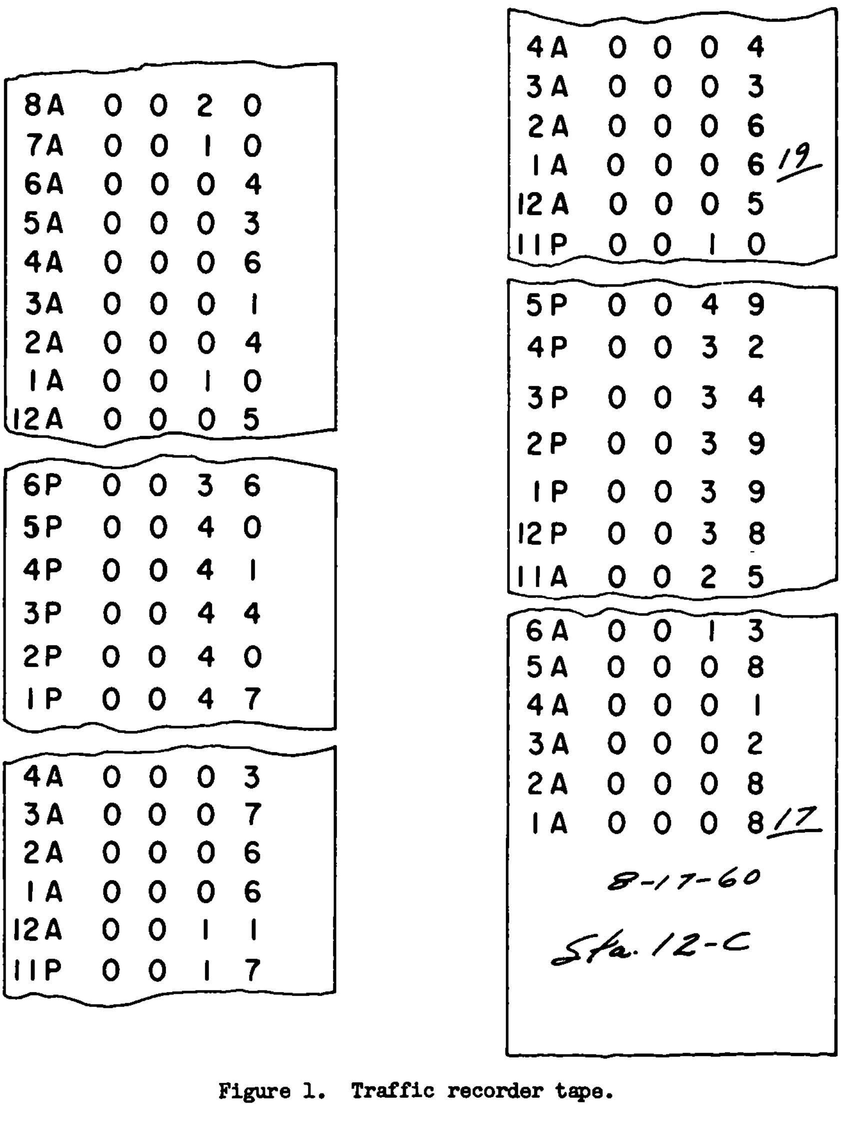

Figure 1. Traffic recorder tape.

by AM and PM resulted from the long practice in Oregon of summarizing daily traffic volumes based on the day beginning at 12 midnight. Recent analysis, described in considerable detail in this paper, indicates that some adjustments may be desirable for the day period for the most efficient operation. If a permanent recorder is operated at a given station every hour of a year, there would be 730 cards for that station in the year period.

The detail cards are used as input cards for the next step of the summarization of the hourly traffic data. An electronic data processing equipment program has been written which uses the permanent recorder detail cards to provide the normally required

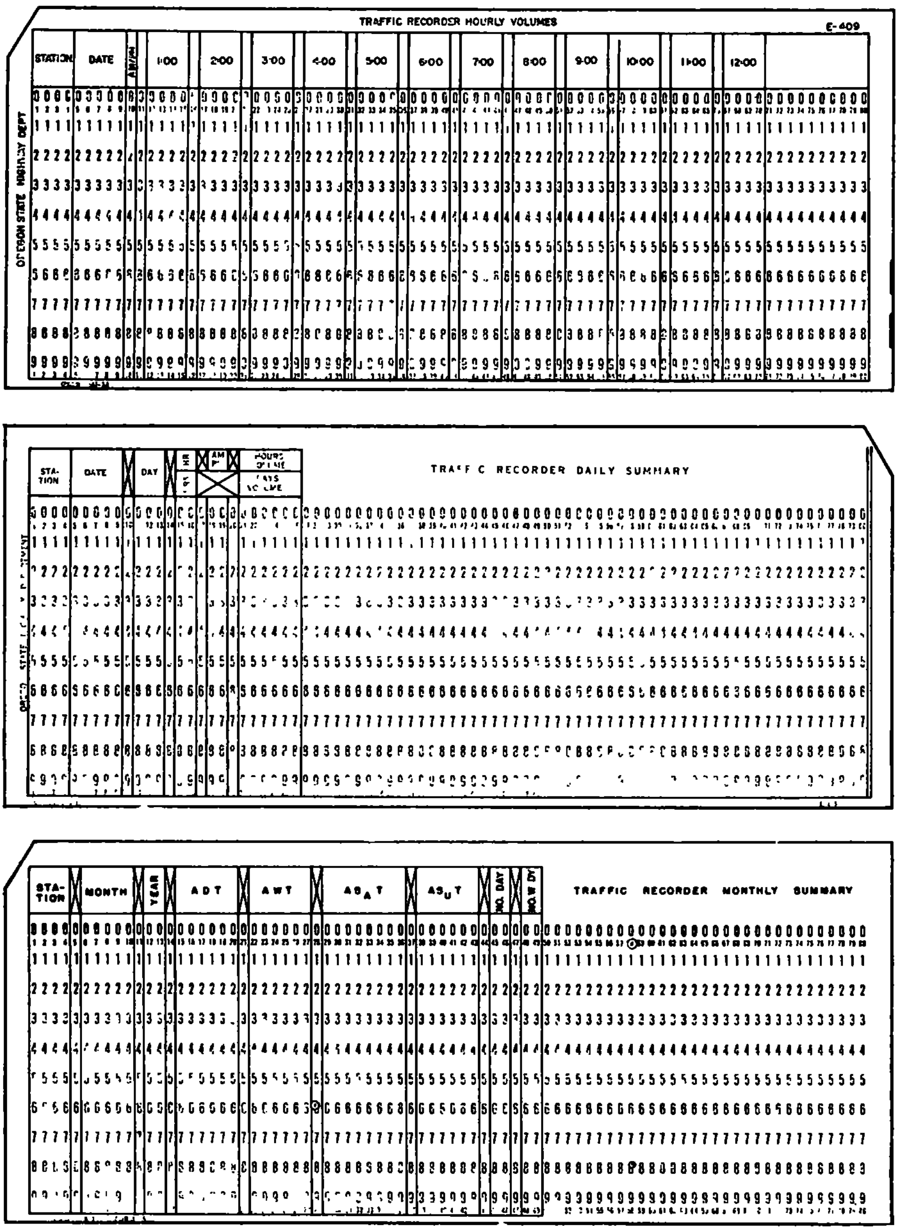

Figure 2. Punch card.

summarizations of the hourly traffic information. The output information is as follows:

1. A daily summary card is punched. This card codes the date, day of the week, number of hours for which there were recorded volumes, and the total volume counted during the day. A sample of this card is shown in the middle of Figure 2. The lower field headings apply to the daily summary.

2. A monthly summary card is prepared. This card codes the month and the year, the average daily traffic for the month, the average weekday traffic, the average Saturday traffic, the average Sunday traffic, and the number of weekdays in the month. A sample of this card is shown at the bottom of Figure 2.

3. Cards containing hourly volumes exceeding a predetermined high critical volume are prepared. These critical volumes are preselected for each permanent recorder, so that the highest 50 hourly volumes during the year will exceed the critical volume. These cards contain the date, the day of the week, the hour, and the traffic volume. (See the middle card in Figure 2.) The upper field headings on the daily summary card apply to the critical hourly volume.

Following the electronic data processing equipment summarization, listings are made for each station from the hourly, daily, and monthly summary cards. Figure 3 is an example of the listing obtained from the hourly cards, and Figure 4 is an example of the listing obtained from the daily and monthly summary cards. These listings are used to analyze the recorder operations to detect obvious machine failures. Following the completion of the review of the recorder operations and modifications or corrections, the corrected data are re-run and revised listings and summaries are prepared.

One way of comparing the mechanical summarization of traffic data to the manual methods is a comparison of the monthly costs. In January 1959, it was estimated that

OREGON STATE HIGH[illegible]Y DEPARTMENT
TRAFFIC ENGINEE[illegible] DIVISION
PLANNING SURVEY SECTION

FORM NO E-411

HOURLY AUTOMATIC TRAFFIC RECORDER DATA

STATION NUMBER	Dir of Travel	Mo.	Day	Yr	A.M. P.M.	12:00-1:00	1:00-2:00	2:00-3:00	3:00-4:00	4:00-5:00	5:00-6:00	6:00-7:00	7:00-8:00	8:00-9:00	9:00-10:00	10:00-11:00	11:00-12:00
26 D	S	9	1	0	AM	226	135	112	83	117	245	1036	1784	956	964	1048	1071
26 D	S	9	1	0	PM	946	1093	1204	1292	1461	1059	794	925	617	519	557	351
26 D	S	9	2	0	AM	209	168	96	88	107	269	984	1762	1051	933	1117	1141
26 D	S	9	2	0	PM	1045	1170	1329	1427	1597	1410	1403	1316	1003	834	754	612
26 D	S	9	3	0	AM	308	275	172	158	147	257	417	675	855	1148	1390	1362
26 D	S	9	3	0	PM	1264	1225	1338	1266	1210	1062	1071	1201	692	523	507	464
26 D	S	9	4	0	AM	471	191	108	72	87	82	211	272	337	566	817	952
26 D	S	9	4	0	PM	1142	1355	1302	1229	1252	1229	1099	1001	811	593	515	337
26 D	S	9	5	0	AM	235	140	91	53	52	113	186	210	334	463	484	850
26 D	S	9	5	0	PM	974	1056	1109	1312	1348	1396	1389	1246	1027	672	505	348
26 D	S	9	6	0	AM	182	143	86	80	126	263	1083	2088	1114	1087	1156	1135
26 D	S	9	6	0	PM	904	1142	1341	1423	1411	1074	994	672	519	406	456	298
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	72	94	273	1020	1961	957	1033	953	1011
26 D	S	9	26	0	PM	[illegible]	[illegible]	[illegible]	[illegible]	1359	998	768	746	[illegible]	466	438	326
26 D	S	9	27	0	AM	172	74	74	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
26 D	S	9	27	0	PM	752	884	1072	1102	[illegible]	1040	686	569	449	353	[illegible]	[illegible]
26 D	S	9	28	0	AM	153	133	95	75	85	214	911	1881	1009	834	979	847
26 D	S	9	28	0	PM	822	874	1024	1119	1253	910	742	650	466	399	364	295
26 D	S	9	29	0	AM	163	110	72	67	116	237	882	1995	932	769	919	793
26 D	S	9	29	0	PM	756	886	1017	1103	1157	994	750	615	493	386	474	316
26 D	S	9	30	0	AM	153	144	80	88	77	240	919	1911	996	823	266	872
26 D	S	9	30	0	PM	848	913	1194	1291	1342	1179	1138	1051	719	636	589	539

Figure 3.

OREGON STATE HIGHWAY DEPARTMENT
TRAFFIC ENGINEERING DIVISION
PLANNING SURVEY SECTION
DAILY AUTOMATIC TRAFFIC RECORDER DATA

STATION NUMBER	MONTH	YEAR	AVERAGE DAILY TRAFFIC	AVERAGE WEEK DAY	AVERAGE SATURDAY	AVERAGE SUNDAY
26DS	SEP	60	17359	17607	17180	16304

DAY OF WEEK	DATE	24 HOUR DAILY TOTAL
SUN	4	16031
SUN	11	16455
SUN	18	16120
SUN	25	16609
MON	5	15883
MON	12	18055
MON	19	17357
MON	26	16716
TUE	6	19173
TUE	13	17111
TUE	20	16659
TUE	27	16030
WED	7	17703
WED	14	16500
WED	21	16449
WED	28	16033
THU	1	18595
THU	8	17493
THU	15	16827
THU	22	16645
THU	29	15793
FRI	2	21825
FRI	9	19660
FRI	16	19289
FRI	23	18961
FRI	30	18608
SAT	3	19097
SAT	10	16878
SAT	17	16464
SAT	24	16281

Figure 4.

it cost approximately $950 per month to summarize traffic volume data obtained at permanent traffic recorders. This cost figure included 280 man-hours to summarize the data, 24 man-hours for final check and review of the data, and 80 man-hours for typing help to transfer the data to permanent records and prepare multiple copies. The use of electronic data processing equipment required 48 man-hours to prepare machine tapes prior to punching, 68 machine hours of punching, processing, and listing of the data, and 24 man-hours for final check and review of the data. For the first six months of 1960, the mechanical summarization of traffic recorder data cost slightly in excess of $700 per month. This indicates that a savings of approximately $250 per month has been realized.

In addition to the cost savings there is, of course, the availability of the punched data for special analysis and study. This additional analysis can be accomplished relatively easily and inexpensively, inasmuch as a large part of the initial cost was incidental to the placing of the data on punch cards. Furthermore, it allows an expansion in the amount of data to be summarized without an increase in cost in direct proportion to the amount of data added.

The procedure of summarizing permanent traffic recorder data once a month, as presently employed in Oregon, has the disadvantage that there is no check on the functioning of the permanent recorder periodically during the month without costly machine processing to review summarized data. Manual procedures did provide a check at weekly intervals. Considerable investigation has been made, but it does not appear feasible to summarize the data by mechanical processes more than twice a month. Each additional summarization will increase the cost in Oregon by approximately $100 per summarization. Therefore, two summarizations per month would cost less than the $950 per month using manual procedures.

There is also the disadvantage that data are occasionally desired for use prior to summarization by mechanical processes. This means that the data are not available when desired, or additional expenditure is required for special summarization by manual procedures. This disadvantage has not caused any serious problems to date; however, it has caused some frustration.

EVALUATION OF TRAFFIC COUNT PROGRAM

The availability of a large mass of permanent traffic recorder data on punch cards presents a fertile field for exploratory analyses and evaluation of presently used and proposed methods of estimating average daily traffic (ADT) from coverage counts. For these analyses, 13 permanent recorder locations of the 92 in Oregon were selected on the basis of their representativeness to provide the basic data for the special analyses. Table 1 presents the summary of the physical data for these 13 recorder locations, and Figure 5 shows the location of the 13 recorders and all other rural permanent recorders in Oregon. The 13 selected permanent recorders provide the basic volume data for the analyses. Seasonal adjustment factors obtained from the same permanent recorders were applied to 24-hr and 48-hr volume information for the purpose of adjusting short period counts (coverage counts) for seasonal variations to obtain average daily traffic (ADT).

This procedure of using the same basic data for coverage counts and for control purposes provides estimates of ADT with a minimum of error. The application of control data to coverage counts obtained at locations other than the control recorder location introduces errors which are not reflected in the analyses contained in this report. Therefore, the results of the following analyses should not be used to represent the normal field condition, but rather should be used to represent the most favorable research condition.

The first area of analysis was the period of time selected to represent a weekday. The normal practice in Oregon has been to consider a day starting at 12 midnight. The development of coverage count seasonal adjustment factors has been as follows: all of the usable 24-hr counts extending from midnight to midnight for the weekdays of a month were averaged to produce an average weekday traffic for the month. This average weekday traffic is compared to the ADT for the same recorder. The resulting ratio with ADT as the numerator becomes the seasonal adjustment factor for that month. The estimate

TABLE 1

SUMMARY OF PHYSICAL DATA FOR SELECT PERMANENT RECORDER LOCATIONS

Recorder			Highway[a]		1959	
No.	Name	Location	Local	Fed. Aid	ADT	Area
11D	Condon	On ORE206 east of Condon	PH	FAS	100	North Central
24T	St. Paul	On ORE219 southeast of St. Paul	SH	FAS	410	Willamette Valley
2E	Kings Valley	On ORE 223 south of Kings Valley	SH	FAS	450	Willamette Valley
24J	Gervais	On county road east of Gervais	Co.	—	530	Willamette Valley
3N	Colton	On ORE211 north of Colton	SH	FAS	540	Willamette Valley
24P	Hubbard	On State Primary Hwy. N. of Hubbard	PH	FAP	790	Willamette Valley
3M	Marquam	On ORE213 north of Marquam	SH	FAS	840	Willamette Valley
9N	Sisters	On US20 north of Sisters	PH	FAP	1,180	Central
10A	Winchester Bay	On US101 south of Reedsport	PH	FAP	2,800	Coast
30D	Pendleton	On US30 (180N) west of Pendleton	PH	FAI	2,950	Eastern
22M	Waterloo	On US20 southeast of Lebanon	PH	FAP	3,490	Willamette Valley
24S	Chemawa	On US99 (15) north of Salem	PH	FAI[b]	9,270	Willamette Valley
20C	Franklin Blvd.	On US99 (15) in Eugene[b]	PH	FAP[b]	22,600	Urban

[a] PH = Primary state highway.
SH = Secondary state highway.
Co. = County road.
FAS = Federal-aid secondary highway.
FAP = Federal-aid primary highway.
FAI = Federal-aid interstate highway.

[b] Traveled route 15.

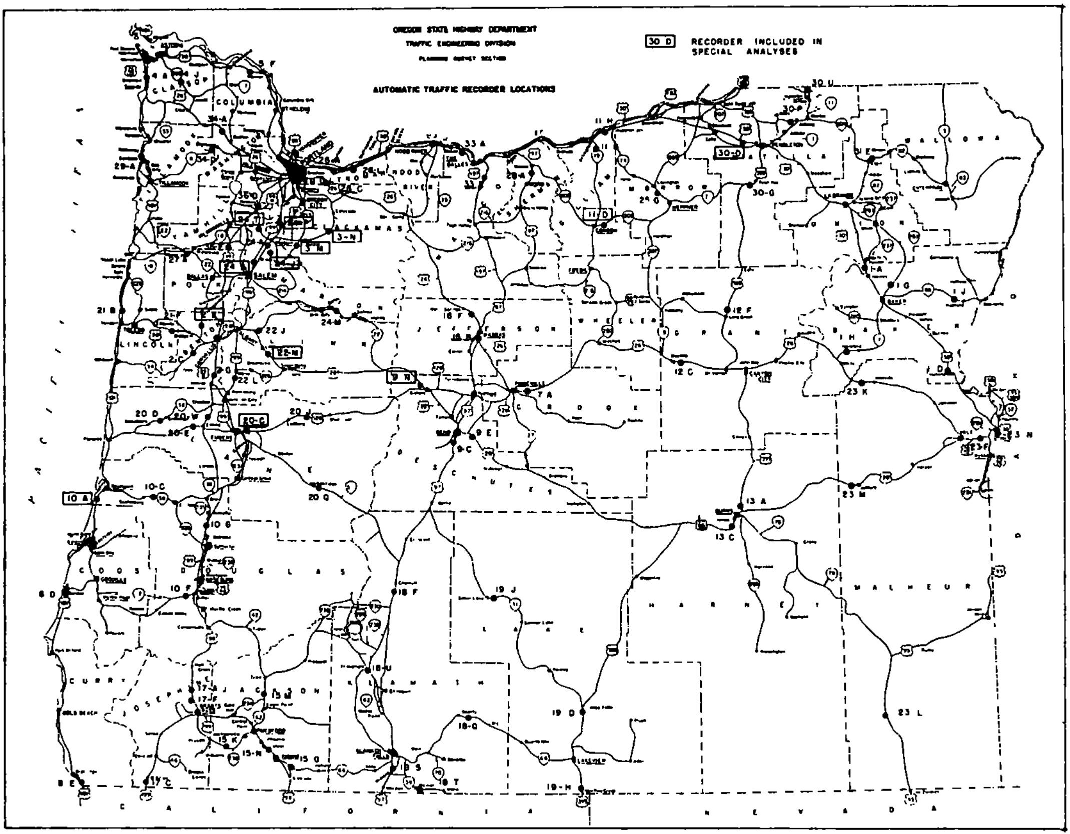

Figure 5.

of ADT is obtained by multiplying any 24-hr weekday volume by the seasonal adjustment factor for the month in which the weekday volume was obtained.

The significant feature of this method of developing seasonal adjustment factors was the use of the 24-hr period extending from midnight to midnight for the weekday. Field practices, however, are such that actual 24-hr coverage counts to be adjusted to an ADT have been more typically obtained for a 24-hr period starting anytime during a day from 8 AM until around 4 PM. This practice results in a weekday averaging from approximately noon to noon. It is important to note that early Monday morning and Friday evening volume information had normally been included in the weekday counts used to develop seasonal adjustment factors; however, they have been omitted from coverage counts obtained for estimating ADT.

The objective of this first area of investigation was to study the development of seasonal adjustment factors based on 24-hr counts for weekdays extending from noon to noon compared to factors based on the normal weekday extending from midnight to midnight. The implication of this comparison becomes obvious considering the probability that the Friday PM volumes are generally rather excessive relative to other similar hours of the week. These volumes tend to inflate the average weekday traffic count, and thus the seasonal adjustment factor would be smaller than it actually should be. The smaller adjustment factors are applied to 24-hr coverage counts which are in reality taken from approximately noon to noon. This procedure would cause a consistent bias towards underestimating ADT.

The second phase of the analysis was occasioned by the variations among the states in specifying minimum coverage count periods. Studies from Minnesota (1), Tennessee (2), Utah (3), and Washington (4), indicate the variability among states in the selection of the minimum coverage count period. In the Washington State article, for example, 24-hr coverage counts were proposed only for roadways with ADT's in excess of 500 in the recommended minimum traffic counting program. In the Minnesota article, counting traffic on low volume roads (less than 1,000 ADT), 24-hr counts were not considered adequate and only 48-hr or consecutive seven-day counts from permanent traffic recorders were analyzed. In the Utah Study, which was based on the study of weekday counts from permanent traffic recorders adjusted by weekly seasonal factors, 48-hr counts were also the rule. The Tennessee Study based on urban traffic recorder data, indicated that weekday 24-hr counts may be assumed to be representative of the ADT, within practical limits. As a result of this contradictory estimate, it was considered desirable to evaluate the accuracy of ADT estimates based on 24-hr as contrasted to 48-hr weekday coverage counts.

The third phase of this analysis was an evaluation of the error of ADT estimates based on the position of the weekday within the week used for the coverage count. Seasonal adjustment factors have been developed in normal practices for each month of the year, based on the premise that the variability of weekday volumes throughout the month would not introduce excessive errors. This procedure has appeared to give satisfactory results in most instances. There are occasions, however, when it appeared that extreme variations in traffic flow from the beginning to the end of the month might well lend itself to substantially increased accuracy by using weekly rather than monthly seasonal factors. This is the fourth phase of this analysis.

The final phase of the special analyses was an evaluation of the various methods of applying seasonal factors to short period coverage counts obtained on sections of roadway removed from any permanent traffic recorder or other control station. The adjustment of coverage counts involves the selection of a control station or a group of control stations which should be used for developing seasonal adjustment factors to produce the best estimate of ADT. Three methods of applying factors are currently in use. They are (a) route control, (b) area control, and (c) section control.

The Route Control Theory (5) implies that many contiguous sections of highways and many non-contiguous highways have the same seasonal traffic patterns. For this initial study, there was an insufficient sample of control stations to investigate the errors in estimating ADT's using route control.

The Area Control Method is based on the assumption that because of economic, climatic, and other conditions, traffic volumes and their seasonal variations are fairly constant within general broad areas. An example of a control area in Oregon would be the Willamette River Valley which extends from the Columbia River on the north to south of Eugene, and from the Cascade Range on the east to the Coastal Range on the west. The average of all monthly factors for all control stations within the control area is used to produce a single seasonal adjustment factor for adjusting coverage counts to estimated ADT. Sufficient control stations were included in these analyses to analyze the area control method for the Willamette River Valley. ADT estimates from area factors are compared to ADT estimates based on section control where a single recorder is selected as being representative to the section of highway for which the coverage count is obtained.

As mentioned earlier in the report, the results of these analyses must be considered as representing the most ideal conditions with minimum errors because of the use of data from the same recorders for assimilating both coverage count and control data.

Midnight vs Noon Seasonal Adjustment Factors

Seasonal adjustment factors in this analysis were based on average weekdays developed in two distinct ways. The first weekday period used for developing seasonal adjustment factors was the midnight-to-midnight weekday. The second weekday period was based on a noon-to-noon weekday which more closely corresponds to actual field practices for obtaining coverage counts. The seasonal adjustment factors developed from these two variations of weekdays will be referred to as "midnight factor" and "noon factor". After the monthly average weekday volume had been obtained by these two methods and respective seasonal adjustment factors computed, they were applied to each of the individual weekday counts in the month. For this analysis, weekday counts used for coverage counts were the noon-to-noon counts.

A summary of the errors resulting from the two methods of estimating ADT is given in Table 2. Errors in this table are reported in two different cases. The median error which is the middle error so that one-half of the errors of estimates are larger than it and one-half smaller than it is shown. Also shown is the mean error which is the absolute average of errors or the arithmetic average error without regard to whether the error is plus or minus. A true arithmetic mean error would be zero for the noon factor and probably quite close to zero for the midnight factors.

A measure of the accuracy of ADT estimates obtained from the midnight and noon factors was based on a statistical test of the significance of the differences in the mean errors for the two methods. The average of the differences in the mean errors was tested by Students "t" test of correlated means. On the basis of these statistical tests,

TABLE 2

ERROR OF ADT ESTIMATES, MIDNIGHT VS NOON FACTORS

Recorder	1959 ADT	Error of ADT Estimates			"t" Ratio	P (Probability)
			Factor			
		Type	Midnight (%)	Noon (%)		
Condon	100	Median	9.3	10.0	-	-
		Mean	12.1	12.2	-0.12	> 0.05
St. Paul	410	Median	6.5	6.7	-	-
		Mean	8.4	8.1	1.65	< 0.05[a]
Kings Valley	450	Median	8.1	7.6	-	-
		Mean	9.0	9.0	-0.19	> 0.05
Gervais	530	Median	5.2	4.9	-	-
		Mean	6.5	6.3	1.29	> 0.05
Colton	540	Median	5.4	5.1	-	-
		Mean	7.2	6.7	2.60	< 0.01[a]
Hubbard	790	Median	4.0	3.8	-	-
		Mean	4.7	4.5	1.49	> 0.05
Marquam	840	Median	5.9	4.5	-	-
		Mean	6.8	5.5	4.26	< 0.01[a]
Sisters	1,180	Median	10.2	7.2	-	-
		Mean	11.9	9.7	3.99	< 0.01[a]
Winchester Bay	2,800	Median	5.4	3.4	-	-
		Mean	6.8	5.1	5.03	< 0.01[a]
Pendleton	2,950	Median	5.6	4.6	-	-
		Mean	6.7	5.7	3.47	< 0.01[a]
Waterloo	3,490	Median	3.0	2.5	-	-
		Mean	3.5	3.0	3.62	< 0.01[a]
Chemawa	9,270	Median	7.2	3.3	-	-
		Mean	8.4	5.0	8.10	< 0.01[a]
Franklin Blvd.	22,600	Median	3.1	1.6	-	-
		Mean	3.3	2.0	8.45	< 0.01[a]

[a]Significant at 5 percent level of confidence.

it was found that all but four of the 13 recorder locations or 70 percent provided more accurate ADT estimates when noon factors were used than when midnight factors were used.

The data obtained in Table 2 have been presented so that recorder locations are in order by ADT. A review of the errors for both midnight and noon factors indicates an inverse relationship of ADT and the magnitude of the error. A rank order correlation of the mean error with ADT yielded a coefficient of -0.87. This indicates strong evidence of a true inverse relationship.

More detailed data in the form of frequency distribution for selected recorders are shown in Figure 6. In this figure the abscissa values show the percent error of ADT estimates. These are absolute percentage errors and do not reflect the degree of over-

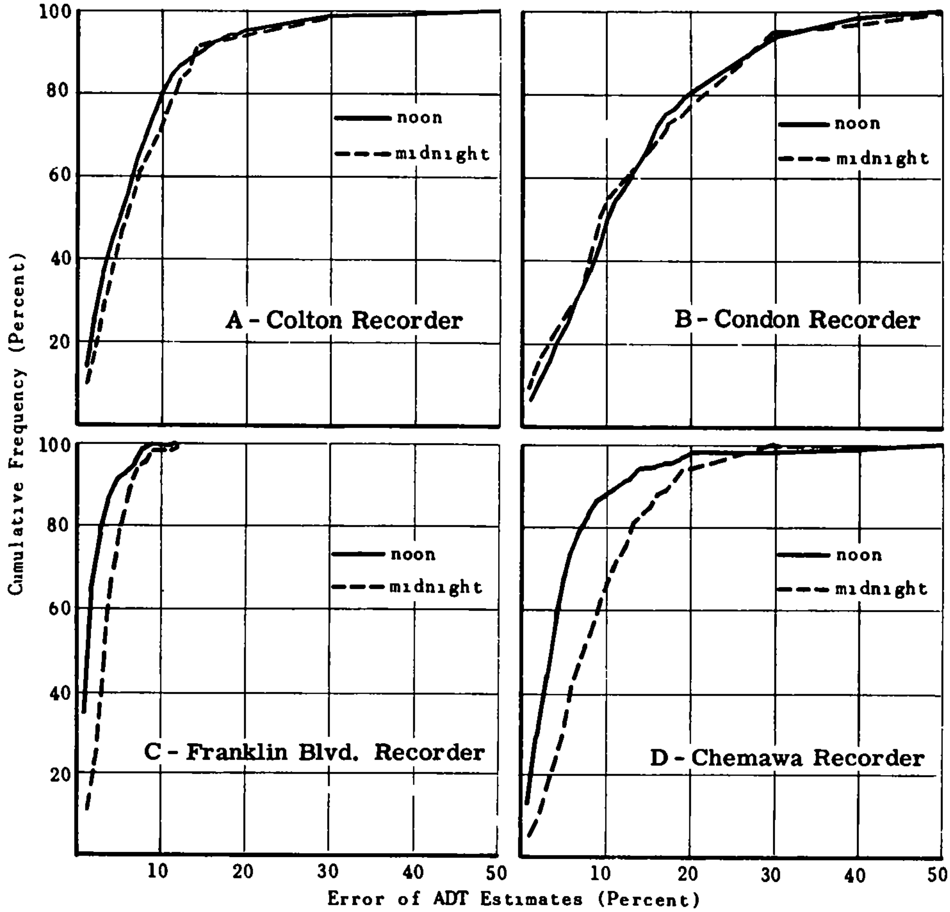

Figure 6. Cumulative frequency distribution of errors of ADT estimates, midnight vs noon factors.

or under-estimation. The ordinate values show the cumulative frequency of ADT estimates obtained during the year.

The data for the Colton recorder (Fig. 6A) were selected as being typical of the majority of the recorders included in the study. From the figure it will be noted that approximately 50 percent of the weekday counts are in error by less than six percent for both midnight and noon factors. It will also be seen that 80 percent of the ADT estimates from noon factors and about 74 percent of the estimates for midnight factors do not over or under-estimate the actual ADT by more than ten percent.

The slightly moderate differences in the accuracy of the estimates based on the midnight factors as contrasted to the noon factors are apparent from Figure 6A. These differences between ADT estimates were analyzed statistically and found to be significantly reliable.

The two recorders which provided the two extremes in accuracy of ADT estimates were used for developing Figure 6B and Figure 6C. The recorder at Condon (Fig. 6B) indicates that 50 percent of the estimates by both midnight and noon factors have errors of more than ten percent or greater. Also evident from Figure 6B is the relatively small differences in errors resulting from estimates by the two methods. The Condon recorder was the only recorder included in the study which showed smaller errors for ADT estimates based on midnight factors compared to estimates based on noon factors. Statistical tests, however, indicated that the differences were not significant. It is interesting to note that the Condon recorder which shows the largest errors in ADT estimates is also the recorder with the smallest ADT. By way of direct contrast, Figure 6C shows the accuracy of ADT estimates for the Franklin Boulevard recorder in Eugene. For this recorder, 50 percent of the estimates have errors of less than four percent when based on midnight factors and less than two percent when based on noon factors. Ninety-eight and 99 percent of the estimates have errors less than 10 percent when based on midnight and noon factors, respectively. Again, it is interesting to note that the Franklin Boulevard recorder which has the smallest errors in estimates of ADT has the largest ADT.

Another pattern of ADT estimates is depicted in Figure 6D. The data for Figure 6D are from the Chemawa recorder. The median error and frequency of error within 10 percent are rather typical. The differences between the accuracy of ADT estimates developed from midnight and noon factors are not typical. The median errors for the midnight factors are slightly more than seven percent, whereas the median errors for the noon factor are only slightly more than three percent. Approximately 67 percent of the ADT estimates from the midnight factor are within 10 percent accuracy. On the other hand, 87 percent of the estimates from the noon factor are within ten percent accuracy.

The differences in errors developed by the two methods were tested statistically and proved to be highly significant as would be expected. There were no other stations which showed such marked differences between the accuracy of ADT estimates based on midnight versus noon factors.

The seasonal adjustment factors for each month are given in each of the 13 recorders in Table 3. The upper entry for each recorder on Table 3 shows the factor based on noon weekdays. The middle figure shows the factor based on midnight weekdays. The lower figure shows the differences in factors expressed in percentage as related to the factor developed from noon weekdays. It will be noted from Table 3 that the noon factors are typically larger than the midnight factors. Careful inspection of this table will show that of 156 pairs of factors, there are only seven which have higher midnight factors, and that five of these seven occur at the Condon recorder.

The predominance of the larger noon factor implies that the omission of the Monday AM and Friday PM volumes from coverage counts used in the application of midnight factors has resulted in underestimation of ADT. The actual amount of underestimation using the midnight factors is reflected in the differences between the factors as given in Table 3.

For the purposes of this phase of the analysis, the factors developed from noon weekdays were considered to be correct inasmuch as the average arithmetic error of ADT estimates for a month period would be zero. This does not mean that there would not be errors in the estimates, but rather that the plus or minus errors for that month will cancel each other.

It will be noted from Table 3 that the difference in factors will almost always be negative indicating an existing underestimation of ADT as estimated from midnight factors. It will also be noted that there are several entries showing underestimation by as much as 10 percent or more. It must be assumed that this underestimation is due entirely to the application of midnight factors to noon weekday coverage counts.

TABLE 3

ADT SEASONAL ADJUSTMENT FACTORS, MIDNIGHT VS NOON FACTORS

Recorder	Factor	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Condon	Noon	1.14	1.30	1.05	0.97	0.95	0.94	0.91	0.85	1.01	0.91	1.13	1.25
	Midnight	1.13	1.24	1.07	0.99	0.96	0.96	0.91	0.86	1.01	0.88	1.09	1.25
	% difference	-0.9	-4.6	1.9	2.1	1.0	2.1	0.0	1.2	0.0	-3.3	-3.5	0.0
St. Paul	Noon	1.44	1.36	1.26	1.22	1.09	0.99	0.84	0.81	1.00	1.15	1.23	1.27
	Midnight	1.41	1.36	1.27	1.22	1.08	0.99	0.81	0.80	0.97	1.11	1.21	1.25
	% difference	-2.1	0.0	0.8	0.0	-0.9	0.0	-3.6	-1.2	-3.0	-3.5	-1.6	-1.6
Kings Valley	Noon	1.39	1.26	1.20	1.02	1.08	0.97	0.92	0.90	0.87	0.90	1.14	1.37
	Midnight	1.34	1.23	1.16	1.00	1.03	0.94	0.90	0.88	0.86	0.87	1.11	1.34
	% difference	-3.6	-2.4	-3.3	-2.0	-4.6	-3.1	-2.2	-2.2	-1.1	-3.3	-2.6	-2.2
Gervais	Noon	1.42	1.35	1.28	1.15	1.29	0.94	0.80	0.78	0.95	0.99	1.06	1.10
	Midnight	1.43	1.31	1.24	1.14	1.24	0.93	0.80	0.78	0.93	0.99	1.03	1.07
	% difference	0.7	-3.0	-3.1	-0.9	-3.9	-1.1	0.0	0.0	-2.1	0.0	-2.8	-2.7
Colton	Noon	1.45	1.47	1.31	1.07	1.01	0.95	0.91	0.87	0.89	1.00	1.00	1.03
	Midnight	1.41	1.44	1.30	1.03	0.96	0.91	0.87	0.83	0.86	0.97	0.98	1.03
	% difference	-2.8	-2.0	-0.8	-3.7	-5.0	-4.2	-4.4	-4.6	-3.4	-3.0	-2.0	0.0
Hubbard	Noon	1.23	1.11	1.08	1.06	1.06	0.95	0.92	0.89	0.93	1.03	1.05	1.05
	Midnight	1.21	1.09	1.05	1.05	1.03	0.93	0.91	0.88	0.93	1.00	1.01	1.04
	% difference	-1.6	-1.8	-2.9	-0.9	-2.8	-2.1	-1.1	-1.1	0.0	-2.9	-3.8	-2.0
Marquam	Noon	1.48	1.37	1.27	1.23	1.16	1.02	0.94	0.96	1.09	1.14	1.17	1.22
	Midnight	1.42	1.33	1.24	1.18	1.09	0.98	0.89	0.93	1.00	1.08	1.11	1.17
	% difference	-4.0	-2.9	-2.4	-4.1	-6.0	-3.9	-5.3	-3.1	-8.2	-5.3	-5.1	-4.1
Sisters	Noon	2.46	2.44	1.81	1.58	1.32	0.90	0.76	0.75	1.01	0.92	1.83	2.05
	Midnight	2.31	2.33	1.72	1.47	1.14	0.84	0.71	0.73	0.90	0.78	1.60	1.94
	% difference	-6.1	-4.5	-5.0	-7.0	-13.6	-6.7	-6.6	-2.7	-10.9	-15.2	-12.6	-5.4
Winchester Bay	Noon	1.52	1.49	1.38	1.31	1.15	0.94	0.75	0.71	1.06	1.29	1.48	1.48
	Midnight	1.44	1.45	1.26	1.26	1.10	0.91	0.72	0.69	0.97	1.22	1.38	1.36
	% difference	-5.3	-2.7	-8.7	-3.8	-4.3	-3.2	-4.0	-2.8	-8.5	-5.4	-6.8	-8.1
Pendleton	Noon	1.64	1.39	1.11	1.11	1.03	0.88	0.87	0.85	0.99	1.00	1.03	1.21
	Midnight	1.58	1.35	1.08	1.10	1.00	0.86	0.84	0.83	0.89	0.95	0.97	1.19
	% difference	-3.6	-2.9	-2.7	-0.9	-2.9	-2.3	-3.4	-2.4	-10.1	-5.0	-5.8	-1.6
Waterloo	Noon	1.21	1.23	1.14	1.02	0.98	0.93	0.87	0.88	0.95	0.96	1.04	1.09
	Midnight	1.20	1.22	1.13	1.00	0.95	0.91	0.85	0.87	0.94	0.93	1.02	1.07
	% difference	-0.8	-0.8	-0.9	-2.0	-3.1	-2.2	-2.3	-1.1	-1.0	-3.1	-1.9	-1.8
Chemawa	Noon	1.43	1.29	1.25	1.24	1.25	1.01	0.96	0.93	1.00	1.19	1.23	1.17
	Midnight	1.34	1.22	1.15	1.18	1.15	0.96	0.91	0.89	0.88	1.11	1.06	1.07
	% difference	-6.3	-5.4	-8.0	-4.8	-8.0	-5.0	-5.2	-4.3	-12.0	-6.7	-13.8	-8.5
Franklin Blvd.	Noon	1.11	1.07	1.06	1.04	1.01	0.94	0.92	0.91	0.96	1.03	1.01	1.01
	Midnight	1.09	1.05	1.00	1.03	0.97	0.92	0.90	0.89	0.94	0.99	0.98	0.99
	% difference	-1.8	-1.9	-5.7	-1.0	-4.0	-2.1	-2.2	-2.2	-2.1	-3.9	-3.0	-2.0

Further review of Table 3 indicates that there is no apparent seasonal trend in the underestimation of ADT developed from midnight factors. This means that the degree of underestimation of ADT from midnight factors is not related to the particular month in which the estimates were obtained.

24-Hr vs 48-Hr Coverage Counts

This phase of the analysis will test the premise established by some states (1, 2, 3, 4) that coverage counts of at least 48-hr duration are necessary to provide the minimum accuracy required for the development of ADT estimates. The need for 48-hr coverage counts appeared to be the most pronounced for low volume roads. The practice in Oregon has been to develop ADT estimates from 24-hr coverage counts. The large portion of Oregon's highways are low volume; therefore, it appeared desirable to check the assumption of greater accuracy from 48-hr coverage counts as contrasted to estimates based on 24-hr coverage counts.

In the first phase of this study, it was found that the seasonal adjustment based on noon weekday factors provided more accurate estimates of ADT than estimates based on midnight weekday factors. Therefore, for this phase of the analysis, noon weekday factors will be used for adjusting for seasonal variations. The estimates of ADT based on 24-hr coverage counts and the resulting errors from these estimates have been developed from the first phase of the analysis. For this phase, then, it was necessary to develop ADT estimates and the resultant errors from 48-hr coverage counts. The coverage count period was obtained by using successive 48-hr periods extending from noon of the first day to noon of the third day. The 48-hr coverage count was then divided by two to provide an average 24-hr coverage count for adjusting to an ADT. It will be noted that in a given week it is possible to obtain only three noon-to-noon 48-hr weekday periods.

Table 4 summarizes the errors of the ADT estimates based on 24- and 48-hr coverage counts. The table includes the median and mean errors of ADT estimates for the 13 recorders selected for the study. The frequency of ADT estimates with errors less than 10 percent are given. The Students "t" test of the significance of the differences in the accuracy of ADT estimates based on 24- and 48-hr coverage counts were performed on the mean error. The "t" ratios and the probability of the difference is given in Table 4. Those recorder locations where significant differences in ADT estimates were noted were indicated by an appropriate footnote. It will be noted that of the 13 recorder locations, all had smaller median and mean errors for 48-hr coverage

TABLE 4

ERROR OF ADT ESTIMATES, 24-HR VS 48-HR COVERAGE COUNTS

Recorder	1959 ADT	Error of Estimate			"t" Ratio	P (Probability)
			Coverage Count (%)			
		Type	24-Hr	48-Hr		
Condon	100	Median	10.2	8.3	-	-
		Mean	12.2	10.0	2.12	<0.05[a]
St. Paul	410	Median	6.7	5.0	-	-
		Mean	8.1	6.6	2.00	<0.05[a]
Kings Valley	450	Median	7.6	6.4	-	-
		Mean	9.0	7.6	1.93	<0.05[a]
Gervais	530	Median	4.9	4.4	-	-
		Mean	6.3	5.6	1.11	>0.05
Colton	540	Median	5.1	3.8	-	-
		Mean	6.7	5.8	1.24	>0.05
Hubbard	790	Median	3.8	2.7	-	-
		Mean	4.5	3.5	2.15	<0.05[a]
Marquam	840	Median	4.5	4.0	-	-
		Mean	5.5	4.6	1.74	<0.05[a]
Sisters	1,180	Median	7.2	7.0	-	-
		Mean	9.7	9.2	0.50	>0.05
Winchester Bay	2,800	Median	3.4	3.0	-	-
		Mean	5.1	4.5	1.11	>0.05
Pendleton	2,950	Median	4.6	4.5	-	-
		Mean	5.7	5.1	1.26	>0.05
Waterloo	3,490	Median	2.5	2.2	-	-
		Mean	3.0	2.4	2.29	<0.05[a]
Chemawa	9,270	Median	3.3	3.4	-	-
		Mean	5.0	4.6	0.59	>0.05
Franklin Blvd	22,600	Median	1.6	1.4	-	-
		Mean	2.0	1.8	1.08	>0.05

[a]Significant difference at 5 percent level of confidence.

counts than for 24-hr coverage counts; however, these differences were statistically significant for only six recorders.

Further study of Table 4 indicates that for roads with traffic volumes less than 1,000 vehicles per day, there is a strong tendency for increased accuracy in ADT estimates developed from 48-hr coverage counts as contrasted to 24-hr coverage counts. No statistical tests have been made for this trait; however, five of the six recorders which had statistically significant differences had an ADT of less than 1,000 vehicles per day.

Figure 7 shows the cumulative frequency of errors of ADT estimates for the recorders at Colton and Condon. A review of this figure will show improvement of ADT estimate by using 48-hr counts as contrasted to using 24-hr coverage counts.

The recorder at Colton had a median error for ADT estimates using 48-hr and 24-hr coverage counts of approximately four and five percent, respectively. About 82 percent of the 48-hr coverage count ADT estimates showed errors of 10 percent or less, whereas only 80 percent of the 24-hr coverage count estimates were 10 percent or less. These differences are not large and subsequent statistical analyses revealed that they are not significantly different. Thus, for the recorder at Colton there was no marked improvement by extending the period of coverage count from 24 to 48 hours.

An inspection of Figure 7B for the Condon recorder indicated that the median error of ADT estimates based on 48-hr and 24-hr coverage counts were approximately eight

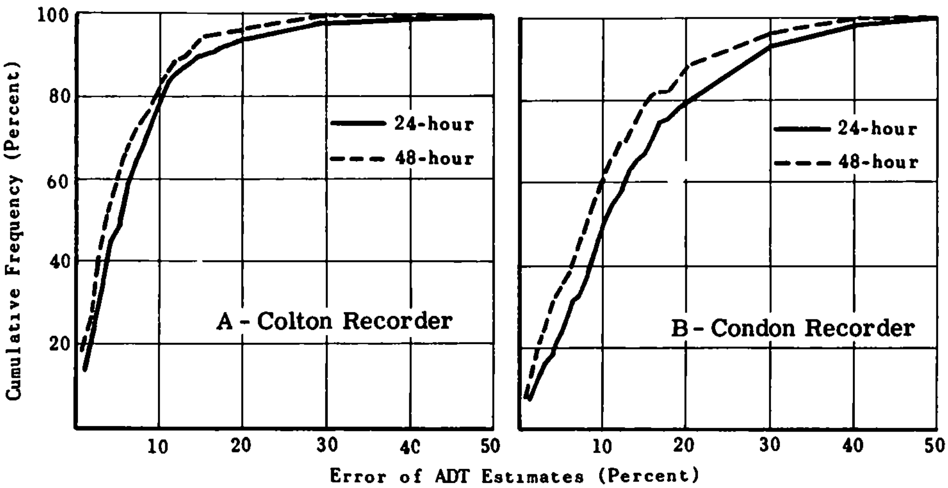

Figure 7. Cumulative frequency distribution of errors of ADT estimates, 24- vs 48-hr factors.

and ten percent, respectively. About 58 percent of the ADT estimates based on 48-hr counts had errors of less than ten percent, whereas only 50 percent based on 24-hr coverage counts had errors of less than ten percent. These differences in the accuracy of ADT estimates were significantly different.

Weekday Position

During the analysis of the second phase of the study, there were patterns which indicated that certain days of the week might tend to provide more accurate estimates of ADT. Therefore, the average error for each day of the week from coverage counts was determined for the 13 recorder locations. This was done for ADT estimates based on both 24-hr and 48-hr coverage counts.

Table 5 contains the error of ADT estimates based on 24-hr coverage counts. This table shows that counts beginning early in the week, Monday, and counts ending late in the week, Friday, were less accurate than those taken from the more central portions of the week, praticularly those beginning on Wednesday and extending to Thursday noon. The least accurate estimates in general are those extending from Thursday noon to Friday noon, and the most accurate estimates are those extending from Tuesday to Wednesday noon or from Wednesday to Thursday noon.

For ADT estimates based on 48-hr coverage counts, a similar analysis was made. It should be remembered that while a 48-hr coverage count may be referred to as a Monday count, it actually involves one-half of Monday, all of Tuesday, and one-half of Wednesday. Because of this extension over three days of the week, it would be assumed that the position of the day within the week effect on ADT estimates would be somewhat obscure. Nevertheless, when these analyses were performed on those six

TABLE 5

ERROR OF ADT ESTIMATES, 24-HR COVERAGE COUNTS BY DAY OF THE WEEK

Recorder	1959 ADT	Mean Error of ADT Estimates			
		Day of Week			
		Mon.	Tues.	Wed.	Thur.
Condon	100	12.9	12.5	12.3	10.7
St. Paul	410	7.9	7.5	7.9	9.4
Kings Valley	450	8.8	8.8	8.6	10.0
Gervais	530	5.6	7.5	5.8	6.3
Colton	540	6.8	6.6	6.0	7.4
Hubbard	790	5.4	5.0	3.4	4.3
Marquam	840	5.6	5.2	5.3	5.8
Sisters	1,180	8.3	10.7	8.5	9.2
Winchester Bay	2,800	5.2	5.4	4.7	5.3
Pendleton	2,950	5.2	5.1	5.2	7.6
Waterloo	3,490	3.3	2.6	2.9	3.5
Chemawa	9,270	5.8	5.0	4.6	4.7
Franklin Blvd.	22,600	2.4	2.0	1.5	2.4
Median		5.6	5.4	5.3	6.3

recorder locations which produce significantly better ADT estimates from 48-hr coverage counts as contrasted to estimates from 24-hr coverage counts, it was found that the estimates based on the Monday PM through Wednesday AM counts were typically less accurate than those estimates from Tuesday PM through Thursday AM and from Wednesday PM through Friday AM. At four of the recorder locations, the greatest accuracy was obtained from those counts beginning Tuesday PM and extending through Thursday AM. No "position of the day in the week" effect was evaluated for the remaining seven recorder locations which did not have significantly greater accuracy of ADT estimates developed from 48-hr coverage counts as compared to 24-hr coverage counts.

Monthly vs Weekly Seasonal Adjustment Factors

The objective of this phase of the analysis was to determine the effect on the accuracy of ADT estimates of adjusting coverage counts for seasonal variation by factors developed for each month as contrasted to factors developed for each week of the year. In the preceding phases of the analysis, coverage counts adjusted to provide ADT estimates have been based on monthly factors. It appeared that the variation in weekday traffic volumes from the beginning to the end of a month might be of sufficient magnitude that substantially increased accuracy could be obtained by developing seasonal adjustment factors for each week in a month. Average weekdays determined for each week would be compared to the ADT to produce weekly seasonal adjustment factors. Before discussing the statistical analysis of this approach, it should be evident to the reader that because the 24-hr coverage count is the lowest common denominator, that if the interval of time used for developing seasonal adjustment factors decreases, the accuracy of the resultant estimates must of necessity increase. To carry this analogy to the extreme, of course, would be to adjust each weekday individually by factors developed for that particular weekday. This would result in absolutely no errors in the ADT estimates. In view of this line of reasoning, it is not unexpected that in many cases the weekly factors produced a rather considerable increase in the accuracy of the ADT estimates.

The summary of the data for the comparison of monthly and weekly seasonal adjustment factors is given in Table 6. Included in the table are the median and mean errors of ADT estimates, the "t" ratio, and the probability. Also given is the frequency of error less than ten percent. It will be noted that in every instance the weekly factor provided smaller errors in the ADT estimates than the monthly factor. As indicated before, this is to be expected. Students "t" tests of the differences of the means were

TABLE 6

ERROR OF ADT ESTIMATES, MONTHLY VS WEEKLY SEASONAL ADJUSTMENT FACTORS

Recorder	1959 ADT	Error of ADT Estimate			"t" Ratio	P[a] (Probability)	Frequency of Error Less than 10%	
			Factor (%)				Factor (%)	
		Type	Monthly	Weekly			Monthly	Weekly
Condon	100	Median	10.0	7.7	-	-	50	66
		Mean	12.2	8.7	5.21	<0.01	-	-
St. Paul	410	Median	6.7	5.1	-	-	67	85
		Mean	8.1	6.2	4.23	<0.01	-	-
Kings Valley	450	Median	7.6	4.9	-	-	62	82
		Mean	8.7	6.0	6.09	<0.01	-	-
Gervais	530	Median	4.9	3.2	-	-	83	95
		Mean	6.4	4.0	5.85	<0.01	-	-
Colton	540	Median	5.1	3.2	-	-	80	95
		Mean	6.7	4.2	5.62	<0.01	-	-
Hubbard	790	Median	3.8	2.8	-	-	93	97
		Mean	4.5	3.6	2.98	<0.01	-	-
Marquam	840	Median	4.5	3.0	-	-	86	95
		Mean	5.5	4.0	4.74	<0.01	-	-
Sisters	1,180	Median	7.2	3.8	-	-	65	89
		Mean	9.4	5.4	7.04	<0.01	-	-
Winchester Bay	2,800	Median	3.4	1.7	-	-	85	99
		Mean	4.7	2.4	6.67	<0.01	-	-
Pendleton	2,950	Median	4.6	2.8	-	-	82	95
		Mean	5.7	3.8	5.34	<0.01	-	-
Waterloo	3,490	Median	2.5	1.7	-	-	98	99
		Mean	3.0	2.2	5.32	<0.01	-	-
Chemawa	9,270	Median	3.3	2.2	-	-	87	97
		Mean	4.7	2.4	7.06	<0.01	-	-
Franklin Blvd.	22,600	Median	1.6	1.4	-	-	99	100
		Mean	1.9	1.3	5.14	<0.01	-	-

[a]Significant differences for all recorders

performed and the results indicated that the increased accuracy in ADT estimates was an actual increase and not due to chance. Although the monthly factor provided estimates of ADT with a mean error of less than ten percent in all but one case, considerable improvement was normally experienced through the use of weekly factors. The range in improvement varied from approximately 25 percent reduction in error to almost 100 percent reduction in errors using weekly factors rather than monthly factors.

The frequency of errors less than ten percent varied for the monthly factor from 50 to 99 percent, with only three recorders indicating a frequency of errors less than ten percent more than 90 percent of the time. The weekly factor, on the other hand, had errors less than ten percent varying from 66 to 100 percent of the time. At nine of the 13 recorders, the frequency of errors less than 10 percent was more than 90 percent, and in only one instance was it below 80 percent.

The cumulative frequency distribution for four selected recorders is shown in Figure 8. For the Colton recorder, the median error using weekly factors was approximately three percent, whereas it was five percent using monthly factors. Also, 80 percent of the ADT estimates based on monthly factors were less than ten percent in error, whereas approximately 93 percent of the ADT estimates based on weekly factors were within ten percent. The data for the other recorders depict about the same

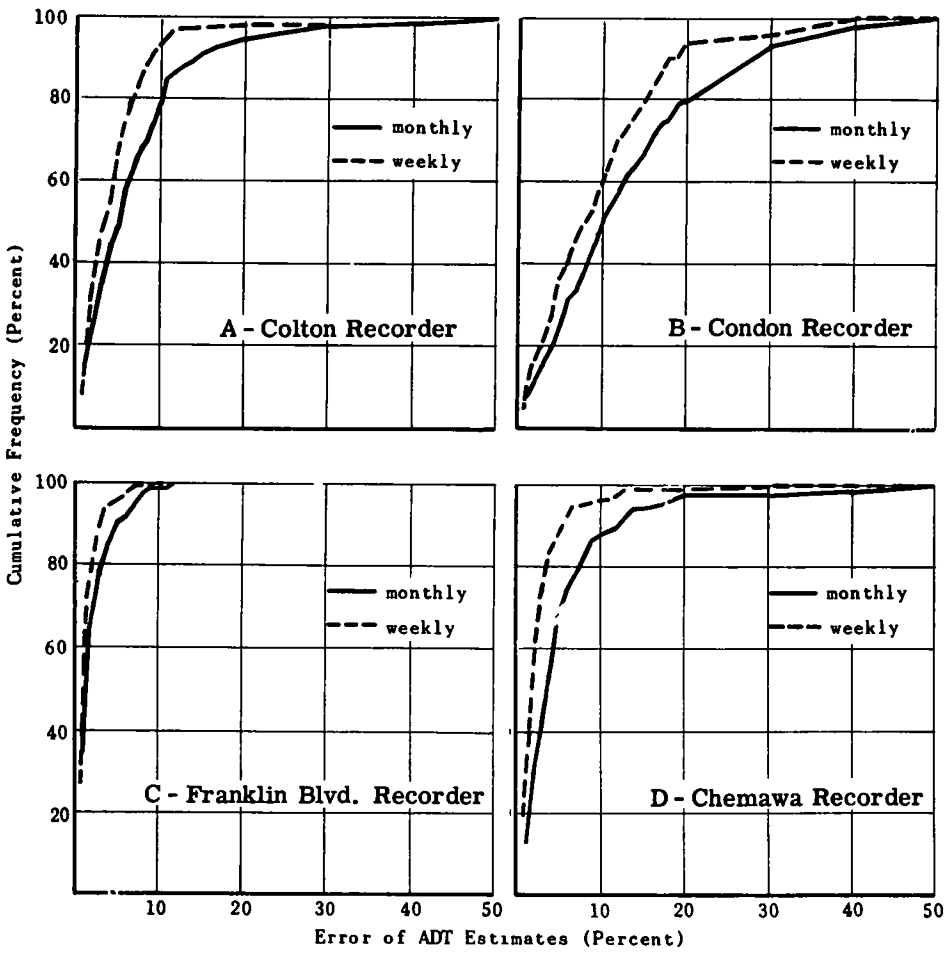

Figure 8. Cumulative frequency distribution of errors of ADT estimates, monthly vs weekly factors.

situation. It is interesting to note that for the Franklin Boulevard recorder the monthly factor had a mean error of ADT estimate of 1.9 percent and a frequency of error less than ten percent 99 percent of the time. The weekly factor improved the mean error to 1.3 percent and the frequency of error less than ten percent included all estimates.

Seasonal variation, travel habits, and the effect on these travel habits from outside sources, suggest that the improvement in ADT estimates obtained by using weekly factors rather than monthly factors might be considerably more in some months of the year than in others. One month which might be expected to cause substantial changes would be September during which most schools resumed after their summer vacations. In general, in Oregon this occurred near the middle of the month and is normally synonymous with the end of the vacation season. It would be expected, then, that considerably more vacation travel would occur during the first week of the month than in the last week of the month resulting in considerably increased accuracy in ADT estimates by using weekly rather than monthly factors.

Table 7 gives the mean error of ADT estimates for each month of the year based on both monthly and weekly factors. A review of Table 7 discloses only one or two recorders which appear to have any seasonal trend in the increased accuracy resulting from the use of the weekly rather than the monthly factors. The Condon recorder, for example, shows a substantial increase in accuracy in the month of August, whereas the increase in accuracy in the other months remains fairly constant throughout the year. The Sisters recorder had abnormally large errors for both the monthly and the weekly factors for October. These errors were caused by the large influx of deer hunters moving into the Sisters area early in October. Although a review of the table indicates that the mean error of ADT estimates varies from month to month, the

TABLE 7

ERROR OF ADT ESTIMATES, MONTHLY VS WEEKLY FACTORS

Recorder	1959 ADT	Mean Error of ADT Estimates (%)												
		Factor	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug	Sept	Oct	Nov	Dec.
Condon	100	Monthly	13.4	14.8	7.4	12.0	9.9	12.8	11.5	21.6	11.2	8.1	13.2	11.2
		Weekly	-	10.2	6.8	9.4	8.7	9.4	9.0	7.6	10.6	6.6	9.1	8.4
St. Paul	410	Monthly	11.6	9.7	4.1	7.4	6.7	5.4	9.0	11.3	13.5	6.5	6.7	7.8
		Weekly	8.4	7.1	3.4	5.9	6.1	4.8	5.4	9.0	7.6	5.8	4.8	5.0
Kings Valley	450	Monthly	14.4	8.7	7.9	10.9	10.7	7.5	5.0	7.1	8.5	11.7	9.7	7.2
		Weekly	5.7	5.8	6.6	6.4	6.7	6.2	4.6	5.2	9.6	4.8	5.5	2.8
Gervais	530	Monthly	9.0	5.1	4.1	5.5	17.3	7.2	5.2	4.8	8.9	5.2	3.9	4.9
		Weekly	6.8	4.4	3.4	3.8	4.1	4.5	3.8	2.4	3.9	4.1	3.6	3.6
Colton	540	Monthly	15.7	13.0	7.0	5.7	5.5	3.5	6.0	3.7	6.5	4.7	3.6	5.6
		Weekly	9.8	3.3	5.1	3.7	4.6	3.7	3.0	3.1	3.9	4.2	2.8	2.3
Hubbard	790	Monthly	9.9	3.0	3.9	4.3	4.5	4.2	3.8	4.0	3.3	3.4	4.1	6.8
		Weekly	9.6	2.7	3.9	2.9	3.9	3.8	2.7	2.9	2.4	2.9	3.0	2.3
Marquam	840	Monthly	10.3	6.0	5.4	6.0	6.4	4.2	4.7	3.9	6.0	4.2	3.8	5.3
		Weekly	7.6	4.1	2.3	5.3	4.8	2.4	3.9	3.3	3.6	4.0	2.9	3.6
Sisters	1,180	Monthly	8.0	11.3	9.7	9.6	4.6	4.9	7.2	6.8	10.5	25.6	8.4	10.3
		Weekly	2.9	7.4	4.6	4.8	3.8	5.3	4.9	4.8	8.2	13.8	4.0	3.6
Winchester Bay	2,800	Monthly	2.9	3.0	4.5	1.9	4.2	7.0	2.3	4.2	12.9	3.0	7.9	8.4
		Weekly	2.8	1.4	2.8	1.5	2.6	3.3	2.2	2.6	1.6	2.2	3.9	2.2
Pendleton	2,950	Monthly	7.1	10.5	5.1	2.7	5.2	3.6	4.4	5.2	6.4	9.0	4.7	4.6
		Weekly	4.8	4.1	3.1	2.0	3.8	2.9	3.9	2.4	4.6	6.9	3.8	4.7
Waterloo	3,490	Monthly	3.5	4.6	3.5	2.5	3.7	1.7	2.6	2.7	3.1	3.3	2.6	3.0
		Weekly	2.3	2.5	1.9	2.1	3.0	1.5	2.1	2.2	2.0	2.5	1.8	2.2
Chemawa	9,270	Monthly	6.8	3.8	4.3	1.6	2.3	4.0	6.4	3.4	9.0	2.1	3.8	11.5
		Weekly	4.1	3.1	1.4	1.6	1.8	3.6	3.0	1.4	1.1	0.6	3.1	2.1
Franklin Blvd.	22,600	Monthly	1.8	1.8	2.2	1.5	1.9	1.9	1.8	1.0	2.7	3.7	1.4	2.3
		Weekly	1.8	1.4	0.8	1.4	1.6	1.6	1.3	0.8	1.7	1.3	0.9	0.8
Median		Monthly	9.0	6.0	4.5	5.5	5.2	4.2	5.0	4.2	8.9	4.7	4.1	6.8
		Weekly	4.8	4.1	3.4	3.7	3.9	3.7	3.8	2.9	3.9	4.1	3.6	2.8

relationship between errors based on monthly and weekly factors generally tends to remain fairly constant, and there is no evidence of any seasonal or monthly period which provides abnormal increased accuracy.

Control Methods

The final phase of this analysis was included to attempt to show realistic estimates of errors which could be expected in adjusting coverage counts to an ADT using data obtained for control locations. Of the three methods of control location application, namely, route, area, and section control, it was possible to compare only section and area control, inasmuch as there were insufficient locations with similar traffic patterns to study route control.

TABLE 8

ERROR OF ADT ESTIMATES, SECTION VS AREA FACTORS

Recorder	1959 ADT	Error of ADT Estimate: Type	Factor (%): Section	Factor (%): Area	"t" Ratio	P[a] (Probability)
St. Paul	410	Median	6.7	9.7	-	-
		Mean	8.1	10.0	4.72	<0.1
Kings Valley	450	Median	7.6	10.1	-	-
		Mean	9.0	10.6	3.41	<0.1
Gervais	530	Median	4.9	7.4	-	-
		Mean	6.3	7.8	4.06	<0.1
Colton	540	Median	5.1	7.8	-	-
		Mean	6.7	8.8	4.83	<0.1
Hubbard	790	Median	3.8	7.1	-	-
		Mean	4.5	8.1	8.20	<0.1
Marquam	840	Median	4.5	10.2	-	-
		Mean	5.5	9.6	8.68	<0.1
Waterloo	3,490	Median	2.5	6.0	-	-
		Mean	3.0	5.9	8.92	<0.1

[a]Significant differences for all recorders.

The estimation of ADT's from section control requires the selection of a single control location to provide seasonal adjustment factors for adjusting select coverage counts. The area control method of ADT estimation is based on grouping control location data and applying the seasonal adjustment factors developed from the grouped data to coverage counts obtained in the same general area. For this study, seven recorder locations in the Willamette River Valley were grouped to provide area control factors. The 24-hr weekday counts obtained at each of the individual recorder locations were adjusted by the seasonal adjustment factor developed from the grouped data. In this analysis, the section control method for adjusting coverage counts for seasonal variation used seasonal adjustment factors developed from raw data obtained from the same recorders used for obtaining the coverage counts. This is an ideal condition and will provide a minimum of error. It is expected that increased errors will result when area control factors are used because coverage count and seasonal adjustment factor data are not obtained from identical sources. It is the purpose of this phase of the analysis to determine the extent of increase in errors resulting from use of area control seasonal adjustment factors.

A comparison of the errors of ADT estimates obtained from section and area control factors are given in Table 8. For each of the seven recorders included in this phase, both the median and mean error of ADT estimates are given. Also given is the "t" ratio and the probability, and the frequency of errors less than 10 percent.

Statistical tests indicate that the increased errors using area control as contrasted to section control for adjusting coverage counts for seasonal variation are significant. It is interesting to note that the increase in errors varies from about 25 percent up to almost 100 percent. The frequency of errors less than ten percent for section control varies from a minimum of 62 percent to a maximum of 98 percent, with the frequency for five recorders exceeding 80 percent. The use of area control, on the other hand, produces a frequency of errors less than ten percent varying from 52 percent to 79 percent. In three instances the frequency of errors less than ten percent was less than 60 percent.

The data obtained from the Gervais recorder which is somewhat typical of the comparison of errors of estimates using section and area control methods are shown in Figure 9. It will be noted from this Figure that the median error for section control was 4.9 percent, whereas the median error for area control was 7.4 percent, or an increase in error of approximately 50 percent. The frequency of error less than ten percent was 83 and 68 percent for section and area control, respectively.

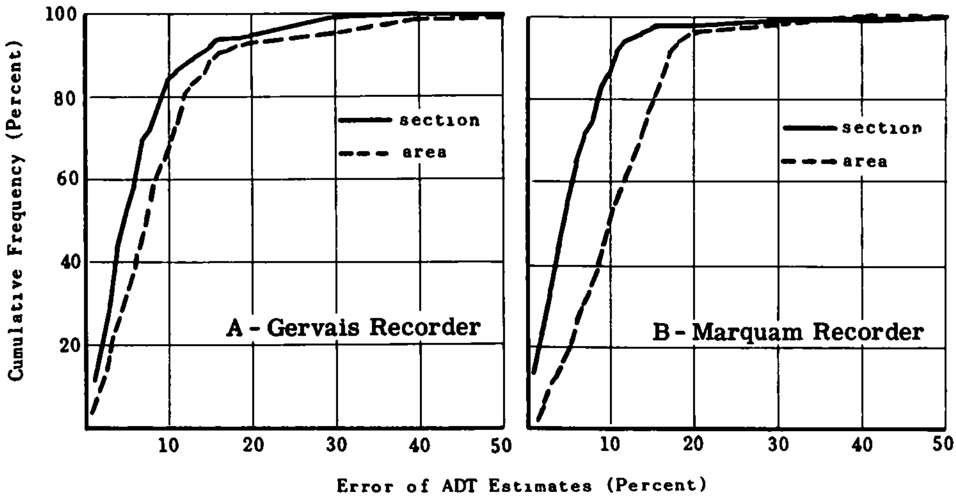

Figure 9. Cumulative frequency distribution of errors of ADT estimates, section vs area factors.

Figure 9 also shows the data obtained at the Marquam recorder. This recorder had extreme differences in the median error using section and area control methods. It will be noted that the median error for section control was 4.5 percent and for area control 10.2 percent, or an increase in error in excess of 100 percent. The frequency of errors less than ten percent varied from 86 percent to 52 percent for section and area control, respectively.

SUMMARY

The use of electronic data processing equipment for summarizing data obtained at permanent automatic traffic recorders has been successfully accomplished in Oregon. The use of mechanical methods rather than manual methods has resulted in monetary savings to the state and made the data more accessible for special studies. However, the mechanical method does not provide interim checks between monthly summarizations without additional costs. The development and use of punch tape traffic recorders, along with other technological developments in this field, will increase the advantages of mechanical summarization of traffic recorder data.

The first phase of the special analysis showed that at nine of the 13 recorder locations, estimates of ADT had statistically significant smaller errors when developed from noon factors rather than from midnight factors. At the other four recorder locations, smaller errors were found using the noon factors; however, the differences were not statistically significant. The change in error varied from practically no increase to increased accuracy of almost 40 percent. It is interesting to note that the four locations which did not show statistically significant increases in accuracy had ADT's of less than 1,000 vehicles per day. Also evident from this phase of the analysis was a tendency for the error in ADT estimates to become lower as the volume on the road increased.

This first phase of the analysis indicates that serious consideration should be given to the period of time considered as representative of the weekday. This is particularly true when the weekday data are used for the development of seasonal factors for adjusting coverage counts to an ADT. Although the improvement in the estimates does not appear to be substantial, it does appear that there is enough improvement to warrant additional study and consideration.

The second phase of the analysis indicated that for seven or more than one-half of the recorder locations, no statistically significant increase in the accuracy was found using 48-hr coverage counts rather than 24-hr coverage counts. These data do not provide strong supporting data to substantiate the need for 48-hr coverage counts. The six locations for which significant increases in the accuracy were obtained using 48-hr coverage counts as contrasted to 24-hr coverage counts were, with one exception, roads with an ADT of less than 1,000 vehicles per day. It does appear that there is a need for longer duration coverage counts on low-volume roads. Even though some statistically significant differences are found, the increase in accuracy was generally very small and did not exceed 25 percent, and was more typically around 10 percent. The use of 48-hr coverage counts normally requires additional traffic counting equipment to provide the same number of coverage counts obtained for a 24-hr period. The additional 24 hours during which the recorders remain in the field introduces an opportunity for more machine failures and erratic counts for other reasons. The additional equipment inventory and the probability of a lower percentage of usable counts offset by a relatively small increase in accuracy indicates that considerable study and evaluation should be made to justify obtaining 48-hr coverage counts.

The analysis for the position of the weekday with respect to accuracy of ADT estimates indicated that counts beginning early in the week and counts ending late in the week were less accurate than those taken from the more central portions of the week. It does not appear practical to confine coverage counts to the middle of the week, therefore this phase of the analysis should serve only to indicate that more reliability can be placed on midweek counts than any other counts. It does not appear that any significant value could be obtained by additional study of these traffic characteristics.

The comparison of seasonal adjustment factors developed both monthly and weekly indicated substantial increases in accuracy using weekly factors. All 13 recorders had statistically significant increases in the accuracy using a weekly factor rather than a monthly factor. The frequency of errors less than ten percent varied for the monthly factor from 50 to 99 percent, with four recorders showing frequencies of less than 80 percent. Weekly factors, on the other hand, had a frequency of errors less than ten percent varying from 66 to 100 percent of the time, and at nine of the 13 recorders the frequency of errors less than ten percent was more than 90 percent of the time. In only one instance was it below 80 percent.

Although additional time is required to compute 52 factors rather than 12 factors, increased accuracy up to approximately 50 percent may be realized by the use of weekly rather than monthly factors. The use of mechanical rather than manual methods of computing seasonal factors should remove any objections to using weekly factors.

There did not appear to be any seasonal trend or a particular season of the year when the accuracy of ADT estimates showed abnormal increases in accuracy using weekly rather than monthly factors.

The final phase of the study was an analysis of ADT estimates resulting from various control methods of applying seasonal adjustment factors. The procedures used in this study of obtaining the raw data for coverage count use and the seasonal adjustment factor use does not introduce errors resulting from variations in traffic patterns at different locations, erroneous counts resulting from faulty and poorly adjusted equipment, or any of the other errors normally encountered in normal traffic counting procedures.

The results of this study indicate that extreme caution and care must be exercised in selecting seasonal adjustment factors for adjusting coverage counts, and the degree of accuracy obtained will be dependent on methods selected. The area control method of applying factors introduces considerable error, and therefore may not be suitable for determining a fairly accurate ADT. For the recorders included in the study, the area control generally provided estimates with errors less than 10 percent about two-thirds of the time. This has been the acceptable error considered desirable in Oregon in developing traffic counting programs. It appears, then, that there is room for use of area control; however, the grouping of control locations should not be done without first reviewing seasonal characteristics to assure that similar patterns are obtained on all locations.

The utlimate use of ADT estimates will normally determine the error which may be tolerated in preparing the estimates. If section volume is required, maximum errors in excess of ten percent will not be tolerated. On the other hand, if the ADT estimates are to be used in the compilation of vehicle-mile data, errors in excess of ten percent may balance out with both plus and minus errors, so that errors of this magnitude are no real serious problem. The needs of each individual traffic counting program must be reviewed to determine the accuracy desired and the financial feasibility of providing the desired accuracy. The development of the traffic counting program in Oregon is the result of a compromise between desired accuracy and the financial requirements to provide the desired accuracy.

Oregon feels, as a result of this study, that it can obtain desirable results by taking 24-hr coverage counts expanded to an ADT by weekly seasonal adjustment factors based on noon-to-noon weekdays. Area and route control can be used with caution when section control is not readily availble.

ACKNOWLEDGMENT

Statistical control for the study reported in this paper was by Noel F. Kaestner, Statistician, Oregon State Highway Department, and Professor of Psychology, Willamette University.

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Some Statistical Evaluations of Truck Weight Characteristics in Mississippi

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The number of trucks needed to be weighed was determined so that the mean weight of each axle and vehicle for each vehicle type would have an error not greater than ±5 percent on 95 percent confidence limit. This was done separately for empty and loaded vehicles. Analyses of mean weights for the periods of weight stations operations (6 a.m.-2 p.m.; 2 p.m.-10 p.m.; 10 p.m.-6 a.m.; and 8 a.m.-4 p.m.) disclosed that the period 2 p.m.-10 p.m. produced mean weights which were not significantly different from the mean weights of 24-hr operations and, therefore, could be substituted for the 24-hr operations provided the mean weights were desired. It was found that there were several statistical populations (or universes) of mean weight characteristics which were significantly different. Individual stations were identified in relation to these populations, thus indicating the stations where reliable mean weights could be obtained and correctly interpreted. An IBM-650 computer was used for statistical computations which basically consisted of (a) mean, (b) standard deviation, (c) standard error of the mean, (d) sample sizes for ± 5 percent error on 95 percent confidence limit, (e) t test of significance of differences between means, and (f) analysis of variance tests.

● TRUCK WEIGHING by state highway departments for highway planning and research has been in effect for the last 25 years or so. The principal objectives have been the determination of mean axle and vehicle weights; the frequency distribution of weights by weight groups; the weight characteristics of the heaviest type vehicles; the frequency of application of heavy loads to the pavements; and the extent of overloading as determined by the individual state laws and as recommended by the American Association of State Highway Officials.

Concerning truck weighing, there is no unanimity of opinion or standardization of practices among the states, and no objective criteria have been developed to determine whether the data are adequate or inadequate for each purpose for which they are used. However, as experience was being gained, the tendency appeared among the states to curtail the weight sampling by reducing the number of stations or the hours of operations or both.

In 1959, Mississippi decided to undertake statistical analyses of its truck weight data. The primary purpose was to investigate the possibilities that mean weights, of quality similar to those obtained by procedures currently in use, could be obtained by simplified methods. More specifically, it was hoped that the studies would show the possibility that an administratively convenient 8-hr daylight period of truck weighing operations would provide the data needed for trend studies, mean axle weights, and mean vehicle weights, with substantially the same degree of accuracy as obtained by the procedures used in 1958 and 1959. Experience and observations led to the belief that this improvement might be possible, and, if so, savings would be effected. Also, it was thought that the inconvenience to the driving public might be reduced and safety fostered because Mississippi felt that the night weighing operations were potentially more dangerous than those of the daylight hours. It was hoped that schedules of truck weighing could be developed to provide the results which would be representative of axle and vehicle mean weights of both daylight and night operations by daytime sampling only. It was realized, however, that other data such as frequency of occurrence of very heavy axles and vehicles and their load distribution characteristics are equally or even more important for the solution of engineering problems.

When the studies were first conceived in Mississippi, the general qualitative appraisal and exploration were only of secondary importance. It was as a byproduct that additional information on the characteristics of some of the truck weight data was disclosed. But it is chiefly these incidental disclosures and the methods used in obtaining them that may be of interest and application outside of Mississippi.

Historically, from the very beginning of truck weighing operations in Mississippi, the location of weighing stations and periods of weighing have been more or less decided on judgment formed from the general knowledge of local traffic characteristics. Truck weight sampling practices have been varied. In the late 1930's, weighing stations were operated during each season of the year on all road systems under the state's jurisdiction. Provision was made in the schedules for 24-hr sampling on both weekdays and weekends. Following the 1930's, the collection of weight data was continued in the summer months at 15 locations, with sampling periods varying from 8 hours in some years to 24 hours in others. A few new locations were added in urban areas and on the state's secondary system during the summer of 1956.

During 1958 and 1959, weight data were collected for 24-hr periods at 42 locations, 18 of which were on the primary system, 7 in urban areas, and 17 on the secondary system. At each location there were three 8-hr operations — from 6 a.m. to 2 p.m., from 2 p.m. to 10 p.m., and from 10 p.m. to 6 a.m. — all during the summer months. The data from these operations provided the material for the studies presented in this paper.

Each study of qualitative appraisal was so designed as to reveal by probability measures the existence of a significant situation or provide the knowledge, if possible, which would lead toward a conclusion that a more efficient and desirable procedure than the one in operation could be developed.

The vehicle and axle weights for any given vehicle type obtained at any station constitute a sample of a larger population of all vehicles of the type passing that station. To aid in designing more efficient samples for future operations, it was necessary to decide whether the population of vehicle and axle weights for a given vehicle type passing one station was sufficiently similar to that of other stations as to be combined into one composite population. The assumption was made that the weight distributions obeyed the well-known Gaussian law. A statistical test, the F test, provided information on whether or not the spread of the underlying normal distribution as measured by the standard deviations obtained at one station differed significantly, in the statistical sense, from the spread obtained at the other stations. The 18 rural stations on the primary state highways were used for this purpose.

The analyses were made by vehicle types for 1959, using data for loaded and empty vehicles.

The vehicle types for which sufficient data were available for analysis are as follows:

Type	Description
13	2-axle motortruck with dual tires on rear axles
14	3-axle motortruck
21	2-axle tractor, 1-axle semitrailer
22	2-axle tractor, 2-axle semitrailer
24	3-axle tractor, 2-axle semitrailer

Following the general statistical practices, the mean values were not computed for a count of less than 5 vehicles. The standard deviation and the variance were not computed for a count of less than 10. The sample size was not computed for a count of less than 30. The standard theoretical distribution values used for determination of significance are for the 95 percent confidence limit. If the computed values are equal to or less than the theoretical values, it is considered that the station mean weights could come from a single statistical population. This test was used only for screening purposes, and the more detailed testing was performed later.

The particular expression of the F test used for this purpose is given, using the following definitions:

A = individual weight;
ΣA_1 = total of weights at station 1;
ΣA = total of weights at all stations;
N_1 = number of weights at station 1;
N = total number of weights;
n_w = degrees of freedom, within stations;
n_b = degrees of freedom, between stations;
S_w = squares, within stations;
S_b = squares, between stations;
V_w = variance, within stations;
V_b = variance, between stations; and
m = last weight.

Within Stations

$$S_w = \left(\Sigma A_1{}^2 - \frac{(\Sigma A_1)^2}{N_1}\right) + \left(\Sigma A_2{}^2 - \frac{(\Sigma A_2)^2}{N_2}\right) + \ldots + \left(\Sigma A_m{}^2 - \frac{(\Sigma A_m)^2}{N_m}\right)$$

$$V_w = \frac{S_w}{n_w}$$

in which

n_w = total number of weights at all stations minus the number of stations or degrees of freedom within stations.

Between Stations

$$S_b = \frac{(\Sigma A_1)^2}{N_1} + \frac{(\Sigma A_2)^2}{N_2} + \ldots + \frac{(\Sigma A_m)^2}{N_m} - \frac{(\Sigma A)^2}{N}$$

$$V_b = \frac{S_b}{n_b}$$

in which

n_b = number of stations or degrees of freedom between stations.

$$F = \frac{\text{Larger variance}}{\text{Smaller variance}}$$

The larger variance could be either V_w or V_b.

Table 1 shows that, for total vehicle weights, the 2-axle tractors with 2-axle semitrailers (type 22) had the computed F-value of 6.20 as compared with the limiting theoretical value of 1.96. From this it was concluded that the distribution between stations as measured by the average value at each station did differ significantly from the distribution of values

TABLE 1

F TEST OF SIGNIFICANCE OF DIFFERENCES WITHIN AND BETWEEN MEAN WEIGHTS OF LOADED VEHICLES AT RURAL STATIONS

Vehicle Type	Axle	F Computed	F Theoretical
13	1	2 14	1 96
13	2	1 91	1 96
Total 13		1 89	1 96
14	1	1 21	2 13
14	2	1 60	2 13
14	3	1 02	2 13
Total 14		1 67	2 13
21	1	2 29	1 92
21	2	2 74	1 92
21	3	4 24	1 92
Total 21		3 65	1 92
22	1	3 55	1 96
22	2	4 12	1 96
22	3	6 61	1 96
22	4	4 58	1 96
Total 22		6 20	1 96
24	1	1 39	2 71
24	2	1 75	2 71
24	3	3 78	2 71
24	4	2 64	2 71
24	5	2 78	2 71
Total 24		2,65	2 71

TABLE 2

COMPARISON OF NUMBER OF LOADED VEHICLES WEIGHED AND NUMBER OF LOADED VEHICLES NEEDED IN SAMPLE, RURAL

Vehicle Type	Axle	No of Vehicles Counted	Mean Weight (lb)	Sample Size No of Vehicles
13	1	1,521	4,634	120
13	2	1,521	10,519	218
13	Total	1,521	15,154	143
14	1	117	6,494	174
14	2	117	14,179	126
14	3	117	12,328	160
14	Total	117	32,897	81
21	1	792	5,483	131
21	2	792	13,536	92
21	3	792	13,577	144
21	Total	792	32,597	77
22	1	2,814	7,710	101
22	2	2,814	15,220	60
22	3	2,814	13,235	109
22	4	2,814	13,624	104
22	Total	2,814	49,789	57
24	1	93	8,355	77
24	2	93	11,353	148
24	3	93	12,181	86
24	4	93	12,582	92
24	5	93	13,384	86
24	Total	93	57,859	46

TABLE 3

COMPARISON OF NUMBER OF EMPTY VEHICLES WEIGHED AND NUMBER OF EMPTY VEHICLES NEEDED IN SAMPLE, RURAL

Vehicle Type	Axle	No of Vehicles Weighed	Mean Weight (lb)	Sample Size No of Vehicles
13	1	1,020	3,933	89
13	2	1,020	5,139	224
13	Total	1,020	9,073	115
14	1	93	5,460	796
14	2	93	6,227	135
14	3	93	4,936	412
14	Total	93	16,624	267
21	1	555	4,938	115
21	2	555	6,523	146
21	3	555	5,805	258
21	Total	555	17,268	121
22	1	1,291	6,379	92
22	2	1,291	7,232	98
22	3	1,291	5,091	163
22	4	1,291	5,302	154
22	Total	1,291	23,993	74
24	1	28	7,953	166
24	2	28	6,435	146
24	3	28	6,203	115
24	4	28	4,932	117
24	5	28	5,139	103
24	Total	28	30,664	80

around each station mean, and therefore it would not be expected that they came from a single population. Similarly, the F-values computed for the individual axle weights for this type of truck combination lead to the same conclusion. The same test when applied to 2-axle trucks with dual rear tires (type 13) and 3-axle motortrucks (type 14) indicated that the weights for these vehicles could come from single respective populations, or that the means for these vehicle types could come from single populations as indicated by the computed F-values which are smaller than the limiting theoretical values.

Under the existing procedures, trucks of all types are being weighed at all stations. It is considered impractical to designate only the particular types of vehicles to be weighed at some stations and not at other stations (as could be erroneously inferred from the data in Table 1). The discrepancy between the computed and the theoretical values for the 2-axle tractors with the 2-axle semitrailers (type 22) was so great that it was felt that similar results could be expected from empty vehicles and also at urban stations where greater dispersions of weights are usually found.

Despite the observed population heterogeneity, random sampling could be applied, and mean values describing the heterogeneous population computed. But for greater efficiency, more homogeneous populations were identified, as is explained later. Inasmuch as random sample sizes from a heterogeneous population would be expected to be larger for the same degree of reliability of the mean than from homogeneous populations, it was decided to determine the sample sizes for the heterogeneous population first. It was reasoned that such samples would provide the necessary data whether or not means would be found later to isolate homogeneous populations.

In these studies the sample size design criterion was set at ± 5 percent standard error of the mean on the 95 percent confidence limit. In other words, the probability would be 19 to 1 in favor that the estimates of mean values yielded by many samples of the specified size would not differ from the true mean by more than twice the corresponding standard errors, in this case by not more than ± 5 percent of the means.

Defining the population as all units weighed, the number of units in the sample according to the design specification was computed from the formula:

$$N = \frac{(1.96)^2 V}{(5\% M)^2}$$

in which

1.96 = T value for sample size between 30 and infinity with a confidence limit of 95 percent;

V = population variance; and

M = population mean weight.

The number of units in the populations and the corresponding number of units needed in the sample as required by the design are given in Table 2. The dispersions or spread of axle weights are larger than vehicle weights. Therefore, the minimum number of vehicles to be selected for the sample is determined by the largest of the indicated minimum number of axles of the vehicle type. For example, for the 2-axle trucks (type 13), the number of units needed to be weighed is 218 as determined by the second axle and not 143 as shown for the total vehicle weight. Thus a random selection of 218 trucks (type 13) would produce mean axle weights and a mean vehicle weight which would be representative of the 1,521 such vehicles actually weighed. This illustrates the point that considerable saving of effort can be accrued if the mean weights only were to be considered. Similar observation applies to 2-axle tractor, 1-axle trailer combinations (type 21), where 144 such vehicles would have provided the representative mean weights instead of 792 actually weighed. The most striking observation is about the 2-axle tractor, 2-axle semitrailer combination (type 22), of which 2,814 vehicles were weighed and only 109 vehicles, randomly selected, were needed to produce reliable mean weights.

The data in Table 2 also lead to the observation that the loading practices vary so widely for some vehicle types that the total number of units in the population (the number actually weighed) was not sufficient to produce reliable means. Thus for the 3-axle trucks (type 14), the minimum needed to assure reliability should be 174, whereas only 117 were actually weighed. Similarly, the 93 units actually weighed of the 3-axle trucks with 2-axle semitrailers (type 24) were not sufficient, as 148 would be needed to assure the mean weights to be within ±5 percent error on the 95 percent confidence limit.

The 14 and 24 vehicle types are of the heaviest in the single-unit trucks and combinations, respectively. They are also the rarest in frequency of appearance.

TABLE 4a

SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS, ALSO BETWEEN VARIANCES, OF 8 AM TO 4 PM PERIOD COMPARED WITH 24 HOURS — 1959

				Axle										Total Vehicle	
				1		2		3		4		5			
System	Type	Vehicle Loading	Value[a]	F	t	F	t	F	t	F	t	F	t	F	t
Rural	13	Loaded	T												
			C												
		Empty	T												
			C												
	21	Loaded	T												
			C												
		Empty	T												
			C												
	22	Loaded	T	1.08			1.96		1.96						
			C	1.12			2 38		2.92						
		Empty	T			1.08	1.96	1 08		1.08				1.08	
			C			1.17	2.36	1.21		1.20				1.20	
Urban	13	Loaded	T												
			C												
		Empty	T												
			C												
	14	Empty	T												
			C												
	21	Loaded	T												
			C												
		Empty	T												
			C												
	22	Loaded	T												
			C												
		Empty	T												
			C												

[a]T = theoretical and C = computed.

Note: Blank spaces indicate that computed values are smaller than theoretical values, thus the differences are "not significant."

TABLE 4c

SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS, ALSO BETWEEN VARIANCES, OF 2 PM TO 10 PM PERIOD COMPARED WITH 24 HOURS — 1959

				Axle										Total Vehicle	
				1		2		3		4		5			
System	Type	Vehicle Loading	Value[a]	F	t	F	t	F	t	F	t	F	t	F	t
Rural	13	Loaded	T												
			C												
		Empty	T												
			C												
	21	Loaded	T												
			C												
		Empty	T												
			C												
	22	Loaded	T								1 96				
			C								2.11				
		Empty	T						1.96	1.13				1.13	
			C						2 73	1.22				1 14	
Urban	13	Loaded	T												
			C												
		Empty	T												
			C												
	14	Empty	T												
			C												
	21	Loaded	T												
			C												
		Empty	T												
			C												
	22	Loaded	T												
			C												
		Empty	T			1.25		1.25						1.25	
			C			1.48		1 33						1.40	

[a]T = theoretical and C = computed.
Note Blank spaces indicate that computed values are smaller than theoretical values, thus the differences are "not significant "

TABLE 4b

SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS, ALSO BETWEEN VARIANCES, OF 6 AM TO 2 PM PERIOD COMPARED WITH 24 HOURS — 1959

				Axle										Total Vehicle	
				1		2		3		4		5			
System	Type	Vehicle Loading	Value[a]	F	t	F	t	F	t	F	t	F	t	F	t
Rural	13	Loaded	T												
			C												
		Empty	T		1.96										
			C		2.17										
	21	Loaded	T												
			C												
		Empty	T												
			C												
	22	Loaded	T	1.08			1.96		1.96		1.96				1.96
			C	1.10			3.38		4.45		2.64				3.31
		Empty	T						1.96						
			C						2.28						
Urban	13	Loaded	T												
			C												
		Empty	T												
			C												
	14	Empty	T		2.40										
			C		2 44										
	21	Loaded	T		1.99										
			C		3.03										
		Empty	T			1.46	2.00		2.00						2.00
			C			1.58	2.40		2.55						2.50
	22	Loaded	T	1.19	1.97		1.97		1.97		1.97				1.98
			C	1.32	2.98		4.90		7.14		6.36				6.50
		Empty	T			1.30		1.30							
			C			1.63		1.40							

[a]T = theoretical and C = computed.
Note: Blank spaces indicate that computed values are smaller than theoretical values, thus the differences are "not significant."

Studies by individual stations produced results generally in agreement with the data in Table 2.

Empty vehicles were analyzed in the same manner. The results for all stations are given in Table 3.

Table 3 indicates that the number of empty units that should be weighed is also considerably less than actually weighed for types 13, 21, and 22. Types 14 and 24 samples should have been larger than actually weighed to satisfy the desired accuracy.

It was the practice to weigh all the heavy vehicles passing by the weighing parties. The mean weights of the heavy vehicles, as they have been obtained for years, were less accurate than ±5 percent error on the 95 percent confidence limit. Therefore, to reduce the error of the mean, the weighing of trucks on more frequent schedules, increasing the number of weighing stations, or development of new procedures would be necessary to obtain samples as large as indicated in Table 2. However, the mean

TABLE 5a

ARRAY OF COMPUTED t-VALUES FOR SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS 2-AXLE MOTORTRUCK, DUAL REAR TIRES (TYPE 13), LOADED — RURAL, 1959

Axle 1			Axle 2			Vehicle		
Sta. No.	Computed Value	Theoretical Value	Sta. No.	Computed Value	Theoretical Value	Sta. No.	Computed Value	Theoretical Value
8 AM-4 PM Period Each Station Compared to Same Period at all Stations								
7	0.034	2.000	56	0.00	2.06	56	0.00	2.06
45	0.044	2.014	10	0.02	2.01	37	0.10	2.05
56	0.050	2.056	37	0.10	2.05	19	0.19	2.03
37	0.102	2.052	50	0.16	1.99	32	0.25	2.10
27	0.115	2.021	32	0.21	2.10	29	0.25	1.99
32	0.205	2.101	29	0.22	1.99	53	0.34	2.00
2	0.224	1.994	1	0.46	2.03	16	0.35	2.02
16	0.632	2.021	19	0.59	2.03	50	0.44	1.99
51	0.924	2.021	16	0.60	2.02	1	0.63	2.03
1	1.026	2.000	7	1.02	2.00	22	0.74	2.00
50	1.033	1.994	53	1.09	2.00	10	0.82	2.01
29	1.511	1.994	27	1.18	2.02	7	0.87	2.00
53	1.621	2.000	2	1.34	1.99	2	0.96	1.99
22	1.702	2.000	22	1.35	2.00	27	1.00	2.02
34	1.716	1.984	45	1.69	2.01	45	1.41	2.01
19	1.940	2.030	51	1.94	2.02	51	1.93	2.02
42	2.194	2.014	42	2.14	2.01	42	2.54	2.01
10	3.678	2.008	34	2.89	1.98	34	3.03	1.98
8 AM-4 PM Period Each Station Compared to 24-Hr Period at all Stations								
2	0.01	1.99	56	0.00	2.06	56	0.00	2.06
56	0.05	2.06	37	0.10	2.05	29	0.01	1.99
37	0.10	2.05	32	0.21	2.10	53	0.13	2.00
32	0.21	2.10	10	0.24	2.01	37	0.15	2.05
27	0.28	2.02	1	0.25	2.03	16	0.20	2.02
45	0.30	2.01	16	0.37	2.02	32	0.21	2.10
7	0.31	2.00	50	0.49	1.99	19	0.29	2.03
16	0.43	2.02	29	0.59	1.99	1	0.49	2.03
50	0.77	1.99	7	0.69	2.00	10	0.65	2.01
51	1.16	2.02	19	0.74	2.03	50	0.66	1.99
1	1.29	2.03	53	0.75	2.00	7	0.67	2.00
34	1.38	1.98	27	0.94	2.02	2	0.76	1.99
53	1.41	2.00	2	0.99	1.99	27	0.85	2.02
29	1.81	1.99	45	1.33	2.01	22	0.92	2.00
22	1.97	2.00	22	1.65	2.00	45	1.19	2.01
42	1.98	2.01	51	1.72	2.02	51	1.79	2.02
19	2.14	2.03	42	2.40	2.01	42	2.71	2.01
10	3.96	2.01	34	3.26	1.98	34	3.27	1.98

weights obtained by existing procedures, such as they were with respect to accuracy, have been used by the engineers and the administrators. So, at least judging by this criterion of usefulness, these mean weights were considered satisfactory. With this thought in mind, the exploration was made into the possibility of further reduction of sample sizes, instead of indicated theoretical increase, so that the accuracy of the mean weights from still smaller samples would not be significantly different from those actually obtained.

This approach is possible in theory when the concept of chance variations is considered. Because the mean values of two samples of different or equal sizes, taken from the same population, are likely to produce different results, it can be determined whether or not these differences could be due to chance variations among the units which compose the means. If within certain measures, as expressed by confidence limits, these differences can be attributed to chance, then it can be concluded that either one of the samples is representative when they are of equal size, or that the smaller sample is representative of the population or of a larger sample. These determinations were made by means of the t tests of significance of differences between means and the F tests of significances of differences between variances, using the equations:

$$t = \frac{M_1 - M}{SE}$$

and

$$F = \frac{V_1}{V_2}$$

TABLE 5b

ARRAY OF COMPUTED t-VALUES FOR SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS
2-AXLE TRACTOR, 1-AXLE SEMITRAILER (TYPE 21), LOADED — RURAL, 1959

Axle 1			Axle 2			Axle 3			Vehicle		
Sta No.	Computed Value	Theoretical Value	Sta No.	Computed Value	Theoretical Value	Sta. No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value
					8 AM-4 PM Period Each Station Compared to Same Period at all Stations						
10	0 00	2.12	53	0 00	2 06	37	0 00	2 20	37	0.00	2.20
53	0 00	2.06	37	0 00	2.20	37	0 00	2.20	37	0.00	2.20
32	0.04	2.15	37	0 00	2 20	29	0 04	2.09	32	0 00	2.15
19	0 04	2 09	32	0.04	2.15	53	0.05	2 06	53	0.05	2 06
51	0 04	2 09	2	0.04	2.11	51	0 08	2 09	29	0 08	2.09
37	0.06	2 20	51	0.04	2 09	32	0 11	2 15	51	0.08	2.09
37	0.06	2.20	7	0.12	2.12	7	0.12	2 12	16	0 09	2 07
7	0.08	2 12	29	0.13	2 09	16	0 14	2 07	45	0 10	2 06
29	0.13	2 09	16	0 14	2.07	19	0.17	2 09	2	0 12	2 11
16	0.23	2.07	45	0 20	2.06	22	0 36	2 03	7	0.16	2 12
50	0 92	2 02	19	0 21	2 09	10	0 43	2 12	19	0 17	2 09
45	1 05	2 06	10	0 35	2 12	56	1 09	2.02	10	0 35	2 12
2	1 12	2.11	27	0 50	2 03	50	1.68	2.02	22	1 06	2.03
27	1 34	2 03	22	0.68	2 03	45	2 30	2 06	50	1 49	2.02
34	2 13	2 02	50	1.02	2 02	2	2 60	2 11	34	2 06	2 02
22	2.66	2 03	56	1.41	2.02	34	2 75	2 02	56	2.26	2 02
56	2 67	2.02	34	1 69	2.02	27	3.67	2 03	27	2 89	2.03
					8 AM-4 PM Period Each Station Compared to 24-Hr Period at all Stations						
10	0 00	2.12	53	0 00	2.06	53	0 00	2.06	53	0 00	2.06
32	0.04	2 15	37	0.00	2 20	37	0.00	2 20	37	0.00	2.20
19	0 04	2 09	37	0.00	2.20	37	0 00	2 20	37	0 00	2.20
53	0 05	2 06	2	0 00	2.11	51	0.04	2 09	32	0 00	2 15
7	0.08	2 12	51	0 04	2 09	29	0.04	2 09	51	0 04	2 09
51	0 08	2.09	32	0.07	2.15	22	0.07	2.03	16	0 05	2.07
37	0 10	2.20	16	0.09	2 07	32	0 11	2.15	2	0.08	2.11
37	0.10	2 20	7	0.12	2 12	7	0 12	2.12	29	0.08	2.09
29	0.13	2.09	45	0.15	2.06	16	0.14	2 07	45	0 10	2 06
16	0 23	2.07	22	0.15	2.03	19	0 21	2 09	7	0.12	2 12
50	0.65	2.02	29	0.17	2.09	10	0 39	2 12	19	0.21	2 09
45	1.05	2 06	19	0.25	2 09	56	1.45	2 02	10	0 35	2 12
2	1.12	2.11	10	0.31	2.12	50	1.89	2.02	22	0 73	2 03
27	1 15	2.03	27	1 10	2 03	45	2.25	2.06	50	1.73	2.02
56	2 45	2.02	50	1 39	2 02	2	2.52	2.11	34	2.38	2.02
34	2.52	2 02	56	2.09	2 02	34	3 02	2.02	56	2.72	2.02
22	2.91	2.03	34	2.16	2 02	27	3 99	2 03	27	3 28	2 03

in which

M_1 = mean of the larger sample,
M = mean of the smaller sample,
SE = standard error of the mean of the smaller sample,
V_1 = larger variance, and
V_2 = smaller variance.

In Tables 4a, 4b, and 4c, the comparisons between the mean weights for different types of vehicles are shown for rural and urban stations. Mississippi was particularly interested in the three 8-hr periods: 8 a.m. to 4 p.m., 6 a.m. to 2 p.m., and 2 p.m. to 10 p.m. Thus the comparisons in Tables 4a, 4b, and 4c were made between these individual 8-hr periods and the 24-hr period. Only the results of the tests showing significance of the differences are given. Blank spaces mean that the computed values of F or t were smaller than the theoretical values and therefore the differences were considered to be statistically not significant. In Table 4c, only 2-axle tractors with 2-axle semitrailers (type 22) showed slight significance for the mean empty vehicle weight and some significance of certain mean axle weights in both rural and urban areas. Vehicles of types other than those given in the tables were not sufficient in numbers to make computations.

These tests indicated that, for all practical purposes, desired information concerning the mean vehicle and axle weights could have been obtained from the data obtained during the 8-hr period, 2 p.m. to 10 p.m., which would have been representative of the 24-hr weighing operations. However, this period would require operations after dark and, therefore, was considered unsatisfactory by Mississippi.

TABLE 5c

ARRAY OF COMPUTED t-VALUES FOR SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS
2-AXLE TRACTOR, 2-AXLE SEMITRAILER (TYPE 22), LOADED — RURAL, 1959

Axle 1			Axle 2			Axle 3			Axle 4			Vehicle		
Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value
8 AM-4 PM Period Each Station Compared to Same Period at all Stations														
42	0.04	2.15	32	0.08	2.09	42	0.00	2.15	32	0.04	2.09	50	0.01	2.01
32	0.17	2.09	42	0.11	2.15	32	0.08	2.09	1	0.07	2.15	16	0.03	1.99
19	0.28	2.03	1	0.11	2.15	1	0.18	2.15	42	0.11	2.15	32	0.04	2.09
16	0.40	1.99	45	0.46	1.99	50	0.36	2.01	53	0.36	1.98	42	0.07	2.15
37	0.74	2.02	53	0.49	1.98	16	0.70	1.99	16	0.42	1.99	1	0.14	2.15
10	0.76	2.03	2	0.68	1.99	7	1.03	2.01	50	0.55	2.01	53	1.09	1.98
1	0.94	2.15	50	0.95	2.01	45	1.91	1.99	2	0.61	1.99	2	1.83	1.99
27	0.96	1.99	29	1.06	2.04	37	1.97	2.02	22	0.84	1.99	7	2.02	2.01
2	1.04	1.99	16	1.23	1.99	29	2.08	2.04	7	1.20	2.01	34	2.10	1.99
53	1.05	1.98	19	1.28	2.03	51	2.32	2.01	29	1.51	2.04	29	2.12	2.04
34	1.25	1.99	22	1.35	1.99	34	2.33	1.99	34	2.19	1.99	37	2.18	2.02
50	1.30	2.01	10	1.87	2.03	53	2.70	1.98	37	2.48	2.02	45	2.38	1.99
7	1.36	2.01	34	1.99	1.99	2	2.95	1.99	10	2.52	2.03	10	2.48	2.03
45	1.42	1.99	37	2.22	2.02	22	3.15	1.99	45	2.85	1.99	22	2.65	1.99
51	2.63	2.01	7	2.96	2.01	19	3.33	2.03	19	2.96	2.03	19	2.69	2.03
56	3.05	2.00	56	3.05	2.00	27	4.07	1.99	56	3.04	2.00	51	3.87	2.01
29	3.24	2.04	27	3.29	1.99	10	2.70	2.03	27	3.35	1.99	27	4.33	1.99
22	3.34	1.99	51	3.96	2.01	56	5.12	2.00	51	3.41	2.01	56	5.08	2.00
8 AM-4 PM Period Each Station Compared to 24-Hr Period at all Stations														
42	0.00	2.15	42	0.07	2.15	42	0.00	2.15	32	0.04	2.09	42	0.04	2.15
32	0.17	2.09	32	0.08	2.09	32	0.13	2.09	42	0.07	2.15	32	0.08	2.09
37	0.46	2.02	1	0.14	2.15	1	0.14	2.15	1	0.07	2.15	1	0.14	2.15
27	0.50	1.99	22	0.47	1.99	7	0.18	2.01	53	0.08	1.98	50	0.40	2.01
19	0.52	2.03	16	0.52	1.99	50	0.99	2.01	50	0.27	2.01	53	0.41	1.98
2	0.69	1.99	29	0.69	2.04	37	1.47	2.02	22	0.37	1.99	16	0.54	1.99
16	0.81	1.99	19	0.86	2.03	16	1.56	1.99	16	0.78	1.99	7	1.52	2.01
34	0.87	1.99	2	1.24	1.99	34	1.56	1.99	7	0.88	2.01	34	1.56	1.99
10	0.97	2.03	53	1.27	1.98	53	1.57	1.98	2	0.93	1.99	29	1.81	2.04
1	0.97	2.15	34	1.38	1.99	29	1.61	2.04	29	1.29	2.04	37	1.85	2.02
45	1.06	1.99	45	1.38	1.99	22	1.93	1.99	34	1.85	1.99	22	1.85	1.99
53	1.48	1.98	50	1.43	2.01	10	2.23	2.03	37	2.27	2.02	10	2.19	2.03
7	1.62	2.01	10	1.44	2.03	19	2.82	2.03	10	2.31	2.03	2	2.31	1.99
50	1.63	2.01	37	1.86	2.02	45	2.92	1.99	19	2.74	2.03	19	2.35	2.03
51	2.41	2.01	7	2.34	2.01	51	2.99	2.01	45	3.30	1.99	45	3.10	1.99
56	2.69	2.00	56	3.82	2.00	2	3.62	1.99	56	3.42	2.00	51	4.27	2.01
29	3.51	2.04	27	4.03	1.99	27	4.89	1.99	51	3.69	2.01	27	4.96	1.99
22	3.87	1.99	51	4.44	2.01	56	6.20	2.00	27	3.71	1.99	56	5.78	2.00

The 8 a.m. to 4 p.m. period (Table 4a) shows some significance for total weights of empty vehicles (type 22), for some of the axle loads empty, and for total weights of loaded vehicles of the same type at weight stations on rural roads. In urban areas during that period, there was no significance for either test for any vehicle type or any axle during the 8 a.m. to 4 p.m. period. The 6 a.m. to 2 p.m. period (Table 4b) is the least representative, having the largest number of instances of significant differences.

TABLE 6a

ARRAY OF COMPUTED F-VALUES FOR SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS 2-AXLE MOTORTRUCK, DUAL REAR TIRES (TYPE 13), LOADED—RURAL, 1959

Axle 1			Axle 2			Vehicle		
Sta. No.	Computed Value	Theoretical Value	Sta. No.	Computed Value	Theoretical Value	Sta. No.	Computed Value	Theoretical Value
8 AM-4 PM Period Each Station Compared to Same Period at all Stations								
34	1.01	1.25	1	1.01	1.59	50	1.01	1.32
42	1.01	1.49	50	1.02	1.32	27	1.04	1.55
56	1.06	1.71	22	1.04	1.39	7	1.08	1.37
16	1.07	1.53	16	1.05	1.53	22	1.09	1.39
50	1.09	1.32	37	1.07	1.69	16	1.09	1.53
7	1.14	1.37	42	1.10	1.49	34	1.09	1.25
29	1.16	1.32	27	1.10	1.55	1	1.10	1.59
19	1.18	1.59	34	1.14	1.25	2	1.11	1.32
22	1.18	1.39	2	1.14	1.32	56	1.11	1.71
37	1.28	1.69	56	1.16	1.71	42	1.13	1.49
45	1.31	1.45	7	1.16	1.37	51	1.13	1.53
10	1.36	1.39	51	1.16	1.53	53	1.15	1.37
27	1.36	1.55	53	1.17	1.37	37	1.18	1.69
51	1.37	1.53	29	1.25	1.32	10	1.20	1.39
53	1.53	1.37	10	1.27	1.39	29	1.32	1.32
2	1.63	1.32	32	1.53	1.96	32	1.54	1.96
1	2.07	1.59	45	1.88	1.45	45	1.69	1.45
32	2.32	1.96	19	2.09	1.59	19	1.76	1.59
8 AM-4 PM Period Each Station Compared to 24-Hr Period at all Stations								
50	1.04	1.32	1	1.01	1.59	27	1.01	1.55
34	1.06	1.25	50	1.02	1.32	50	1.04	1.32
42	1.06	1.49	22	1.04	1.39	22	1.05	1.39
56	1.11	1.71	16	1.05	1.53	16	1.05	1.53
16	1.12	1.53	37	1.07	1.69	34	1.05	1.25
7	1.20	1.37	42	1.10	1.49	56	1.07	1.71
29	1.22	1.32	27	1.10	1.55	2	1.08	1.32
19	1.24	1.59	34	1.14	1.25	51	1.09	1.53
22	1.24	1.39	2	1.14	1.32	7	1.12	1.37
27	1.30	1.55	56	1.16	1.71	1	1.13	1.59
37	1.35	1.69	7	1.16	1.37	10	1.16	1.39
45	1.37	1.45	51	1.17	1.53	42	1.17	1.49
10	1.43	1.39	53	1.17	1.37	53	1.19	1.37
51	1.44	1.53	29	1.25	1.32	37	1.22	1.69
53	1.46	1.37	10	1.27	1.39	29	1.37	1.32
2	1.55	1.32	32	1.53	1.96	32	1.60	1.96
1	2.16	1.59	45	1.87	1.45	19	1.70	1.59
32	2.43	1.96	19	2.09	1.59	45	1.74	1.45

Historically speaking, in all states, the truck weighing stations were almost universally selected intuitively rather than on the basis of the representative characteristics of mean weights obtained. In Mississippi the weighing stations were located at what was believed to be "representative" locations. As a part of this study, it was decided to evaluate this "representativeness" by measuring the extent of variations or similarities that exist among the weights obtained at individual stations and their collective means for the respective truck types.

The measures of significance of these variations were the t and F tests. If both tests showed no significance (that is, if the computed values are smaller than the theoretical values), the interpretation then is that the station data could be representative of the mean of all 18 stations. The results are given in Tables 5a, 5b, 5c, and Tables 6a, 6b, and 6c for the respective measures. The stations are arrayed in ascending order of computed values with the theoretical values for the 5 percent confidence level given side by side.

The primary interest of Mississippi was in the characteristics of the 8 a.m. to 4 p.m. period as they were related to the total 24-hr period. From the arrays of t and F tests, Tables 5a, 5b, and 5c, and 6a, 6b, 6c series, it was found that, for loaded vehicles of types 13, 21, and 22 in the predominant instances, no significance was indicated for the variances of weights at stations 1, 7, 10, 27, 29, 32, 34, 37, 42, 51, and 53. That is, each one of these 11 stations for practical purposes could be considered representative of the means of the 18 stations. The results of this study indicated the possibility of reducing the number of stations.

TABLE 6b

ARRAY OF COMPUTED F-VALUES FOR SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS
2-AXLE TRACTOR, 1-AXLE SEMITRAILER (TYPE 21), LOADED — RURAL, 1959

Axle 1			Axle 2			Axle 3			Vehicle		
Sta. No.	Computed Value	Theoretical Value	Sta. No.	Computed Value	Theoretical Value	Sta. No.	Computed Value	Theoretical Value	Sta. No	Computed Value	Theoretical Value
8 AM-4 PM Period Each Station Compared to Same Period at all Stations											
7	1.01	2.07	10	1.04	2.07	34	1.02	1.51	34	1.01	1.51
32	1.02	2.21	34	1.06	1.51	19	1 02	1.92	19	1.10	1.92
29	1.08	1.92	51	1.12	1.92	10	1.04	2.07	51	1.16	1.92
50	1.10	1.53	19	1.18	1.92	51	1.21	1.92	22	1.21	1.59
22	1.12	1 59	7	1.19	2 07	32	1.22	2.21	10	1.26	2.07
10	1.14	2.07	37	1.19	2.54	29	1.26	1.92	29	1.28	1.92
19	1.14	1.92	53	1.21	1 73	7	1.31	2.07	37	1.28	2.54
53	1.17	1.73	2	1.36	2.01	37	1.33	2.54	7	1.30	2.07
45	1.23	1.71	22	1.43	1 59	22	1.38	1.59	32	1.37	2.21
2	1 36	2.01	50	1.66	1.53	53	1.39	1.73	53	1.47	1.73
56	1 38	1.53	27	1.72	1.57	27	1.48	1.57	27	1.66	1.57
16	1.41	1.81	16	1.86	1.81	45	1.56	1.71	45	1.70	1.71
27	1.62	1.57	45	1.91	1.71	50	1.64	1.53	50	1.81	1.53
51	1.63	1.92	29	1.93	1.92	16	1.71	1.81	16	2.03	1.81
34	2.25	1.51	56	2.00	1.53	56	1.73	1.53	56	2.05	1.53
37	5.42	2.54	32	2.17	2.21	2	2.69	2.01	2	2.43	2.01
8 AM-4 PM Period Each Station Compared to 24-Hr Period at all Stations											
32	1.00	2.21	51	1.04	1.92	32	1.11	2.21	22	1 10	1.59
7	1.03	2.07	7	1.10	2.07	34	1.12	1.51	34	1.11	1.51
29	1.07	1.92	10	1.12	2.07	19	1.12	1.92	7	1.17	2.07
50	1.08	1.53	53	1.13	1.73	10	1.14	2.07	19	1.22	1.92
22	1.10	1.59	34	1.14	1.51	7	1.20	2.07	51	1.28	1.92
19	1.12	1 92	19	1.27	1.92	53	1.27	1.73	53	1.33	1.73
53	1.15	1.73	2	1.27	2.01	22	1.27	1.59	10	1.39	2.07
10	1.16	2.07	37	1.28	2.54	51	1.32	1.92	29	1.41	1.92
56	1.40	1.53	22	1.33	1.59	27	1.36	1.57	37	1.41	2.54
16	1.43	1.81	27	1.61	1.57	29	1.38	1 92	27	1.50	1 57
51	1.61	1.92	16	1.73	1.81	37	1.46	2.54	32	1.51	2.21
27	1.64	1 57	45	1.77	1.71	16	1.56	1.81	45	1.54	1.71
34	2.22	1.51	50	1.78	1.53	56	1.58	1.53	16	1.84	1.81
2	2.78	2.01	56	1.86	1.53	50	1.79	1.53	56	1.86	1.53
45	4.77	1.71	29	2.07	1.92	45	5.49	1.71	50	2.00	1.53
37	5.35	2.54	32	2.33	2.21	2	9.17	2.01	2	2.20	2.01

Such arrays can be useful for identification of stations with relation to populations of similar characteristics, particularly within and among the administrative road systems. An important observation is that of the 11 stations previously mentioned, all but one are located on primary state highways. The 7 stations for which the F-values indicated significance of differences of variances and, therefore, could not be identi-

TABLE 6c

ARRAY OF COMPUTED F-VALUES FOR SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS
2-AXLE TRACTOR, 2-AXLE SEMITRAILER (TYPE 22), LOADED — RURAL, 1959

Axle 1			Axle 2			Axle 3			Axle 4			Vehicle		
Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value	Sta No	Computed Value	Theoretical Value
8 AM-4 PM Period Each Station Compared to Same Period at all Stations														
53	1 01	1 25	10	1 01	1 59	16	1 02	1 28	50	1 00	1 44	16	1 01	1 28
34	1 02	1 28	16	1 04	1 28	50	1 05	1 44	16	1 03	1 28	34	1 09	1 28
2	1 05	1 32	53	1 08	1 25	27	1 06	1 28	51	1 06	1 46	51	1 11	1 46
19	1 06	1 57	19	1 10	1 57	19	1 07	1 57	19	1 07	1 57	19	1 13	1 57
37	1 07	1 53	32	1 10	1 92	1	1 10	2 21	29	1 07	1 64	2	1 18	1 32
56	1 08	1 35	27	1 14	1 28	51	1 14	1 46	27	1 08	1 28	32	1 19	1 92
42	1 10	2 21	51	1 15	1 46	34	1 16	1 28	2	1 11	1 32	53	1 19	1 25
16	1 15	1 28	50	1 20	1 44	29	1 17	1 64	1	1 17	2 21	50	1 21	1 44
50	1 21	1 44	34	1 22	1 28	10	1 24	1 59	34	1 17	1 28	29	1 22	1 64
7	1 24	1 45	29	1 24	1 64	37	1 31	1 53	32	1 18	1 92	27	1 22	1 28
45	1 25	1 28	2	1 31	1 32	45	1 31	1 28	53	1 21	1 25	7	1 35	1 45
10	1 36	1 59	7	1 43	1 45	32	1 38	1 92	10	1 24	1 59	37	1 40	1 53
29	1 36	1 64	22	1 44	1 28	2	1 40	1 32	37	1 29	1 53	1	1 41	2 21
27	1 42	1 28	56	1 52	1 35	53	1 51	1 25	7	1 35	1 45	10	1 47	1 59
32	1 51	1 92	42	1 58	2 21	42	1 73	2 21	45	1 36	1 28	45	1 47	1 28
22	1 62	1 28	37	1 61	1 53	7	1 74	1 45	56	1 36	1 35	42	1 55	2 21
51	1 84	1 46	45	1 63	1 28	22	1 86	1 28	22	1 46	1 28	22	1 77	1 28
1	10 43	2 21	1	2 47	2 21	56	2 03	1 35	42	1 55	2 21	56	1 92	1 35
8 AM-4 PM Period Each Station Compared to 24-Hr Period at all Stations														
16	1 02	1 28	10	1 02	1 59	16	1 03	1 28	51	1 01	1 46	16	1 06	1 28
42	1 02	2 21	16	1 03	1 28	1	1 05	2 21	50	1 05	1 44	53	1 13	1 25
37	1 05	1 53	53	1 07	1 25	51	1 09	1 46	16	1 09	1 28	34	1 14	1 28
56	1 05	1 35	32	1 10	1 92	50	1 11	1 44	29	1 12	1 64	51	1 16	1 46
50	1 08	1 44	19	1 11	1 57	19	1 12	1 57	19	1 13	1 57	27	1 16	1 28
53	1 11	1 25	27	1 13	1 28	27	1 12	1 28	27	1 13	1 28	19	1 18	1 57
34	1 15	1 28	51	1 15	1 46	34	1 22	1 28	53	1 15	1 25	2	1 24	1 32
2	1 18	1 32	50	1 21	1 44	29	1 24	1 64	2	1 17	1 32	32	1 25	1 92
19	1 19	1 57	34	1 23	1 28	45	1 25	1 28	1	1 23	2 21	50	1 27	1 44
29	1 21	1 64	29	1 25	1 64	10	1 31	1 59	34	1 23	1.28	29	1 29	1 64
27	1 26	1 28	2	1 32	1 32	37	1 38	1 53	32	1 24	1 92	7	1 29	1 45
32	1 35	1 92	7	1 42	1 45	53	1 43	1 25	7	1 28	1 45	1	1 34	2 21
7	1 40	1 45	22	1 43	1 28	32	1 46	1 92	56	1 29	1 35	45	1 40	1 28
45	1 41	1 28	56	1 51	1 35	2	1 47	1 32	45	1 30	1 28	37	1 47	1 53
22	1 44	1 28	42	1 59	2 21	7	1 66	1 46	10	1 30	1 59	10	1 55	1 59
10	1 52	1 59	37	1 62	1 53	22	1 76	1 28	37	1 35	1 53	42	1 62	2 21
51	2 07	1 46	45	1 62	1 28	42	1 82	2.21	22	1 39	1 28	22	1 68	1 28
1	9 27	2 21	1	2 45	2 21	56	1 93	1 35	42	1 64	2 21	56	1 83	1 35

TABLE 7a

SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS, ALSO BETWEEN VARIANCES, OF THE 8 AM TO 4 PM PERIODS OF 11 SELECTED STATIONS, RURAL, COMPARED TO THE 24 HOURS OF THE TOTAL 18 STATIONS

Year	Type	Vehicle Loading	Value[a]	Axle 1 F	Axle 1 t	Axle 2 F	Axle 2 t	Axle 3 F	Axle 3 t	Axle 4 F	Axle 4 t	Axle 5 F	Axle 5 t	Total Vehicle F	Total Vehicle t
1958	13	Loaded	T			1.08	1.96							1.08	1.96
			C			1.16	2.57							1.09	2.14
1959	13	Loaded	T												
			C												
1958	21	Loaded	T		1.97	1.13	1.97		1.97						1.97
			C		2.58	1.14	3.42		2.34						2.36
1959	21	Loaded	T	1.22	1.98	1.22		1.22						1.22	
			C	1.44	3.63	1.32		1.26[b]						1.28	
1958	22	Loaded	T					1.13		1.13				1.13	
			C					1.18		1.25				1.22	
1959	22	Loaded	T	1.13		1.13			1.97					1.13	
			C	1.26		1.15[b]			2.00[b]					1.17[b]	

[a]T = theoretical and C = computed.
[b]Between 0.05 and 0.01 level of significance.
Note: Blank spaces indicate that computed values are smaller than theoretical values, thus the differences are "not significant."

TABLE 7b

SIGNIFICANCE OF DIFFERENCES BETWEEN MEAN WEIGHTS, ALSO BETWEEN VARIANCES, OF THE 24-HR PERIODS OF 11 SELECTED STATIONS, RURAL, COMPARED TO THE 24 HOURS OF THE TOTAL 18 STATIONS

				Axle										Total Vehicle	
				1		2		3		4		5			
Year	Type	Vehicle Loading	Value[a]	F	t	F	t	F	t	F	t	F	t	F	t
1958	13	Loaded	T												
			C												
1959	13	Loaded	T												
			C												
1958	21	Loaded	T				1.96		1.96						1.96
			C				2.35		2.05						2.10
1959	21	Loaded	T												
			C												
1958	22	Loaded	T	1.00	1.96	1.00		1.00	1.96	1.00				1.00	
			C	1.01	2.24	1.02		1.08	2.44	1.10				1.07	
1959	22	Loaded	T		1.96										
			C		2.51[b]										

[a]T = theoretical and C = computed.
[b]On 0.01 level of significance, table value t = 2.58.
Note: Blank spaces indicate that computed values are smaller than theoretical values, thus the differences are "not significant."

fied with the population of the 18 rural stations, all are located on the roads of the Interstate System. Station No. 51, however, showed no significance on F test but showed appreciable and consistent significance on t test. Table 5c and its inclusion in the population of stations on primary system other than Interstate would require additional investigation.

Although of no immediate importance because there was no intent to reduce the number of stations in Mississippi, the arrays gave promise as a methodology when sampling by populations is considered. As a further step in the possible application of these findings, a study was made of the 11 selected stations, comparing them with the total population of 18 stations. These analyses were made for the years 1959 and 1958.

The manner of presentation in Tables 7a and 7b is similar to that used in Tables 4a, 4b, and 4c. In Table 7a, most of the mean weights obtained at the selected stations for the period 8 a.m. to 4 p.m. did not differ significantly from the corresponding mean weights at 18 stations; others were borderline cases of significance on the 5 percent level. Axle 2 of vehicle type 21 in 1958 and axle 1 in 1959 show very significant difference on t test. As would be expected, better agreement was found in a comparison of 24-hr periods for the 11 selected stations with the total population of 18 stations, as given in Table 7b. The larger difference was significant on the 5 percent level but still nonsignificant on the 1 percent level for t values.

Inasmuch as the t test is sensitive to the precision of the mean — that is, the standard error of the mean — its practical implication sometimes is not self-apparent. When neither the t nor F test indicates significance, no further investigation may be needed for such uses as comparisons in trend studies, as the observed difference could be ascribed to chance variations. But when interest is evidenced in absolute numerical values, then these values should be observed from the point of view of their practical application. For instance, in Table 7a for axle 1, vehicle type 21, in 1959, t = 3.63, indicating that the difference between the mean weights was very significant and that the likelihood of such a difference being due to chance alone was highly improbable. On further investigation it was found that the standard error of the 11 station mean of 5,344 lb was quite precise, only 56 lb, and that the difference between the means was only 139 lb or 2.5 percent of the 18 station mean of 5,483 lb. This difference would not be considered too important by Mississippi for many practical uses.

CONCLUSIONS

The application of statistical method in the analysis of truck weights resulted in the decision by Mississippi to reduce, on a trial basis for one year, the truck weighing operations by limiting them to the 8 a.m. to 4 p.m. period. In addition, certain analytical methodologies have been successfully tried which might find further application in future search for other needed information. Certain areas where additional research is needed are delineated as the result of findings of this study.

1. Considering only the mean weights, it was found that for the vehicle types for which data were available in sufficient quantities it is possible to reduce the number of vehicles weighed and/or stations operated and to select an 8-hr period of weight station operations which would give mean weights representative of a 24-hr period during the summer months. Of the periods available for study, the period from 2 p.m. to 10 p.m. produced the best results, but the period from 8 a.m. to 4 p.m., although not quite so good, was considered satisfactory and was decided on for use in Mississippi because the increased accuracy obtained using the 2 p.m. to 10 p.m. period was not sufficient to justify the operating inconvenience. Any 8-hr operation can be expected to produce less accurate mean weight data than will a 24-hr operation for heavy, comparatively rare vehicles, and less accurate data for other characteristics such as numbers or frequencies of vehicles or axles loaded over the legal or recommended limits because these vehicles are encountered infrequently. Estimating the proportion of empty and loaded vehicles in the total traffic volume can be regarded as a separate problem which needs its own solution.

2. It was found that by using techniques of statistical analyses, weight stations can be identified by statistical populations and grouped accordingly, which affords the basis in developing maximum efficiency for obtaining meaningful average weights. In Mississippi it was indicated that 11 selected rural stations would provide representative mean weight data that had been previously obtained at 18 stations. However, for reasons peculiar to every individual station, it was decided to continue the weighing operations at all 18 rural stations. Because of this decision, it is expected that larger samples will be obtained than the indicated minimums for the types of vehicles which were investigated.

3. The study indicated that the samples of very heavy loads were too small to draw accurate conclusions. Because all heavy trucks were already weighed in accordance with the current practices to increase the sample of extremely heavy vehicles and those loaded above the recommended limits, it would be necessary to increase the number of stations or expand the schedules of operations, or find new means of obtaining the weight data.

4. The sampling procedures developed in Mississippi are designed for errors of estimate of ±5 percent on 95 percent confidence limit. It should be kept in mind, however, that for some purposes such as establishment of trends in mean weights, greater accuracy may be needed.

5. To develop optimum truck weighing procedures, in addition to the observations already made in Mississippi, some methods, yet unknown, need to be developed and measures provided for the other important elements discussed in this paper. For the benefit of future truck weighing studies, the nature of data needed to resolve problems arising from requirements of highway design and administration of highway transportation should be more precisely defined. Whether the frequency of occurrence of critical weights for design purposes, the extent of overloading above legal or specified limits for enforcement, or data on average and maximum weights for economic studies are needed, the tolerance limits for the accuracy of estimates should be established so that optimum sampling procedures can be properly designed.

A Study of Urban Travel Times In Pennsylvania Cities

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Because traffic congestion or vehicular delay is a logical factor to consider when programing new construction in urban areas, it becomes necessary to compare the degree of congestion at one location with that at any other location. Inasmuch as information is not available to make such comparisons, it was the purpose of this study to investigate methods by which relative traffic congestion (or vehicular delay) could be estimated at any given location when only limited information of the local conditions was available. This study was confined to urban state highways but does not include limited access expressways.

To develop estimating parameters it was necessary to measure the effect that many variable factors have on travel time, such as traffic volume, traffic controls and regulations, classification of streets, percentage of heavy vehicles, street width, type of area. Trip times were sampled through 15 test sections located in five cities. These sections varied in length from 0.3 to 1.5 miles. All data were recorded on Esterline Angus tapes as vehicles entered and left the test sections. Information was then transferred to IBM cards using a 650 computer for analysis.

It is believed that the relationship between travel time and volume/capacity ratio was the most useful method for estimating travel time. This study is limited to the problem of developing estimating parameters and does not include their application to a statewide evaluation of relative traffic congestion for programing new construction.

● IF TRAFFIC CONGESTION or vehicular delay is used as a warrant for programing urban highway improvements, it is necessary that the degree of congestion at one location be compared with the degree of congestion at any other location. Such a comparison is useful either in terms of accumulated vehicular delay or of total costs resulting from "excessive" delay.

Because such information is not available for use in statewide urban programing, it was the purpose of this study to investigate methods by which relative traffic congestion (or resultant vehicle delay) could be estimated at any given location when only limited information about the location is known.

Because vehicle delay in a given section is proportional to the time required to traverse the section, it follows that relative travel time could be a measure of relative congestion. Therefore, this study is concerned with travel time and the interrelated factors that affect it. Such factors include the classification of streets, types of area through which streets pass, traffic controls and regulations, traffic volumes, street width, etc.

Fifteen test sections were selected in five different cities ranging in population from 28,000 to 100,000 persons. These test sections were located in both central business districts (CBD) and adjoining intermediate areas. They included both local and arterial streets and one-way and two-way streets. Lengths of test sections varied from 0.3 to 1.5 miles. Trip times were determined by recording license numbers as vehicles entered and left the test section. Data were recorded on Esterline Angus tapes during both peak and off-peak hours for 30-min periods. Each 30-min period was broken down into five 6-min intervals and placed on IBM cards for analysis. A 650 computer program was written in which license numbers were matched and resultant travel times analyzed in a continuous operation.

Over-all results showed that mean travel time equaled 3.69 min per mi during periods with less than critical density and 6.12 min per mi during periods where critical density had been exceeded. Equivalent speeds are 16.3 and 9.8 mph, respectively.

A significant difference existed between mean travel time through central business districts and intermediate areas on both one-way and two-way arterial streets. However, the difference in travel time between one-way and two-way streets was not significant in either type of area.

The correlation between travel time and street width or travel time and percent heavy commercial vehicles was less than ten percent, indicating neither variable, separately, has much effect on travel time.

Traffic signals were timed for progressive movement in four of the fifteen locations studied. Although the mean travel time in coordinated sections was less than non-coordinated sections (3.50 and 3.75 min per mi, respectively), the difference was not statistically significant.

It was found that a stratightline relationship existed between mean travel time and signal density, below critical volume densities. Combining data from all locations, regression equations were developed for various volume/capacity ratio levels. Thus, by knowing only the average number of signals per mile, mean travel time could be estimated with reasonable accuracy, so long as critical densities were not exceeded.

The most useful parameter found was the relationship of travel time to volume/capacity ratio. Volume used in the ratio refers to the equivalent hourly volume (10 times actual 6-min counts) in a given test section, whereas capacity refers to the average of the practical capacities of individual intersections within the test section. By using this ratio, changes in traffic volume, signal timing and other variables affecting travel time, are combined into a single variable. This relationship is described by parabolic curves for one-way arterial streets, two-way arterial streets, and two-way local streets.

Although volume/capacity correlation with travel time is not as high as the correlation with signal density, the volume/capacity relationship can be used to estimate travel time above and below critical densities. On one-way arterials a coefficient of correlation of 0.65 was obtained, and on two-way arterials the correlation equaled 0.72. Standard error of estimate for each was 0.15. Although the coefficients are relatively low, in view of the many variables that affect travel time, it is believed that the parabolic equations developed for arterial streets will be adequate for estimating mean travel time for the statewide evaluation program. On two-way local streets, however, a coefficient of correlation of only 0.45 was obtained. Inasmuch as this was based on only twelve samples, it is believed that additional data should be collected to determine if a higher correlation exists or if other parameters than volume/capacity ratio should be used to estimate travel time.

OBJECTIVE

In programing urban improvements, it is usually desirable to expend funds at locations where the most relief from traffic congestion can be obtained per dollar spent. Thus, it is necessary to be able to evaluate relative traffic congestion and compare the intensity of congestion at one location with any other location. Such a comparison can be made in terms of average travel time, accumulated delay, "excessive" delay, or costs of excessive delay.

It is the purpose of this study to develop a method of estimating travel time on urban state highways, when only limited information of local conditions is available, such as contained in the 210 Needs Study. (Highway Needs Study of all road systems as required under Section 210 of the Federal-Aid Highway Act of 1956). To do this, it is necessary to measure the effect many variable factors have on travel time. Some of these factors include: traffic volumes, traffic controls, street width, percent of heavy commercial vehicles, classification of streets, type of area, direction of flow, etc.

If a relationship of travel time to any of these factors or combination of factors can be found by studying several different locations, then it would be possible to use this relationship, together with the information in 210 Needs Study to estimate travel time (and therefore the degree of existing congestion) on all urban state highways. This report, however, is limited to the problem of developing estimating parameters and does not include this application to statewide evaluation of relative traffic congestion and future programing of urban improvements.

DEFINITIONS

Arterial street. — A major state highway within an urban area, with little or no control of access, serving both "local" and "through" traffic; usually a continuous, U.S. numbered route.

Local street. — A minor street within an urban area, used primarily for access to abutting property and for "local" traffic; usually an unnumbered secondary state highway carrying very little "through" traffic.

Central business district. — The "downtown" area of the city where the abutting property is used principally for retail business and commercial purposes.

Intermediate area. — The area adjacent to the central business district where the abutting property is a combination of commercial, residential, or industrial land uses.

Critical density. — The density where the volume of traffic appeared to have reached the possible capacity of the test section.

METHOD

Many successful methods have been developed for measuring efficiency of traffic movement or relative traffic congestion on urban streets. Congestion ratings, which are often expressed in terms of travel time, occupancy time, or speed, usually relate actual values with optimum values.

The method investigated in this study, in which travel time is considered, is based on two previous studies: (a) the use of the volume/capacity ratio concept as a congestion index by Rothrock (1), and (b) the change in travel time as related to the changes in traffic volume by Rothrock and Keefer (2).

To measure the effect many variable factors have on travel time, 15 sections of streets were sampled in five cities in eastern Pennsylvania. Population ranged from

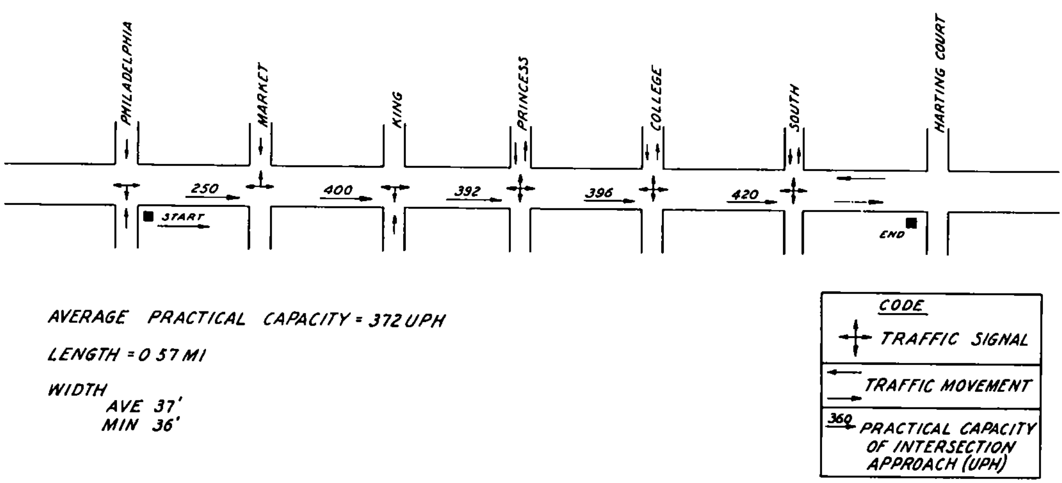

Figure 1. Typical urban test section.

28,000 to 100,000 persons. Six streets were located in CBD's and nine were in intermediate areas adjacent to CBD's. Seven were one-way streets and eight were two-way. Twelve streets were classed as "arterial", whereas three were classed as "local". The lengths of the test sections through which vehicle trips were timed varied from 0.31 to 1.54 miles. There were at least three signalized intersections within each test section.

Figure 1 shows a typical test section. The entrance (Station A) to the section is located on the outbound throat of the adjoining intersection, whereas the exist (Station B) is located midblock or at a non-signalized intersection. Traffic would normally be free flowing past each station.

Vehicles were timed in one direction only as they entered and left the test section. At a predetermined time, recorders were started simultaneously at both stations. The last three digits of license numbers were recorded on tape as each vehicle entered or left the test section. License plates containing letters in any of the last three places were not recorded, to simplify the problem of later matching license numbers. It was not necessary to record entrance and exit times of each vehicle because both tapes were started together and were moving continuously at a constant speed. Therefore, trip time for a given vehicle was the time difference recorded at Stations A and B. Because tape speeds were set at 6 in. per minute, there was little problem of writing three digits at the proper location on the tape at the instant a vehicle passed the station point. The maximum recording error observed was 2 sec. In addition to recording license numbers, all vehicles were counted and classified on tape as they passed each station.

Two operators were required at each station. Also, an observer was needed to observe traffic movements between Stations A and B, noting any unusual conditions that would affect trip time, particularly those periods when the critical density had apparently been reached or exceeded.

All measurements were made in clear, dry weather during both peak and off-peak hours and at times when there were no unusual obstructions to normal traffic flow. Data was collected continuously for 33- to 35-min periods. Each of these periods was later subdivided into five 6-min intervals and all data were transferred to IBM cards for analysis. A 650 computer program was written which would match the license numbers, compute the mean travel time of each 6-min interval and perform statistical analysis of the results. To later relate capacity to travel time, the practical capacity was calculated for each signalized intersection in all test sections, by the methods outlined in the Highway Capacity Manual (3). The average capacity computed for each test section is the average of the practical capacities of the signalized intersections within the section.

TABLE 1

MEAN TRAVEL TIME AT ALL LOCATIONS COMBINED

	Below Critical Density	Above Critical Density
Mean travel time (min/mi)	3.69	6.12
Standard error of estimate	0.10 (5.2%)	0.20 6.6%)
Confidence level (%)	95	95
No. of 6-min sample	118	17
Equivalent mean speed (mph)	16.3	9.8

ANALYSIS OF DATA

Previous studies by Berry (4) indicated that during congested conditions, at least 36 license matches are needed on two-lane urban streets, whereas 102 matches were needed on multi-lane streets to determine mean travel time within 5 percent error at 95 percent confidence level. A somewhat smaller number of matches are needed during uncongested conditions.

Because it was intended to expand the 6-min counts to equivalent hourly volumes, it was believed that at least six license matches in each 6-min interval (or 60 per hr) would be necessary to achieve 90 percent accuracy or better. Therefore, with few exceptions, all 6-min periods with less than 6 samples were discarded. The exceptions were those in which the individual travel times were nearly equal and were well-spaced over the 6-min period.

A total of 10,899 vehicles passed through the entering stations (Station A) during all checks. License numbers were recorded for 6,100 vehicles at Station A from which 1,550 valid license matches were obtained. Thus, the over-all sample amounted to 14 percent of the traffic stream. Of the 190 6-min intervals, 135 contained sufficient samples to warrant further analysis. Although the usable number of license matches averaged slightly higher than 11 for each 6-min interval, actual matches varied from three to 26 per interval. Of the 135 6-min intervals used in the analysis, 17 occurred during highly congested periods when the critical density appeared to have been exceeded. It was necessary to establish a maximum allowable travel time for each test section to eliminate the short-time parker from the data analysis. Maximum allowable times were generally set at 2½ times the estimated average travel time.

Combining data at all locations, the over-all mean travel time, below critical density, equaled 3.69 min per mi which is equivalent to 16.3 mph (Table 1). Above critical density the mean travel time was 6.12 min per mi or 9.8 mph. Standard errors of estimate were less than 7 percent at 95 percent confidence levels in both instances.

It was desired, however, to determine the relative effects each variable had on travel time and, if possible, derive a general parameter which would express these effects. Such a parameter would be used to estimate travel time on urban highways when a limited amount of information was known about the highways. The variable factors studied were (a) the type of area through which the street passes (that is, CBD and intermediate urban sections adjacent to the CBD), (b) street type (arterial or local), (c) direction of flow (one-or two-way), (d) street width, (e) traffic volume, (f) percent of heavy commercial vehicles, and (g) traffic signal coordination.

TABLE 2

EFFECT OF LOCATION AND STREET TYPE ON TRAVEL TIME (BELOW CRITICAL DENSITY)

Location	Type Street	Mean Travel Time (min/mi)	Equivalent Mean Speed (mph)
Central business district	Arterial (one-way)	4.45	13.5
	Arterial (two-way)	3.94	15.2
Intermediate area	Arterial (one-way)	3.12	19.2
	Arterial (two-way)	3.07	19.5
	Local (two-way)	4.22	14.2

Area, Direction of Flow, and Street Type

Table 2 gives a comparison of travel times in the CBD and intermediate areas adjoining the CBD. Because an equal proportion of samples taken above the critical density to the total number of samples was not obtained from each street type, travel times included in this table are only those which occurred during periods below the critical density. Within the CBD there was no significant difference in mean travel time between one- and two-way arterial streets, although, surprisingly, traffic moved slightly faster on two-way arterials. Also, there was no significant difference between travel times on one- and two-way arterials in intermediate areas. However, in comparing travel times in CBD's with intermediate areas, a significant difference did exist on both one-way and two-way arterials. Of the three local streets checked, all were two-way, located in intermediate areas. As would be expected, the travel time on local streets was higher than on arterials and the difference in mean travel times was significant.

Heavy Commercial Vehicles

To examine the effect of heavy commercial vehicles on travel time, the three classes — one-way arterial, two-way arterial, and two-way local — were again used except that no differentiation was made between CBD and intermediate areas.

Although the volume of commercial vehicles reached as high as 28 percent of total traffic during individual 6-min periods, the mean for each of the three classes was less than 10 percent. On local streets the coefficient of correlation of travel time with heavy commercial percentage equaled 0.23, indicating that only 6 percent of the variation in travel time could be attributed to heavy commercial vehicles. To reduce the effect changes in volume might have on travel time for constant commercial vehicle percentages, a correlation was made at various volume/capacity ratio levels. It can

TABLE 3

EFFECT OF HEAVY COMMERCIAL VEHICLES ON TRAVEL TIME (BELOW CRITICAL DENSITY)

Type Street	Volume/ Capacity Ratio	No. of Samples	Mean Travel Time (min/mi)	Commercial Vehicles (%)	Coefficient of Correlation
Arterial (one-way)	Less than 0.4	5	3.11	8.8	0.10
	0.4-0.59	22	3.45	7.3	0.28
	0.6-0.79	18	4.52	7.4	0.21
	0.8-0.99	9	4.86	7.5	0.12
Arterial (two-way)	Less than 0.4	-	-	-	-
	0.4-0.59	-	-	-	-
	0.6-0.79	14	2.67	7.2	0.25
	0.8-0.99	9	3.81	7.1	0.01
	1.0 & over	7	4.32	7.1	0.06
Local (two-way)	0.37-1.0	12	4.21	7.9	0.23

TABLE 4

EFFECT OF STREET WIDTH ON TRAVEL TIME (BELOW CRITICAL DENSITY)

Type Street	No. of Samples	Street Widths (ft)	Mean Travel Time (min/mi)	Coefficient of Correlation	Equivalent Speed (mph)
Arterial (one-way)	49	35, 40, 47, 48	3.92	0.23	15.3
Arterial (two-way)	31	37, 43, 45, 49	3.46	0.02	17.3
Local (two-way)	8	35, 36	4.21	0.52	14.2

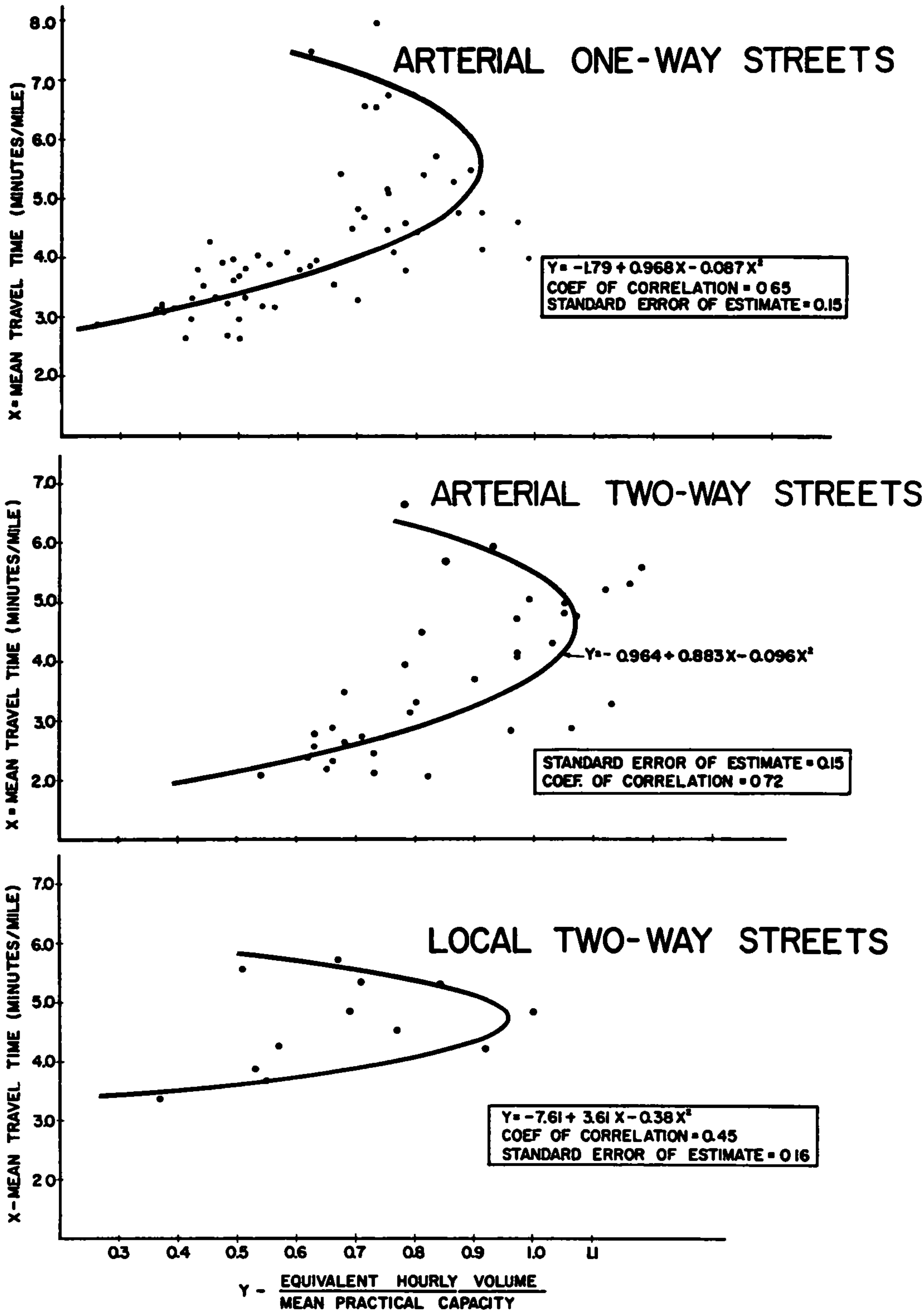

Figure 2. Relationship between travel time and volume/capacity ratio.

be seen (Table 3) that the highest correlation coefficient is 0.28 which indicates only an 8 percent effect on travel time. Thus it can be concluded that if a linear relationship is assumed, commercial vehicles had little influence on travel time within the volume percentages occurring during these studies. It should also be pointed out, however, that with the exception of one short 4 percent grade, all test areas were nearly flat.

Street Width

It was generally expected that as street width increased, speeds would increase, as does capacity. Table 4 gives the relationship between street width and travel time. On local streets the coefficient of correlation was 0.52. This, however, was based on only two streets whose widths were nearly equal. On arterial streets the highest coefficient was 0.23 which indicates that less than 5 percent of the variation in travel time could be explained by the change in street width.

Signal Coordination

Of the 15 locations studied, it was found that only four streets had signals timed for progressive movement. The remaining locations were either partially coordinated or

TABLE 5

EFFECT OF SIGNAL COORDINATION ON TRAVEL TIME (BELOW CRITICAL DENSITY)

Signal System	Volume/ Capacity Ratio	No. of Samples	Mean Travel Time (min/mi)	Equivalent Speed (mph)
Coordinated	Less than 0.4	2	3.26	18.4
	0.4-0.59	14	3.55	16.9
	0.6-0.79	14	3.12	19.2
	0.8-0.99	9	4.16	14.4
	1.0 & over	2	3.08	19.5
	Over-all	41	3.50	17.2
Non-Coordinated	Less than 0.4	5	3.14	19.1
	0.4-0.59	15	3.24	18.5
	0.6-0.79	30	3.62	16.6
	0.8-0.99	19	4.14	14.5
	1.0 & over	8	4.68	12.8
	Over-all	77	3.75	16.0

were not coordinated at all. Table 5 gives a comparison of travel times when critical densities were not exceeded. The mean travel time in coordinated sections was slightly lower than non-coordinated sections; nowever, the difference was not statistically significant. In examining the four coordinated streets further, it was noted that 3 were arterials, one of which was in the CBD. Two were one-way and two were two-way. The signal density on the coordinated streets averaged 7.1 signals per mile as compared to 7.3 per mile on non-coordinated streets.

Signal density was found, as will be discussed later, to have high correlation with travel time. The results of signal coordination were somewhat unexpected particularly at the lower volume/capacity ratio levels. Walker (7), in his discussion of probable effects of coordination on speed, concluded that a wide range of over-all speeds could be expected near the level of possible capacity in either system and there would be little, if any, difference between coordinated and non-coordinated systems.

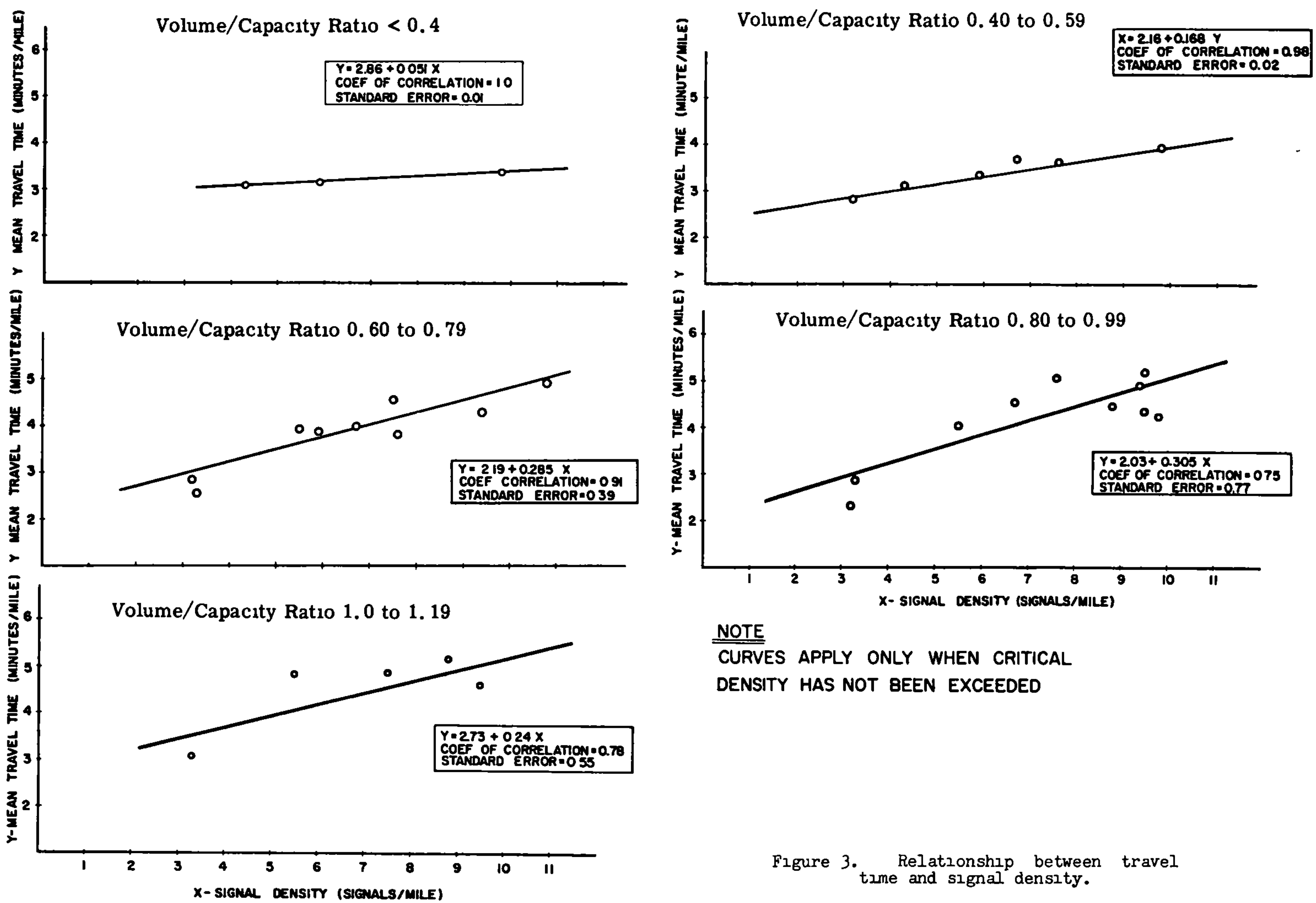

Figure 3. Relationship between travel time and signal density.

Volume/Capacity Relationship

Rothrock and Keefer (2) related changes in travel time to changes in traffic volume. Inasmuch as their study was limited to one location, capacity remained constant and could be neglected. Because of the variation of capacities in the 15 test sections in the study, it became necessary to use a volume/capacity ratio rather than volume alone. For each of the 15 sections, the volume refers to the equivalent hourly volume (ten times the actual 6-min interval count), whereas the capacity refers to the average of the practical capacities of each signalized intersection in the test section in direction of flow being measured. Practical capacities were calculated by the method outlined in the Highway Capacity Manual. Turning movements at each intersection were based on 10-min traffic counts. Thus, by using the volume/capacity ratio, many of the variables which might affect travel time were combined into one variable, which would permit ready comparison of one section with another.

Although it was found that type of area significantly affected travel time on a given class of street, it would not be possible to determine the limits of a CBD from the adjacent intermediate areas without an extensive field survey. Therefore, for future application to a statewide evaluation of urban highways, type of area, as a variable, was neglected and travel times were related to volume/capacity ratio for each of the 3 classes of streets regardless of their location. Parabolic curves were fitted to this data and estimating equations were developed (Fig. 2). For each type street the travel time increases with increase in volume/capacity ratio until the apparent critical density is reached. At that point the travel time continues to increase, although the volume/capacity ratio decreases. This characteristic is discussed by Greenshields (5) and was found in studies by Huber (6) as well as Rothrock and Keefer (2).

Considerable scatter occurs particularly at high volume/capacity ratios near the point of critical density. This wide variation in travel times is believed to have been caused when saturation occurred for short periods of time (for example, one to two minutes) or within only a short portion of the test section. Either of these conditions would cause relatively high mean travel times for some 6-min intervals as compared to other intervals at the same volume/capacity ratio where these conditions did not occur. In both instances the travel times were recorded as occurring below critical density because the major portion of test sections was not saturated for the full 6-min interval. A second cause for scatter was the variation of volume within the test sections. The volume used in determining the ratio was based on the volume entering the section at Station A. Thus, if the actual average volume of the test section varied greatly from the Station A volume because of turning movements between Stations A and B, resultant changes in volume which affected travel time would not be accounted for in establishing the volume/capacity ratio. A third cause of scatter might be the wide range of practical capacities that existed at individual intersections within a given test section. For example, at one location the average capacity was 490 vehicles per hour although the range varied from 204 to 798 vehicles per hour at the individual intersections. As volume through the section increased, low capacity intersections would become congested first, causing greater over-all delay than would occur at locations where the individual intersection capacities were more nearly equal to the average.

Other conditions, such as double parking and left turns, directly cause a difference in travel times under otherwise apparently similar conditions. It is believed, therefore, that the correlation obtained between travel time and volume/capacity ratio for arterial streets is reasonably good.

The curves for both one-way and two-way arterial streets are similar in general shape, although travel time on one-way streets was higher than on two-way streets for a given volume/capacity ratio. The apparent critical density on two-way arterials occurred slightly above a ratio of 1.0, whereas on one-way streets it occurred just below a ratio of 1.0. Theoretically, the critical density would be expected to occur near the point of possible capacity which would be equivalent to a volume/capacity ratio of 1.2.

The curve for local streets is flatter than those for arterial streets. This would indicate that after the critical density had been reached, the entering volume decreases rapidly although the rate of increase in travel time is slight. This curve, however, is based on only twelve 6-min intervals. The coefficient of correlation was 0.45. Additional study of local streets is needed to determine if a higher correlation exists or if parameters other than volume/capacity ratio would better describe the variation of travel time.

Signal Density

Traffic signals are known to be the greatest single cause of delay to through movement on urban streets. In examining the over-all effects of traffic signals on trip times, it was found that a straightline relationship existed between mean travel time and signal density expressed in number of signals per mile, so long as the critical density was not exceeded. Figure 3 shows this relationship at various volume/capacity levels. Data for all 3 classes of streets were combined and regression equation were developed as shown. As would be expected, travel time increased as signal density increased, although it should be noted the rate of increase below a volume/capacity ratio of 0.4 is much less than rates above 0.4.

Of the five volume/capacity ratio levels examined, the lowest coefficient of correlation (0.75) was obtained at the 0.80-0.99 level. The highest standard error of estimate (0.77) also occurred at this level. However, it appears that travel time could be estimated reasonably well over a given section of urban streets based on the average signal density in that section, regardless of the other variables that exist. This relationship is limited to volume levels below critical density. Further study would be required to determine if a similar relationship existed above critical density.

CONCLUSIONS

Results of this study lead to the following conclusions:

1. The volume/capacity ratio is a suitable parameter for estimating travel time on any given urban highway.
2. The parabolic curves developed in this study relating travel time to volume/capacity ratio for arterial streets can be applied to the statewide evaluation of traffic congestion on urban state highways.
3. Although a parabolic curve is developed for local streets, more study should be made of local streets before using this curve because of the low correlation obtained.
4. Traffic volume and signal timing have major effects on travel time, whereas other measurable quantities such as street width, percent of heavy commercial vehicles, direction of flow, and type of area have minor effects, when considered as separate variables.
5. Travel time along any section of urban highway is directly proportional to the average number of traffic signals per mile in that section so long as the critical traffic density has not been reached. Thus, signal density could be used to estimate mean travel time within this limitation. More study is needed to determine the relationship above critical density.

ACKNOWLEDGMENTS

The author is grateful to other divisions of the Pennsylvania Department of Highways for providing personnel necessary to complete this study. A total of fourteen persons were loaned by the Highway Planning, Traffic Engineering Bureau, and Federal-Aid Sections during various stages of the study. Mr. Evan H. Gardner, Director of Economic Research, wrote the 650 computer program which aided in the analysis of field data.

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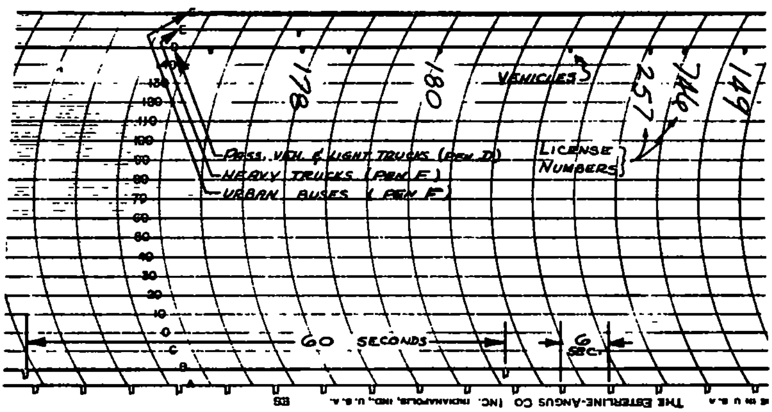
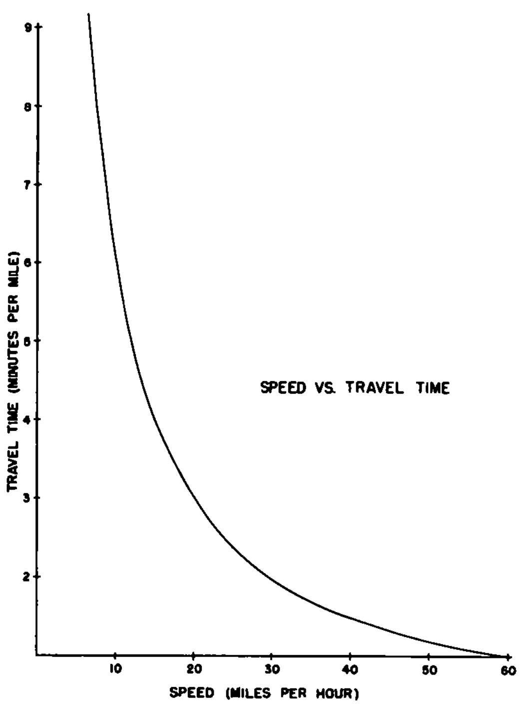
Appendix



SAMPLE TAPE SHOWING FIELD DATA



STATION A (TYPE R CARD)

Words: 0001 (cols. 1–10), 0002 (cols. 11–20), 0003 (cols. 21–30), 0004 (cols. 31–40)

STATION	DATE	PERIOD			LICENSE NO		TIME HR MIN SEC		MAX TIME ALLOWED
52	08	01			194		12 18 16		0500

STATION B (TYPE Q CARD)

Words: 0001 (cols. 1–10), 0002 (cols. 11–20), 0003 (cols. 21–30), 0004 (cols. 31–40)

STATION	DATE	PERIOD			LICENSE NO		TIME HR MIN SEC	
52	08	00			194		12 21 39	

OUTPUT CARD (ONE VEHICLE)

Words: 0027 (cols. 1–10), 0028 (cols. 11–20), cols. 21–30

STATION	DATE	PERIOD	TIME HR MIN		LAPSED TIME MIN.		
52	08	01	12 00		0323		

CONTROL CARD (INPUT) (ONE 6 MINUTE PERIOD)

Words: 0001 (cols. 1–10), 0002 (cols. 11–20), 0003 (cols. 21–30), 0004 (cols. 31–40), 0005 (cols. 41–50), 0006 (cols. 51–60)

STATION	DATE	PERIOD	TIME HR MIN		EQUIV VOLUME (VPH)		CAPACITY (VPH)	NO. OF PASS. CARS		NO. OF TRUCKS	NO. OF BUSSES		TOTAL VEH		LENGTH OF SECTION
52	08	81	12 00		0510		0865	046		005	000		051		0059

OUTPUT CARD (ONE 6 MIN. PERIOD)

Words: 0027 (cols. 1–10), 0028 (cols. 11–20), 0029 (cols. 21–30), 0032 (cols. 51–60), 0033 (cols. 61–70), 0034 (cols. 71–80)

STATION	DATE	PERIOD	TIME HR MIN		MEAN TRAVEL TIME (MIN/MILE)		EQUIV VOLUME (VPH)		PERCENT TRUCKS		PERCENT BUSSES		PERCENT PASS. CARS
52	08	81	12 00		0393		0510		0981		0000		9019

STATISTICAL OUTPUT CARD (ONE 6MIN PERIOD)

Columns 1–50

STATION	DATE	PERIOD	HR MIN		MEAN TRAVEL TIME (MIN/MILE)		STANDARD DEVIATION		$\sqrt{\text{NO. OF SAMPLES}}$		STANDARD ERROR
52	08	81	12 00		0393		08490		40000		2122

SUMMARY OF DATA — CHART NO. 1

City	Code	Street Name	Type of Area	Class of Street	Direction of Flow	Length of Section (Mi.)	Width (Ft.) Average	Width (Ft.) Minimum
York	11	Philadelphia	CBD	Arterial	One Way	0.45	35	32
	12	Princess	Inter.	Local	Two Way	0.67	35	32
	13	George	CBD	Arterial	One Way	0 66	48	46
	14	George	Inter	Arterial	One Way	0 51	48	48
	15	Queen	Inter.	Arterial	Two Way	0.57	37	36
Ardmore	21	Lancaster, Pa.	CBD	Arterial	Two Way	0.74	43	40
Chester	31	Ninth St.	Inter.	Local	Two Way	0.82	36	36
Lebanon	51	Cumberland	CBD	Arterial	One Way	0.56	40	38
	52	Walnut	Inter	Arterial	One Way	0.70	40	40
	53	Chestnut	CBD/Inter.	Local	Two Way	0.42	37	32
Harrisburg	62	Second	CBD	Arterial	One Way	0.53	47	43
	63	Market	CBD	Arterial	Two Way	0 54	49	48
	64	Paxton	Inter.	Arterial	Two Way	0.91	45	44
	65	Front/Paxton	Inter.	Arterial	One Way	1 54	43	30
	66	Forster	Inter.	Arterial	Two Way	0.52	84*	84

* 42' Each Side of 10' Median

Traffic Signals				Practical Capacity (Veh./Hour)			Below Critical Density	
Number	Number Per Mile	Progressive Timing	Cycle Length (Sec.)	Average	Maximum	Minimum	Mean Travel Time (Min.)	Equiv. Speed (MPH)
3	6.7	Partial	37 5o 54	935	990	895	4.01	14.9
5	7.5	Partial	45 to 54	297	360	234	4.72	12 7
5	7.6	Partial	45 to 56	870	1030	684	4.52	13.3
3	5.9	YES	45	1150	1380	980	3.48	17 2
5	8.8	Partial	45 to 53	372	420	250	4.94	13.4
6	9.5	Partial	45 to 60	517	621	398	3.62	16.6
8	9.8	YES	50	490	798	204	3.29	18.2
1	10.8	Partial	50 to 70	703	846	614	4.92	12.2
3	4.3	Partial	45 to 70	1062	1175	970	3.11	19.3
4	9.5	None	50 to 70	424	465	381	3.95	15.2
5	9.4	YES	80	1077	1250	890	4 82	12.4
3	5.5	Partial	70	411	447	346	4.26	14.1
3	3.3	YES	50	887	980	740	2.69	22.3
5	3.2	Partial	70	1284	1460	1160	2.67	22 5
4	7.7	Partial	70	905	1340	560	3.36	17.9

Above Critical Density	
Mean Travel Time (Min.)	Equiv. Speed (MPH)
6.91	8.7
6.65	9.0
5.50	10.9
5.38	11.1
6.01	10.0
7.25	8.3
5.93	10.1

Effects of Commercial Roadside Development On Traffic Flow in North Carolina

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This study reveals the magnitude and severity of the problem of commercial roadside development in North Carolina. It attempts to evaluate the effects of this development by means of a study of speed and delay through nine selected test sites.

An attempt was made to define and measure the effect of the various elements which influence traffic operation. Results are based on data collected through the use of two test-car techniques — the average-car method and a newly developed maximum-car method. Various mathematical relationships, which facilitate the prediction of speeds and turning movements along any section of commercially developed highway, were derived and field tested. Such an evaluation may be utilized by highway planners to help determine the desirable degree of access control for proposed facilities or the need for redesign and modification of existing facilities.

● THE heavily traveled highways leading into and around many cities have been reduced in capacity and, in some cases, abondoned as major, traffic-moving facilities because of the congestion and delay brought about by commercial roadside development. A knowledge of the various elements of the congestion and their individual and collective effects on the adjacent traffic stream is necessary to determine needed improvements and solutions to the problem.

Communities generally expand along the traffic arteries which serve them. From the point of view of the businessman, parcels of land adjacent to these major streets and highways make desirable locations for business establishments, but commercial development is not always in the best interest of the motoring public. A section of highway which is bounded by commercial development is frequently plagued by several conditions which are undesirable to the motorist desiring to use the facility as a major thoroughfare. Among these are (a) turning movements of various kinds, (b) delays to vehicular traffic, (c) distractions to the driver, and (d) an increasing accident rate. The accident rate is directly affected by the magnitude of the first conditions.

It is readily apparent that all roads and streets cannot have limited control of access. Decisions must be made concerning which facilities should serve adjacent land use and which should primarily serve the movement of traffic.

The problem of commercial roadside development has been solved in the past by the construction of additional highways which bypass the congestion and accompanying hazards encountered in the fringe areas of many cities. If, however, land use and highway access are not controlled along the new facility, congestion and hazards similar to those on the previous location may seriously impede the highway traveler. If and when this occurs, another bypass may be needed. It is obvious that successive construction of bypasses cannot be considered as an economical or otherwise feasible solution to the problem created by commercial roadside development.

As an outgrowth of a more comprehensive investigation of the effects of commercial roadside development on traffic flow in North Carolina, this report attempts to describe these effects primarily by means of a study of speed and delay through several selected sites. The comprehensive investigation (Highway Research Project ERD-110-B) was transacted within the Highway Research Program conducted by the Department of Civil Engineering at North Carolina State College. It was completed in June 1960, following two years of research in cooperation with the North Carolina State Highway Commission and the U.S. Bureau of Public Roads.

Roadside development at several selected locations in North Carolina was studied, and an attempt was made to define and measure the effect of the various elements of roadside development which influence traffic operation. In particular, the manner and degree to which they affect traffic movement were considered.

Results of this investigation are based on data collected through the use of two test cars. The average speeds of traffic in commercially developed and adjacent undeveloped sections of highway were determined by making successive trips through these sections with an average car. The maximum speeds in the same sections were determined in a similar manner through the use of a maximum car. Other major tools used in this investigation include statistical analyses of elements affecting traffic flow and manual counts of the number of turning movements generated within a commercially developed section of highway.

Investigations of a similar nature may be used in the future by highway engineers who desire to justify such engineering decisions as: (a) the desirable degree of control of access for new facilities, (b) the partial control of access and design of traffic devices for existing facilities encumbered by commercial development, and (c) the desirability of relocating existing highways which have intensive commercial roadside development.

DEFINITIONS

Test car. — Either of the cars used to measure the average speed or the average-maximum speed.

Average speed. — The over-all-average speed of the vehicles in the traffic stream, as measured by the average-car method.

Developed section. — That portion of a test site, approximately one-half mile in length which has commercial roadside development and complete freedom of access.

Undeveloped section. — That portion of a test site, approximately one-half mile in length which has no commercial development or development of such a nature as to cause negligible delay to the traffic stream.

Average car. — A test car driven at a speed which, in the opinion of the driver, is representative of the average speed of the stream of traffic in which the test car is being driven.

Maximum speed. — A speed measured by the maximum-car method which is the average-maximum speed at which a section of highway may be traversed consistent with the maximum-legal speed limit and necessary safety measures.

Maximum car. — A test car whose driver is instructed to drive at a speed as near the maximum-legal speed limit as is consistent with necessary safety measures.

Test site. — A length of highway which contains one developed section and one undeveloped section with a transition section between.

Test. — Fifteen round trips through a site by both the average car and maximum car.

DESIGN OF EXPERIMENT

It was assumed at the beginning of this research endeavor that the average speed of traffic along a section of highway affords one measure of the delay and congestion present on that particular section. However, the average speed of traffic is the collective result of many individual elements which influence drivers. To be included among such elements are (a) the volume of traffic present; (b) the legal speed limit in force on the highway; (c) the geometric design of the facility, including lane width, shoulder width, horizontal and vertical alignment, number of lanes, intersections, and other geometric characteristics; (d) the devices used for traffic control such as signals, markings, and signs; and (e) the degree of abutting commercial, residential, or rural development. Therefore, to use average traffic speed as a measure of the delay and congestion caused by commercial roadside development, it was desirable to eliminate or hold constant, from one test section to another, all elements except roadside development which influence the speed at which the driver will travel.

All of these conditions obviously could not be met. Nevertheless, nine test sections were found which adhered to a preselected definition. A test section was defined as a section of commercially developed two-lane road at least one-half mile in length with a maximum-legal speed limit of 55 mph; medium to heavy traffic volumes; and free from traffic signals, major intersections, or appreciable horizontal or vertical curves.

The general location of all the test sites used in this investigation is shown in Figure 1. A map of the developed section of a typical test site including some of the physical characteristics is shown in Figure 2.

The investigation included the following phases: (a) pilot studies to develop methods

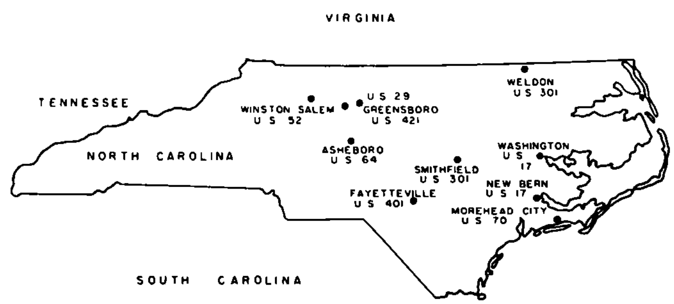

Figure 1. Location of test sites.

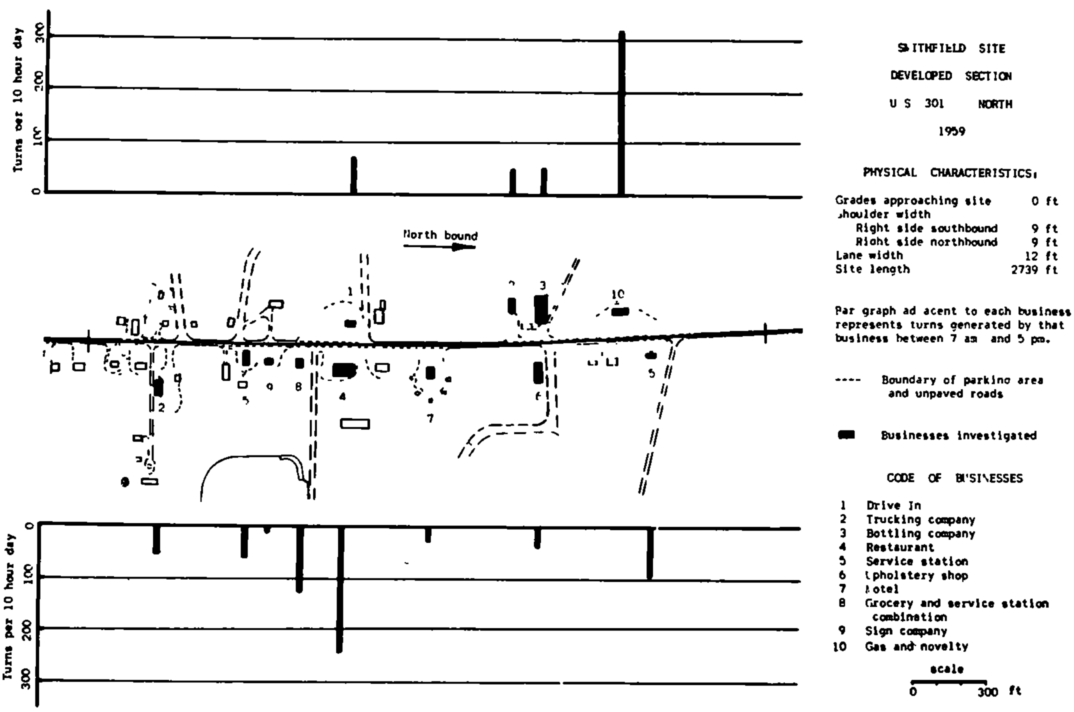

Figure 2. Developed section at Smithfield.

for collecting and analyzing data; (b) a turning movement study to determine the turning movement generation characteristics for various types of businesses; (c) a speed and delay study to analyze the effects of different types of impedances on the traffic stream; (d) an analysis of turning movements versus gross income of various types of roadside businesses; and (e) a road-user cost study to determine the economic losses to the highway user resulting from commercial roadside development. This report includes only the findings pertaining to the turning movement and the speed and delay studies.

SPEED AND DELAY STUDY

Pilot studies conducted at the beginning of this investigation indicated a need for a more detailed study of the elements affecting traffic operation. The degree to which different types of turning movements and other delaying situations affect the speed of the traffic stream was unknown. A vehicle making a left turn from a two-lane road during high traffic volumes probably would create more delay than a number of right turns occurring under the same conditions. However, a driver, under similar conditions, preferring to drive at a speed less than the average or safe and legal speed might delay the traffic stream more than either the left or right turns.

It was noted in the average speed study that commercial roadside development causes a measurable quantity of delay, but data collected by the average-car method did not differentiate between delays caused by physical traffic movements and delays caused by psychological factors. Physical delays are defined as those delays caused

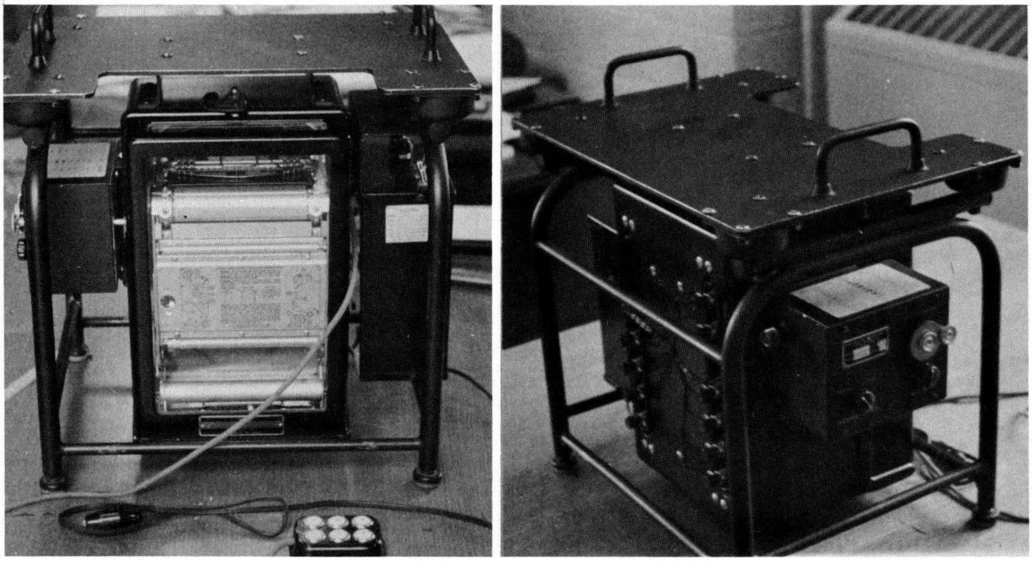

Figure 3. The speed and delay recorder.

directly by the turning movements or other conflicting movements associated with roadside development. Psychological delays are those which are caused by the driver's mental attitude towards congestion or his desire to drive slowly for other reasons. For example, the driver may be anticipating physical traffic movements which may not materialize or he may be searching for a particular business establishment.

Development of Maximum-Car Technique

A technique, later referred to as the maximum-car method, was developed for measuring the speed reduction resulting from physical impedances only. The speed measured by the maximum-car method is the average-maximum speed at which a section of highway may be traversed consistent with the maximum-legal speed and necessary safety measures. It is believed that a test car driven in this manner is subjected only to physical delays and that delays resulting from psychological factors are reduced to a minimum. It is imperative that the driver of the maximum car understand and be constantly aware of the definition and purpose of the maximum-car-tests. Otherwise, the data collected approaches that collected by the average-car method.

The average-maximum speeds determined by the maximum-car method were considered only as a standard and not as an absolute value. It was realized that drivers not adhering to the definition would produce different average-maximum speeds because of varying reactions to psychological factors. For this reason, the same driver was used during all maximum-car tests conducted during this investigation. Test cars with state emblems or permanent-type license plates tend to arouse suspicion among other drivers, resulting in slower speeds and biased data. Because of this the average car and maximum car used in this and subsequent phases of the investigation had no official markings. Further, in order not to attract the attention of other drivers, the test cars entered the traffic stream from points one-fourth to one-half mile beyond the end of the site.

If data from more than one source are to be compared, it is essential that the data be collected under similar conditions. Variations in the data resulting from changes

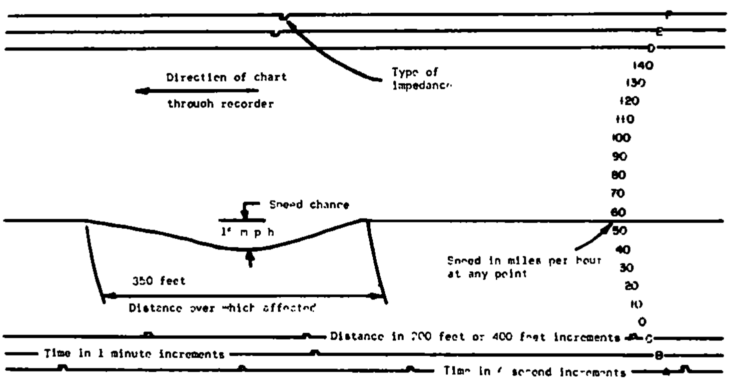

Figure 4. Speed and delay recorder chart with hypothetical speed curve.

in character and volume of traffic were eliminated in this study. This was accomplished by operating the average and maximum cars simultaneously. Thus, 15 round trips through a site resulted in 30 passes through both the developed and undeveloped sections for both the average and maximum cars.

The sample size of 30 passes for each of the cars was selected after consideration of several factors. First, in order that the maximum-car data be comparable to that of the average car, it was necessary that both test cars be subjected to the same traffic stream. Second, the time required to complete 30 passes through both sections is slightly less than two hours. Volume changes and driver fatigue become critical at this point.

Because the problems associated with commercial roadside development are most critical during periods of high traffic volume, tests for this entire investigation were conducted during either the morning, noon, or evening rush hours.

The purpose of the speed and delay study was to determine the relative significance of the various types of turning movements and other delay-causing factors and also to determine the location and average magnitude of these delays.

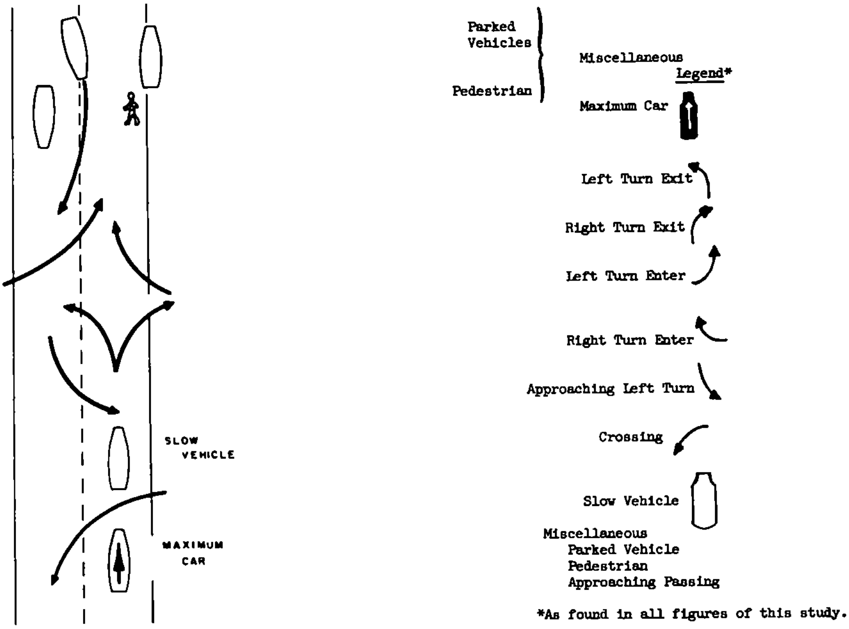

Figure 5. Types of impedances studied and legend used for presenting results.

Recording Instrument

An automatic speed and delay recorder (Fig. 3) was installed in the maximum car for the purpose of collecting extensive data concerning the various types of impedances which affect the speed of the maximum car. The speed and delay recorder registers these data in an accurate and useful form.

A speed and delay recorder is a machine which has a rotating drum over which passes a continuous chart. It is equipped with a speedometer-indicator hand on the end of which is attached an ink pen which records the instantaneous and continuous speed of the test car in which the machine is installed. The recording drum of the machine is turned by a connection with the regular speedometer cable of the test car, so that it rotates at a speed which is proportional to the speed of the test car. The

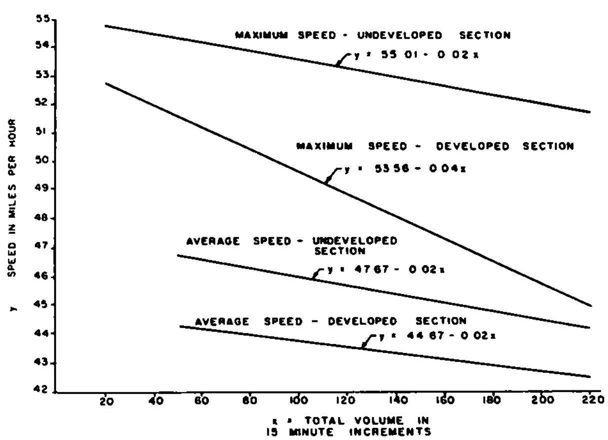

Figure 6. Speed versus volume curves.

recorder is also equipped with six additional pens which mark the chart with an ink blip when any one of them is activated by an electrical impulse. The purpose of three of the six pens is to enable a rider in the maximum car to code various incidents which may take place as a pass is made through the site. Two of the other three pens mark ink blips on the chart at 6-sec and 1-min intervals. The sixth pen is used to indicate either 200- or 400-ft distance increments.

A short section of the chart used in the speed and delay recorder is shown in Figure 4. The hypothetical speed curve shown in Figure 4 reflects the speed change induced by a delay-causing turn. The impedance causing this delay was coded on the two lines, D and F. Lines D, E, and F are the lines on which are coded the various types of

TABLE 1

RESULTS OF LEAST SQUARES ANALYSIS FOR SPEED VERSUS VOLUME

Test Car		Intercept	Slope	90% Confidence Limits on Y'		Standard Deviation,	Correlation
Type	Section	a	b	at $X = \bar{X}$	at $X = \bar{X} \pm 40$	$s_{Y/X}$	Coefficient
Avg.	Developed	44.67	-0.02	0.16	0.22	1.13	-0.43
Avg.	Undeveloped	47.67	-0.02	0.56	0.77	3.33	-0.22
Max.	Developed	53.56	-0.04	0.37	0.50	3.68	-0.43
Max.	Undeveloped	55.01	-0.02	0.08	0.11	0.46	-0.91

impedances. On lines C, A, and B are marked the 200- or 400-ft increments, the 6-sec intervals, and the 1-min intervals, respectively. The distance scale on line C is a constant scale of 1 in. of chart equal to 100 ft traveled by the test car. The chart moves through the recorder at a variable speed; thus, the time scale on lines A and B is variable.

The speed and delay recorder was used to record the following data:

1. Type of impedance.
2. Speed change from the maximum-legal speed.
3. Speed change from the previous speed of the test car.
4. The distance over which the speed of the maximum car was affected by the impedance.
5. The seconds of delay to the maximum car caused by the impedance.

These data can be read directly from the speed and delay recorder chart (Fig. 4)

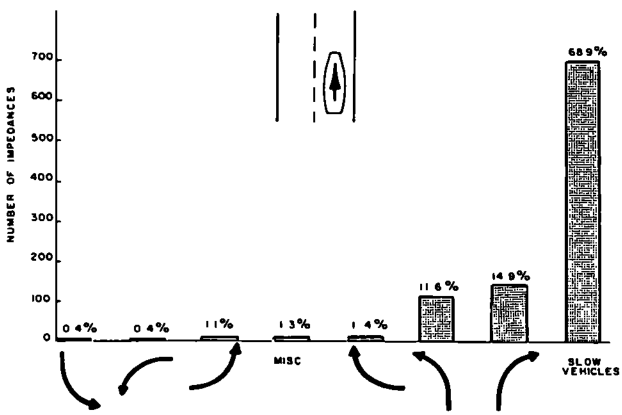

Figure 7. Frequency of occurrence of different types of impedances.

with the exception of the seconds of delay to the maximum car. This requires a simple calculation. First, the travel time, in seconds, is computed for the distance over which the speed of the maximum car was affected, had the impedance not occurred. Second the actual travel time over the same distance, in seconds, is scaled directly from the chart. The difference between these two travel times is the seconds of delay caused by the impedance.

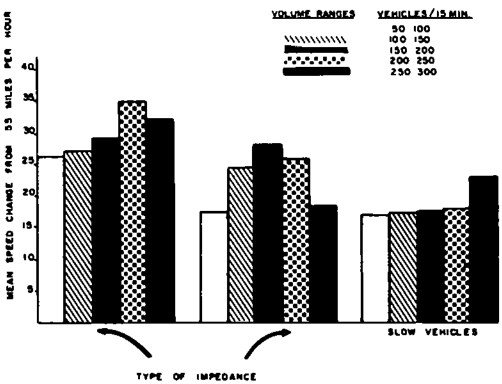

Figure 8. Speed change from 55 mph by type of impedance.

During this study these data were collected for eight types of impedances. These impedances are shown individually in Figure 5. The maximum car is shown traveling in the indicated direction at the bottom of the figure preceded by a slow vehicle impeding the maximum car. In front of the slow vehicle are shown six directional arrows indicating the six kinds of turning movements which can possibly affect the speed of

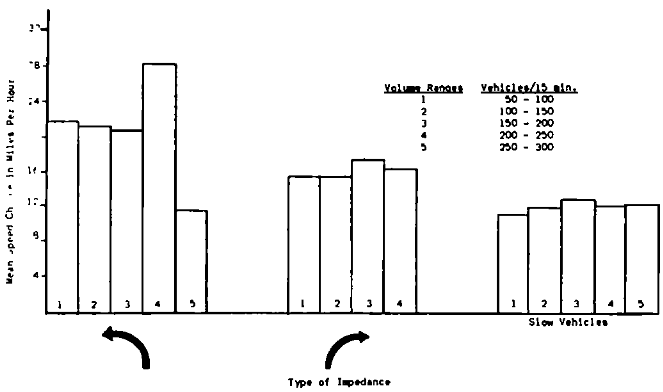

Figure 9. Speed change in miles per hour by type of impedance (for maximum car).

the maximum car. The eighth type of impedance, miscellaneous, is shown at the top of Figure 5. Any occurrence which affected the speed of the maximum car, except the first seven mentioned, was coded onto the recorder chart as miscellaneous. The three miscellaneous occurrences shown in Figure 5 are (a) a passing maneuver taking place in a direction opposed to the direction of the maximum car, (b) a parked vehicle, and

(c) a pedestrian. The legend shown in Figure 5 was used in all figures and tables of this study to facilitate the presentation of data and results.

Data collected during this study, in addition to the data collected by the speed and delay recorder, consisted of the directional roadway traffic volume in 15-min intervals and the average and maximim speeds. These speeds which were obtained by the average and maximum cars, respectively, were measured on the basis of the same 15-min intervals.

Conduct of Study

The average- and maximum-car methods were employed simultaneously during the

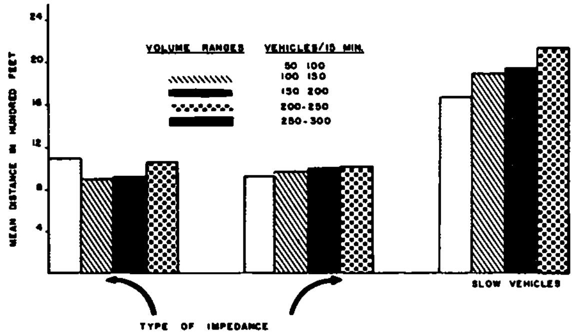

Figure 10. Distance impeded by type of impedance.

speed and delay study. Data were collected during the months of March and April 1959, at each of eight sites. Approximately 30 passes were made through each site with each of the test cars during the two most prominent peak-traffic periods. The same procedure was followed during the subsequent summer months. These procedures resulted in approximately 900 passes, or lines of data, for each of the test cars. These passes were divided equally between each direction of travel at the sites. In addition, approxi-

TABLE 2

RESULTS OF LEAST SQUARES ANALYSIS FOR SPEED VERSUS IMPEDANCE ENCOUNTERED BY TYPE

Impedance and Test Car Type	Intercept a	Slope b	90% Confidence Limits on Y' at $X = \bar{X}$	90% Confidence Limits on Y' at $X = \bar{X} \pm C$	Correlation Coefficient
Maximum-car speed versus number of turns per 30 runs	51.43	-0.30	0.80	c = 3 1.00	-0.43
Maximum-car speed versus number of slow vehicles per 30 runs	55.08	-0.37	0.73	c = 10 0.79	-0.67
Average-car speed versus number of turns per 30 runs	44.29	-0.18	-	-	+0.14
Average-car speed versus number of slow vehicles per 30 runs	34.98	0.00	-	-	0.00

mately 500 additional passes of supplemental maximum-car data were collected during the late summer months of 1959.

Analysis of Data

The data collected during the speed and delay study were treated by a least squares analysis of speed versus roadway traffic volume and speed versus frequency of occurrence of specific types of impedances. An application of confidence limits to determine the reliability of the least squares equations was also made. A detailed explanation of the technique is included in the Appendix.

Findings

Speed Versus Volume. — The results of the four least squares speed versus volume equations are given in Table 1. The equation for the maximum car in the undeveloped

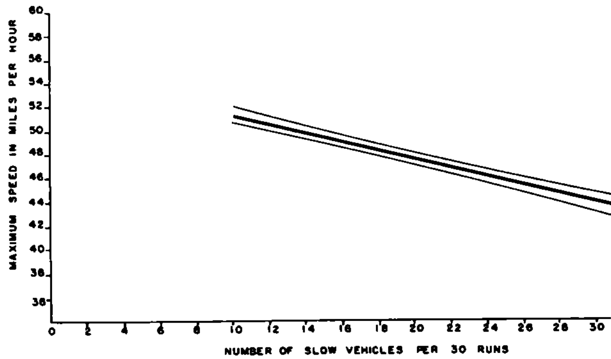

Figure 11. Effect of slow vehicles on maximum speed.

section has an intercept of 55.01 mph. This is most significant because the theoretical value for the intercept is 55 mph, the maximum-legal speed. When observing the value for each of the intercepts, it should be realized that these speeds occur only when the roadway traffic volume is equal to zero, and therefore do not represent average speeds for either test car.

It is notable that the slope is the same in each case except for the maximum car in the developed section. This might be explained in terms of the fact that the theoretical

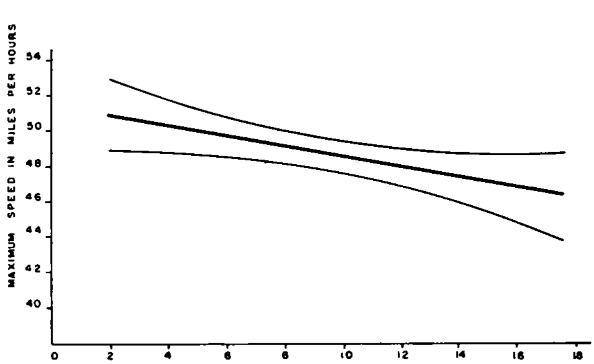

Figure 12. Effect of number of turns on maximum speed.

intercept, a, for both maximum-car equations is 55 mph. But because maximum speeds in the developed sections are lower than those in the undeveloped sections, the slope, b, must be greater for the equation representing maximum speed in the developed section.

The confidence limits given in Table 1 exhibit a great deal of variation from test car to test car and section to section. This is readily explained in terms of the residual standard deviation of Y with respect to X. Confidence limits are directly proportional to the standard deviation.

As an example, the equation for the speed of the average car in the developed section is $Y = 44.67 - 0.02X$, in which X is a measured 15-min traffic volume and Y is the resulting average speed. These equations, of course, are applicable for all undeveloped sections of any highway and for all other developed sections of highways similar to those studied in this investigation.

The correlation coefficients of Table 1 illustrate the degree of linear relationship between the Y and X variables, speed and volume. The equations are shown in Figure 6.

Analysis of Impedances. — This analysis is concerned with the maximum car only and with the delay-causing impedances encountered by the maximum car while collecting data for the speed and delay study. The maximum car encountered approximately 1,000 impedances of the eight types studied. These are shown in Figure 7. This figure indicates the distribution of these impedances both by percentage and by absolute number. It may be seen readily that more than two-thirds of the impedances encountered consisted of slow vehicles. The two other major impeders, right and left turns off the highway from the direction of the maximum car, are individually quite significant. The other five impedance types, however, are not significant, either individually or collectively. They combine to yield only 4.6 percent of the total impedances encountered. In much of the following analysis and discussion, the five minor impedance types are not considered because the quantity of data does not permit reliable analysis.

Figure 8 is a distribution in terms of impedance type, roadway traffic volume, and the average change in speed from 55 mph induced by various combinations of these two conditions. The illustrated increase in speed change with higher traffic volumes, up to a point, resulted from the increased difficulty encountered by the driver endeavoring to turn. The speed change induced by slow vehicles continues to increase with increasing volume because of the inverse relationship between speed and volume (Fig. 6). The size of the speed change induced by turns tended to decrease at the higher volume ranges. This might have been caused by the accumulation of very large slugs of traffic in which case a turning vehicle from near the front of the slug would induce a smaller speed change in the vehicles near the rear of the slug.

In Figure 9, the mean speed change induced in the maximum car is measured from the speed of the maximum car before the effect of the impedance. The magnitude of the speed changes in this instance is smaller than in Figure 8 because, in most cases, the maximum car was traveling at a speed less than 55 mph. A turning movement may slow the maximum car to a stop. Therefore, the speed changes induced by turns are larger than those induced by slow vehicles. The speed changes induced by turns tend to decrease in the high volume ranges because the maximum car is already moving at a low rate of speed.

It is significant that slow vehicles affect the speed of the maximum car over almost twice as much distance as do the turns (Fig. 10). A slow vehicle may continue through the entire length of the test section. The impedances, right turns and slow vehicles, tend to affect the maximum car over longer distances as the traffic volume increases because opposing vehicles make passing difficult for the maximum-car driver. The same tendency associated with the left turns results from the increased difficulty in turning across the opposing traffic stream.

As shown in Figure 10, there was not a great deal of difference between the mean seconds of delay caused by slow vehicles and that caused by left turns. Of greater significance, as shown in Figure 5, is the fact that six times as many slow vehicles impeded the maximum car as did left turns. The seconds of delay caused by right turns was less than that caused by left turns, yet, the two have almost equal significance because right turns occurred somewhat more frequently than left turns. The longer delay associated with high volume ranges resulted from the difficulty encountered by the maximum car trying to pass and by the impeding vehicle trying to turn. It should be noted that slow vehicles cause about the same time delay as turns, but they occur more frequently and thus are more significant.

Speed Versus Impedance. — The average speeds of the maximum and average cars were both analyzed by the least squares method in terms of number of turns encountered and number of slow vehicles encountered by the maximum car during 30 passes through the developed section. The results are given in Table 2. The correlation coefficients for the average-car analyses indicate no linear relationship in one case and practically no relationship in the other case. These coefficients might be explained in terms of the speed of the average car being determined by the average speed of the traffic stream which in turn might be the result of psychological factors rather than actual impedances. Because no relationships existed, the average-car analyses were not completed.

In the case of the maximum-car analyses, the correlation coefficients indicate a rather high degree of linear relationship. The confidence intervals are quite narrow, but only one of the intercepts given in Table 2 approaches the theoretical value of 55 mph.

It must be realized in studying the results in Table 2 that these constants do not relate a completely true picture because the analysis of speed versus slow vehicles was affected to an unknown degree by the turns encountered by the maximum car. The same is true in the reverse order concerning the analysis of speed versus the number of turns.

The equations for these analyses may be written in the same manner as previously shown for the analyses of speed versus volume.

The equations and confidence intervals for the foregoing are shown in Figures 11 and 12. The curves are shown only for the range over which the data were collected.

Conclusions

Based on the findings of this speed and delay study, the following conclusions are drawn:

1. The least squares analyses for speed versus volume indicate that the average speeds of both the maximum and average cars can be predicted, in terms of roadway traffic volume, with a high degree of confidence.
2. Slow vehicles are by far the most significant of the eight impedance types studied.
3. Left turns and right turns from the highway and in the direction of the test car are individually significant as impedances.
4. The remaining five types of impedances are insignificant, both individually and collectively.
5. There is a definite, inverse relationship between the speed of the maximum car and the number of slow vehicles encountered by the maximum car.
6. There is a definite, inverse relationship between the speed of the maximum car and the number of turns encountered by the maximum car.

FIELD TESTING OF DERIVED RELATIONSHIPS

Various mathematical relationships were derived and presented in the preceding section. The purpose in developing these relationships was to permit prediction of the speed and turning movement characteristics of any section of highway. Any section for which predictions are to be made would necessarily be required to have physical characteristics similar to those on which the relationships are based.

Unless the derived relationships are field tested in some practical manner, their validity may be subject to question. The purpose of this section is to relate the manner in which the relationships were tested and the results of the test.

Summary of Relationships and Method of Testing

For predicting average speeds in the sections indicated, the following equations were presented previously:

Developed section: $Y = 44.67 - 0.02X$
Undeveloped section: $Y = 47.67 - 0.02X$.

Similar equations for the maximum-car speeds are as follows:

Developed section: $Y = 53.56 - 0.04X$
Undeveloped section: $Y = 55.01 - 0.02X$.

The X and Y variables in these four equations are, respectively, average speed and roadway traffic volume in 15-min increments.

The equations for predicting turning movements for the indicated business types were not previously discussed in this report, but are as follows:

Service stations: $Y \,.\, 0.77X_1 + 0.01X_2$
Restaurants: $Y = 1.15X_1 + 0.01X_2$
Cafes and drive-in cafes: $Y = 2.13X_1 + 0.01X_2$
Grocery and grocery-service station combinations: $Y = 1.04X_1 + 0.01X_2$
Supermarkets and large open-air markets: $Y = 2.42X_1 + 0.02X_2$
Furniture and restaurant equipment stores: $Y = 0.04X_1 + 0.01X_2$
Miscellaneous: $Y = 0.66X_1 + 0.01X_2$.

The Y, X_1, and X_2 variables in these seven equations are, respectively, turning movements, dollar income, and roadway traffic volume (all for a 10-hr period between 7:00 AM and 5:00 PM).

Because the turning-movement equations may be used only to predict the total number of turns for the entire 10-hr period, the frequency distributions in Figures 7 through 10 are used to predict the number of turns for specific periods, such as the

TABLE 3

PREDICTIONS FOR THE WASHINGTON SITE

Time	Section Type	Maximum Speed (mph)		Average Speed (mph)	
		Predicted	Actual	Predicted	Actual
7:00 - 8:00 AM	Developed	49.2	49.5	-	-
	Undeveloped	53.0	53.6	-	-
7:10 - 9:10 AM	Developed	-	-	43.0	40.4
	Undeveloped	-	-	46.1	45.0
12:00 - 1:00 PM	Developed	49.0	46.5	-	-
	Undeveloped	52.9	51.8	-	-
3:10 - 4:10 PM	Developed	47.9	47.7	-	-
	Undeveloped	52.1	49.7	-	-
3:10 - 5:10 PM	Developed	-	-	42.2	40.6
	Undeveloped	-	-	45.3	46.4

Business	Turning Movement	
	Predicted	Actual
Esso service station	87 ± 26	97
Etna gas	117 ± 26	160
Sinclair service station	98 ± 26	115
Ayers Restaurant	224 ± 71	181
Franks Restaurant	60 ± 71	115
Mobley's Tire Company	70 ± 30	67
Clearview Television	58 ± 14	27

lunch hour period. For example, if the frequency distribution for a particular type of business indicates that the period between 11:30 AM and 12:30 PM generates 19.5 percent of the total number of turns occurring during the 10-hr period, then 19.5 percent of the value arrived at by the equation represents the number of turns to be expected between 11:30 AM and 12:30 PM.

An important phase of this test was the selection of a typical test site. It was desirable to locate a test site which included all seven of the business types studied previously. Further, it was necessary that the site not contain any major intersections or traffic signals. In short, the site needed to be in general conformity with the test site definition.

After investigating a number of possible locations, a site on US 17, south of Washington, North Carolina, was chosen. The developed section of this site has three businesses of the service station type, two businesses of the restaurant type, one business of the furniture and restaurant equipment type, and one business of the miscellaneous type. The furniture and restaurant equipment type of business is a television sales and service store. It was classified in this manner because of the non-impulse-type buyers which patronize the business. The miscellaneous-type business is an automobile-tire recapping and vulcanizing business. The operations of both of the latter-type businesses were rather limited in scope. Each of these businesses employed only two persons. It is unfortunate that the site did not contain businesses of each of the seven types. It is most difficult, however, to find a site having all the business types.

The data necessary for computing all the desired predictions consisted only of: (a) the total roadway traffic volume between the hours of 7:00 AM and 5:00 PM, and

TABLE 4

PREDICTED AND ACTUAL NUMBER OF TURNS FOR SHORT TIME PERIODS

Business	Time	Predicted No. of Turns for 10 Hours	Percent	Predicted	Actual
Esso station	8:30 - 8:45	87	3.0	3	0
	12:00 - 1:00	87	14.0	12	10
	3:00 - 4:00	87	6.0	5	3
Sinclair station	8:30 - 8:45	117	3.0	4	0
	12:00 - 1:00	117	14.0	16	14
	3:30 - 4:00	117	6.0	7	6
Etna gas	8:30 - 8:45	98	3.0	3	7
	12:00 - 1:00	98	14.0	14	9
	3:00 - 4:00	98	6.0	6	14
Ayer's Restaurant	10:30 - 10:45	224	3.0	7	1
	12:00 - 1:00	224	18.4	41	41
	3:00 - 3:30	224	5.8	13	4
Frank's Restaurant	10:30 - 10:45	60	3.0	2	3
	12:00 - 1:00	60	18.4	11	11
	3:00 - 3:30	60	5.8	3	5
Mobley's Tire Company	10:00 - 10:30	70	10.0	7	6
	12:00 - 1:00	70	11.5	8	9
	4:45 - 5:00	70	5.6	4	1
Clearview Television	10:00 - 10:30	197	10.0	20	1
	12:00 - 1:00	197	11.5	23	1
	4:45 - 5:00	197	5.6	11	0

(b) the total dollar income for each business for the same period of time. Because the predictions were being made for a particular day, it was necessary to determine the actual values on that day. Therefore, the following data were all collected on the same day:

1. Total roadway traffic volume, in 15-min intervals, between the hours of 7:00 AM and 5:00 PM.
2. Total dollar income for each business in the developed section for the same time period.
3. Average speeds by the average- and maximum-car methods for both the developed and undeveloped sections during the peak traffic periods of the day.

All the tests were conducted and the data collected in a manner similar to that employed in previous studies of this investigation.

Findings

The effects tabulated in this section are presented in terms of the Washington test site versus the average of results for all the other test sites studied during this investigation.

The predicted and actual values of speeds and turning movements are given in Table 3. In some cases the difference between predicted and actual speed was greater than the confidence limit as computed and tabulated in the previous section. This resulted from the fact that the actual speeds given in Table 3 are in each case based on the average speed of 30 passes through the site. The roadway traffic volume during the required 1- to 2-hr periods varied from one 15-min period to the next. The predicted speeds of Table 3, however, were computed from the speed versus volume equations which were based on average speeds and roadway traffic volumes for 15-min periods only. The volume variation during this short period was rather small. There-

TABLE 5

PERCENTAGE OCCURRENCE BY TYPE OF IMPEDANCE

Type Impedance	Washington Data (%)	Average Data[a] (%)
Slow vehicles	77.7	68.9
	5.6	14.9
	13.0	11.6
	0.0	1.4
Miscellaneous	0.0	1.3
	3.7	1.1
	0.0	0.4
	0 0	0 4
 Maximum car	100 0	100.0

[a]Source Figure 7 of text.

TABLE 6

SPEED AND DELAY DATA AT WASHINGTON VERSUS THE AVERAGE FOR ALL SITES

Characteristic	Ranges of Volumes per 15 Min					Slow Vehicles	
		Washington	Average	Washington	Average	Washington	Average
Mean speed change	50 - 100	-	21.5[a]	-	15.0[a]	10.9	11.0[a]
	100 - 150	-	21.0	39.0	15.0	11.4	12.0
	150 - 200	12.0	20.5	14.0	17.0	12.5	13.5
Mean speed change from 55 mph	50 - 100	-	26.5	-	17.5	20.7	18.0
	100 - 150	35.5	27.0	39 0	24.0	22.6	18.3
	150 - 200	34.0	29.0	30.0	28.0	19.5	19.0
Mean seconds of delay	50 - 100	-	7.5	-	3.8	6.6	7.0
	100 - 150	4.5	7.0	10.0	5.8	6.4	8.5
	150 - 200	10.5	7.0	15.0	6.0	6.0	9.0
Mean length of impedance	50 - 100	-	11.0	-	8.2	16.3	17.0
	100 - 150	15.0	8.5	23.8	9.0	16.5	19.0
	150 - 200	18.6	9.0	23.8	9.8	17.1	19.5

[a]Source: Figures 7-10 of text.

fore, the actual speed cannot be expected to lie exactly within the confidence interval of the predicted speed. The difference between actual and predicted speeds ranges between 0.2 and 2.6 mph. Considering these points, this is deemed adequate.

The actual and predicted number of turns for each of the seven businesses in the Washington test site are given in the lower portion of Table 3. The confidence intervals were computed for the average Y, number of turns, not the predicted Y. Yet, in only two cases did the confidence interval not include the actual value. The Clearview Television Store, a furniture and restaurant equipment type of business, had an actual value outside the confidence interval. This business deals with non-impulse buyers of expensive items resulting in a high degree of dollar income variation from day to day. This fact makes predictions difficult. The error in prediction for the Etna Gas Station was possibly a result of random variation outside the 90 percent probability level on which the confidence interval is based.

The predicted and actual number of turns for various time periods during the 10-hr test period are given in Table 4. The values in the percent column were obtained from frequency distributions not shown. Except for the noon-hour time period, the periods in the time column were chosen at random. The values tabulated give some indication of the validity and utility of the frequency distributions. In the majority of instances, the difference between actual and predicted values was quite small in magnitude. However, the predictions having a large error indicate the difficulty arising from the large variation of the number of turns, during a short-time (15-min) period as opposed to a smaller variation when considering a time period of perhaps an hour.

A tabulation of the eight types of impedances studied, and the percentage each contributes to the total number of impedances is presented in Table 5. In general, the Washington data follow the trend established by the averages of all the sites. This is readily understood when considered in light of the fact that the Washington data represent only a small fraction of the sample size on which the average values are based.

Table 6 gives the Washington speed and delay data and the average of data from all the sites studied. Only the three major impedance types are considered along with their four characteristics as measured by the speed and delay recorder and the maximum car. Again the Washington data represent a very small sample size as seen from the blank spaces in the table in which cases no impedances occurred under the indicated conditions. It is evident, however, that the Washington data follow the trend of the average data.

Conclusions

Based on the data collected during this investigation, the statistical analyses conducted, and the field testing of the results of the analyses, the following conclusions are drawn:

1. The equations of average and maximum speeds for developed and undeveloped sections are adequate for predicting these speeds.
2. The seven turning-movement equations can be used to predict within a reasonable degree of accuracy the number of turns to be expected at any of the seven types of businesses.
3. Using the time-frequency distributions of turns for each business type, the number of turns can be estimated for time periods as short as one hour.
4. The percentage distribution of occurrence of the eight types of impedances and the speed and delay characteristics for each type are essentially the same for any developed section of highway similar to the sections studied during this investigation.

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Appendix

THE LEAST SQUARES METHOD OF ANALYSIS

For the speed versus volume relationships, four equations were fitted for the four combinations of average speed in developed and undeveloped sections and maximum speeds in the same sections. This least squares analysis is based on the primary assumption that the speeds of the average and maximum cars vary linearly with the independent variable, roadway-traffic volume. Based on this assumption, the following model equation may be written:

$$Y = A + BX$$

in which

Y = the mean speed of the test car based on a 15-min increment of roadway traffic volume,
X = the roadway traffic volume during a 15-min interval,
A = the intercept or the value of Y when X is equal to zero, and
B = a constant which is the slope of the speed-volume curve.

The least squares analysis computes estimates of the true parameters, A and B, such that the sum of the squares of the vertical distances from the line ($Y = A + BX$) will be a minimum. The estimates of A and B are designated by a and b, respectively. By completion of the least squares analysis the following equations may be written:

$$Y' = a + bX'$$

in which

$$b = \frac{\Sigma (X-\bar{X})\ (Y-\bar{Y})}{\Sigma (X-\bar{X})^2}$$

$$a = \bar{Y} - b\bar{X}$$

$$R = \sqrt{\frac{\Sigma (X-\bar{X})\ (Y-Y)}{\Sigma (X-\bar{X})^2\ \ \Sigma (Y-\bar{Y})^2}}$$

in which

Y' = the resulting predicted value of speed,
X = the observed 15-min roadway traffic volume,
Y = the average value for speed measured during the 15 min for which roadway traffic volume was measured,
$\bar{Y} = \frac{\Sigma Y}{N}$,
R = correlation coefficient,
$\bar{X} = \frac{\Sigma X}{N}$, and
N = the number of observations or 15-min intervals for which X and Y were measured.

Two other least squares analyses were conducted on the speed and delay data. In both analyses, the model equation is identical with that of the speed versus volume analysis previously discussed. In the first analysis, the dependent and independent variables have the following meaning: Y = the average speed of the maximum car, and X = the number of slow vehicles encountered by the maximum car during 30 passes through the developed section. In the second of the two analyses, the dependent and independent variables have the following meaning: Y = the average speed of the maximum car, and X = the number of delay-causing turns encountered by the maximum car during 30 passes through the developed section.

The procedure and equations used in the speed versus volume analysis apply equally well for these two analyses. It is necessary to adjust only the meanings of the terms in those equations to fit the meanings given these dependent and independent variables.

Development and Use of Maximum-Car Technique for Measuring Travel Time

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● DURING recent years many traffic engineers have been concerned with a need for more accurate and less costly and time-consuming methods of measuring travel time and delay along sections of roadway. Various techniques — the license-plate-matching method, the average-car method, and the floating-car method — have been employed with varying degrees of success, depending primarily on local conditions and available funds. More modern techniques using photography have also been tested.

In an attempt to attain a more satisfactory technique of measuring travel time and delay, the "maximum-car" method was developed within the Highway Research Program conducted by the Civil Engineering Department at North Carolina State College. The method was actually an outgrowth of a more comprehensive research effort investigating the effects of commercial roadside development on traffic flow conducted over a two-year period in cooperation with the North Carolina State Highway Commission and the U.S. Bureau of Public Roads.

This report traces the development of the maximum-car technique, explains the theory on which it is based, and offers suggestions regarding possible applications in the analysis of traffic problems.

Essentially, the method exploits a "test vehicle" for measuring travel time through a given section of roadway. Physical occurrences within and adjacent to the traffic stream affect speed which, in turn, can be related to some standard datum and measured. It is believed that this approach, when properly understood and used, can produce far more significant travel time and delay information than is now possible under current practices.

One commonly employed method of calculating travel time is the "average-car" approach. This car is driven at speeds which, in the opinion of the driver, are representative of the average speed of all the traffic in the stream. Excellent results have been obtained from the use of this method. In research at North Carolina State College, the average-car method was compared with the "license-plate-matching" method, and the average car results were found to be a valid measure of the average speed. The license-plate-matching method, which is a positive measure of the speed of a sample of vehicles from the traffic stream, was used as a standard to check the accuracy of the results of the average-car method. Ninety-five out of 100 times the average car gave the "true" average speed within ± 2 mph.

Another test car, employed in traffic analysis, is the "floating car." The driver of this car is instructed to pass as many vehicles as pass the test vehicle. This method is used, most successfully, in low volumes and over long distances, although it has several obvious disadvantages.

The acceptance and use of these two methods in determining average speeds of traffic flow substantiate the validity of the test-car approach in traffic analysis. As will be explained later, these presently used methods have certain disadvantages. To overcome these disadvantages, the maximum-car technique was developed as a test-vehicle approach to the measurement of traffic-stream characteristics.

A search of the literature in this field indicated that little progress has been achieved using a maximum-car approach to travel-time measurement. Donald S. Berry and Forest H. Green, in their investigations of techniques for measuring over-all speeds in urban areas, dealt briefly with a method called the "faster test run." The driver of this test car maintained maximum speed consistent with safety and existing traffic regulations. Berry and Green, in their work, were primarily interested in average speeds and travel time for vehicles through specific test sites. They concluded that the travel time and speed of this faster car were not representative of the average vehicle in the traffic stream. This conclusion was also substantiated by preliminary studies during research at North Carolina State College.

DEFINITION

For the purposes of this study, the maximum car was defined as a test car driven, consistent with safety, at the posted speed limit unless impeded by actual traffic conditions. For valid comparative results it was essential that the driver of this car understand the definition and attempt to drive with the same attitude in all testing. A safe level of operation was maintained at all times by observing minimum safe-following distances, minimum passing distances, and reasonable acceleration and deceleration. The driver's attitude might be described as that of a businessman on a 300-mi trip, during which time he observes minimum safe-driving practices and desires to drive at the speed limit when not impeded. This qualification of the driver's attitude makes it imperative that the same driver perform all testing for a particular analysis.

In an effort to determine the effects of auto performance characteristics on speeds, automobiles of different horsepower and age were compared using the same driver. The variation resulting from the use of different automobiles was small; but, to eliminate as much error as possible, the same car should be used throughout a particular analysis.

In addition to the requirements for a standard car and driver, the test vehicle must have no distinctive markings such as seals, permanent license tags, or antennas that might indicate a police vehicle and thus arouse the suspicion of a motorist along the route of travel.

The maximum car, as defined with the stated restrictions, gives a constant measurable consequence that is standard in any given set of physical conditions. The speed reduction or delay that is recorded by the maximum car is a result of physical factors caused by roadway characteristics and the traffic stream. The psychological factors are minimized and standardized by adherence to the definition. The only attempt to measure phychological effects on the traffic stream was to subtract the physical effect, measured by the maximum car, from the total effect, measured by the average car, to obtain the aggregate psychological effect.

THEORY

The basic theory of the maximum-car technique is simply that the method is a measuring device with which it is possible to gage the effects of physical factors on traffic operations. As stated in the definition, the maximum car is operated at the posted speed limit unless impeded by some physical occurrence, such as a slow-moving vehicle or a turning movement by a preceding vehicle. The speed limit acts as a datum from which performance is measured. If a left turn takes place in front of the maximum car, under a given set of conditions, the effect is the same each time these conditions are encountered. Hence, the results will be standard from pass to pass as well as from site to site. In comparison, the only datum remotely possible in the average-car method is the average speed. At best this is a sensitive variable. Under a standard set of conditions and with a mere variance of the population of drivers within the traffic stream, the effect recorded by the average car will change.

The absolute value of the recorded maximum speed is of little value because it represents the driving characteristics of only one man. Hence, this maximum-car method can be used only as a measuring mechanism in the analysis of traffic behavior.

One other important aspect of the theory is the number of runs required to constitute a reliable test. This information should be determined by pilot studies. From data obtained in such studies, the number of runs required to constitute a test can be determined. The findings of this study were based on 2,000 passes through 10 test sites. The total period of time to complete a test is governed by driver fatigue and changing characteristics of the traffic.

DATA COLLECTION EQUIPMENT

A 1959 blue Ford with white sidewall tires, a nongovernmental North Carolina liense plate, and automatic transmission was used as the maximum car in this research. As far as could be determined, this car did not arouse undue suspicion during operation in the traffic stream. The automatic transmission provided smooth acceleration for the speed curve on the recorder chart.

The maximum car was equipped with the Model M, Electro-Matic Traffic Speed and Delay Recorder (Fig. 1). This instrument is manufactured on special order by the Automatic Signal Division of Eastern Industries in Norwalk, Connecticut.

The speed and delay recorder transcribes a continuous chart (Fig. 2) of the speed of the vehicle in which it is installed. This speed can be plotted against either time or distance. The time is recorded in 6-sec and 1-min increments, while the distance traveled is plotted in 200- or 400-ft stations. Six coding pens permit the recording of

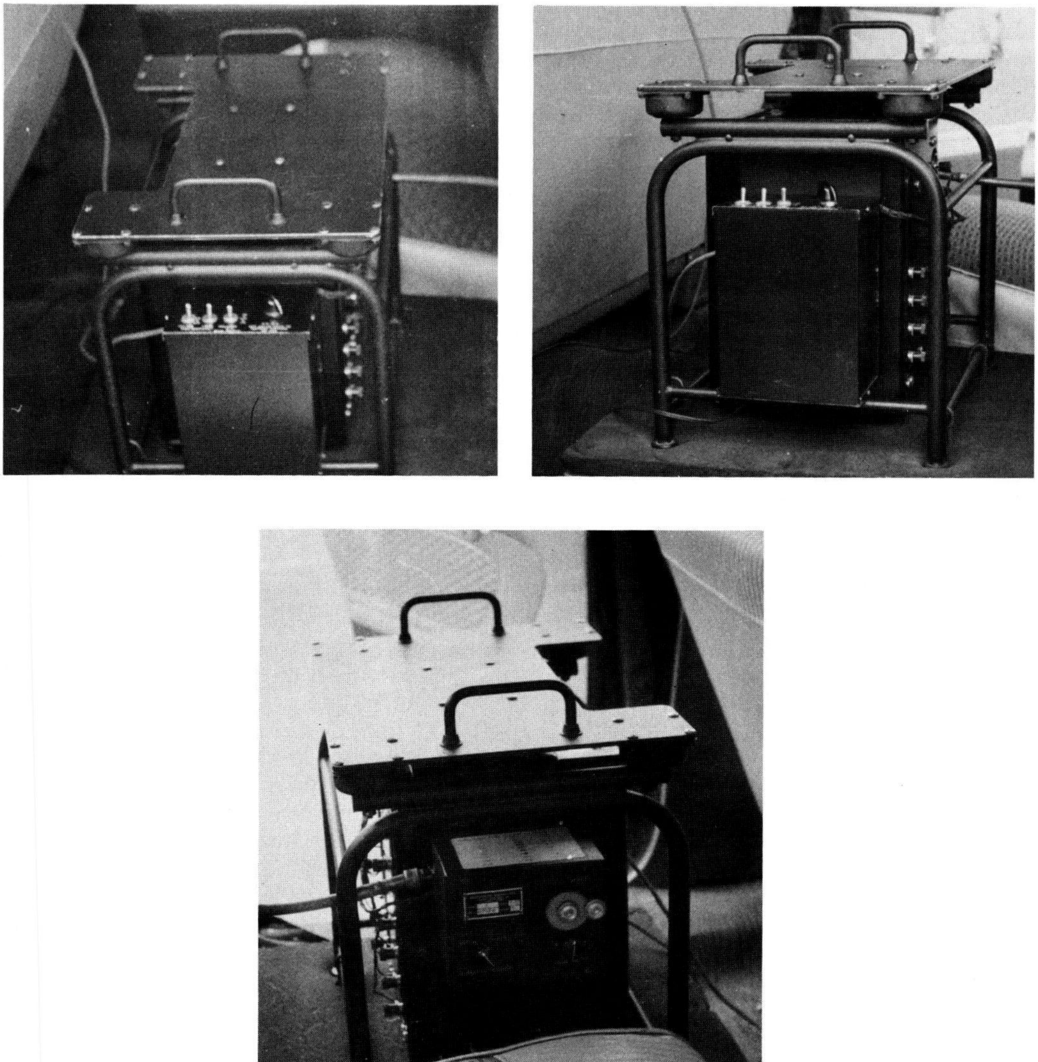


Figure 1. Installation of speed and delay recorder.

events encountered during testing. An interchangeable set of drive gears permit varied scales of speed versus time or distance. The recorder was mounted on a platform built into the rear seat of the test vehicle. The speedometer cable of the test vehicle was extended to the recorder and operated both the recording tachometer and the dash-mounted speedometer. Electric power was received from a cigar-lighter plug mounted on the rear floor tunnel. The code buttons were operated by an observer seated in the right-front seat (Fig. 3) who had an unobstructed view of the road ahead or of the chart being produced by the recorder. This observer also used a stop watch to measure travel time through the site. Traffic volumes were obtained from another observer along the site.

FACTORS AFFECTING THE MAXIMUM CAR

Physical and Psychological

To analyze the effects of commercial development on traffic operations, restrictions to traffic were divided into two basic categories—physical and psychological. The physical restrictions are the tangible elements that can be measured quantitatively and examined. The psychological restrictions are, for the most part, the mental attitudes

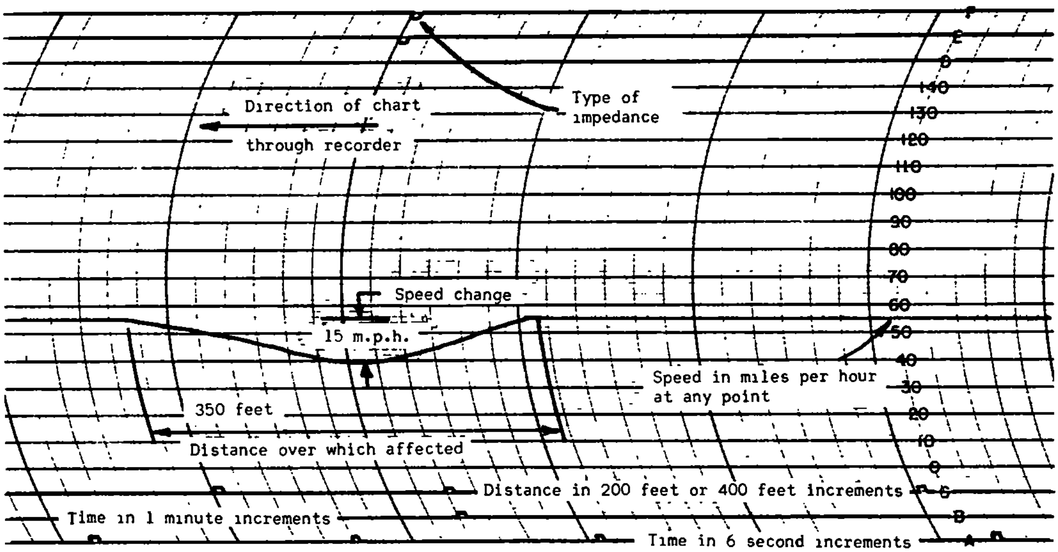


Figure 2. Speed and delay recorder chart with hypothetical speed curve.

of the driver. For example, the driver may not be accustomed to the speed limit or may feel unsafe at that rate.

Use of the maximum car, as defined, eliminates the psychological factors and records only the effects of the physical restrictions. An exception to this condition develops when the maximum car is operated in extremely high volumes. Under such conditions, the maximum car is unable to pass; therefore, the maximum car is forced to record psychological restrictions being applied to the traffic stream.

The following relationships are useful in the comparison of average and maximum speeds:

1. Any average car could become a maximum car if its driver desires to overcome the psychological restrictions. Therefore, the average speed could equal the maximum speed.
2. The maximum speed without the physical or other restrictions is equal to the speed limit.

3. The operating speed is equal to the speed limit minus the restrictions.

All the data collected by the maximum-car method were analyzed to determine the effect of each physical restriction on the speed of the maximum car. These results are given herein and are representative of the effects that would be found in any maximum-car operation.

Speed and Volume

An analysis of the maximum-car speeds in commercially developed and undeveloped

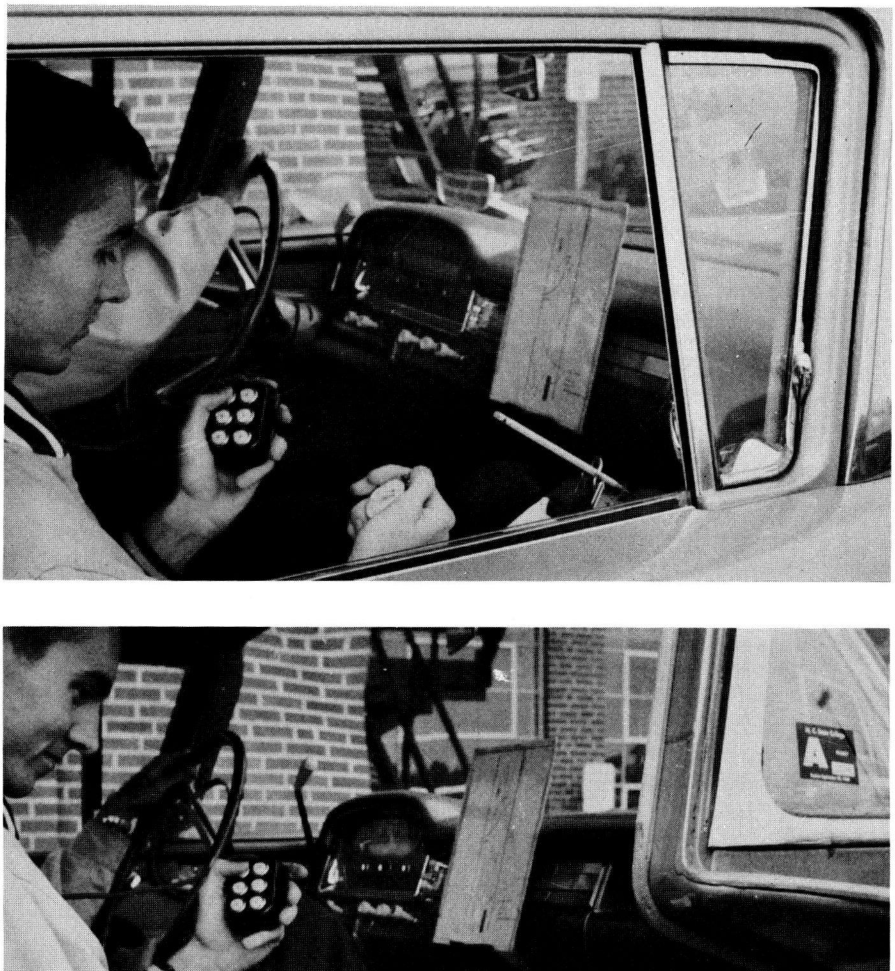


Figure 3. The maximum-car observer.

sections of roadway versus traffic volume was accomplished to explore the effect of traffic volume on the speed of the maximum car. A least-squares-regression fit was made for the speed and volume data, and the equations of the curves were computed for the developed and undeveloped sections. A linear model was used that had the form:

$$y = a - bx$$

in which

y = speed of the maximum car per 15 min,
a = slope intercept,
b = constant, and
x = traffic volume in 15-min intervals.

The curve for the developed section is based on 88 observations in 9 test sites and is shown in Figure 4. The equation of this curve is

$$y = 55.01 - 0.02x.$$

The curve for the developed section is based on 191 observations in 9 test sites and is also shown in Figure 4. The equation of the curve is

$$y = 53.56 - 0.04x.$$

It is interesting to note that an equal volume change in both sections produces a greater speed change in the developed section. That is, the slope of the curve for the developed section is equal to -0.04, whereas the slope for the undeveloped section is equal to -0.02. The negative slope of each indicates that increasing volumes decrease speed. The differing rate of effect, shown by the comparison of the two slopes, indicates that something other than volume affects the speed of the maximum car in commercially developed sections of roadway.

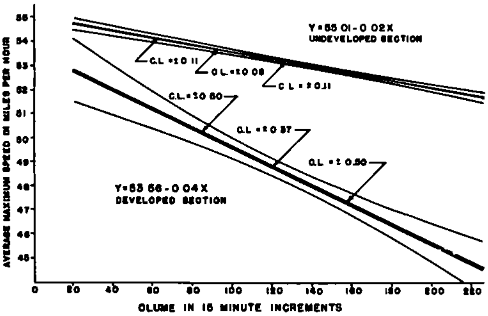


Figure 4. Speed vs volume curves for developed and undeveloped flow.

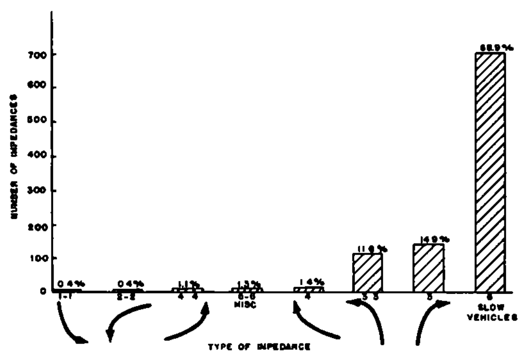


Figure 5. Type of impedance vs number of impedances encountered.

Very narrow confidence limits were obtained for these curves. The 95 percent confidence interval is shown for the curves in Figure 4. This high degree of confidence was possible because of the low variance and a large number of observations. Within the volume ranges used in this analysis, the average maximum speed in either section can be predicted with a high degree of accuracy.

Impedance

The data from the speed and delay recorder concerning the effects of impedance on the maximum car were analyzed in several ways. The most significant summary of this analysis is presented in Figure 5. The following conclusions can be drawn from this analysis:

1. Slow-moving vehicles are by far the largest cause of impedance (69 percent). The only other significant impedances are the right turn (14.9 percent) and left turn (11.6 percent) off the road in front of the impeded vehicle. Figure 5 is a graphical representation of these factors based on the impedance code found in Figure 6.
2. The effect of impedance tends to increase as volume increases with one notable exception. This exception can be explained by the fact that high volumes reduce speeds; therefore, the magnitude of the speed change from the previous speed is not as large.
3. Left turns cause more interference in terms of seconds of delay and speed change than do right turns.
4. Left turns and right turns off the highway affect the traffic over a shorter distance than do slow-moving vehicles.
5. Left turns cause the largest speed change.

These conclusions are based on 8 hr of running time per site for nine sites, or a total of 72 hr of testing.

Impedance and Speed

To investigate the relationship between impedance and speed, plots were made of speed versus number of slow-moving vehicles encountered and speed versus number of turning movements encountered. These plots indicated the possibility of a linear relationship between these variables. A least-squares-regression fit was made for these two relationships using the linear model:

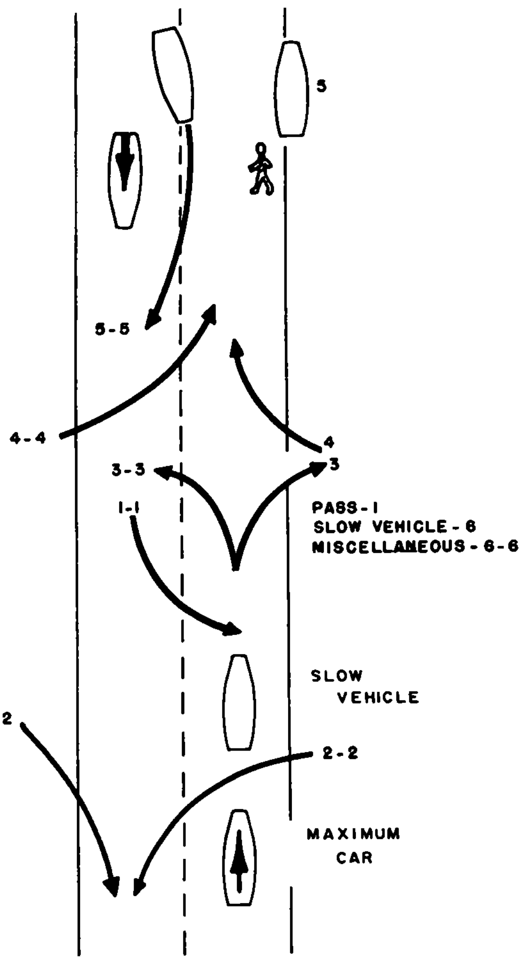


Figure 6. Types of impedances studied.

$$y = a - bx$$

in which

y = average maximum-car speed per 30 passes,
a = intercept,
b = slope, and
x = total number of turning movements encountered per 30 passes, or the total number of slow-moving vehicles encountered per 30 passes for the second graph.

For each of these curves the correlation coefficient, r, was computed. This coefficient acts as a measure of the linear relationship of the two variables.

The average maximum-car speed versus the number of slow-moving vehicles encountered per 30 passes curve is shown in Figure 7, for which the equation is $y = 55.08 - 0.366x$, in which the computed r is equal to -0.67. The 95 percent confiinterval is also shown in Figure 7.

The average maximum-car speed versus number of turning movements encountered per 30 passes curve is shown in Figure 8, and the equation is $y = 51.43 - 0.297x$, in which the computed r is equal to -0.43. The 95 percent confidence level is also plotted for the curve in Figure 8.

A comparison of the slopes of these two curves indicates that speed is more sensitive to changes in the number of slow-moving vehicles encountered than to the number of turns encountered. Also, the association between the average maximum-car speed and the number of slow-moving vehicles is more nearly perfect than in the speed versus turning movements relationship. This fact is seen by the comparison of correlation coefficients.

USE OF THE MAXIMUM-CAR METHOD IN TRAFFIC RESEARCH

Application of the maximum-car method would appear to have great potential in traffic engineering research. Four specific examples are cited in the following discussion to explain the possibilities of the method.

Delay Studies

The maximum-car method was used in commercial roadside development research to obtain a complete analysis of physical impedance on the traffic stream. The following information was received by operating the maximum car through developed sections of roadway:

1. The type of impedance encountered.
2. The magnitude of the speed change for each impedance, both from the speed limit and from the speed immediately preceding the impedance.
3. The distance over which the speed is affected by each impedance.
4. The seconds of delay caused by each impedance.
5. The location of each turning-movement impedance.
6. Speed of the maximum car.
7. Roadway volume.

The impedance data were obtained from the charts produced by the speed and delay recorder. With this information it was possible to make a detailed analysis of each type of impedance and to show its relative effect on the traffic stream.

This method could be applied to urban-delay studies as an effective measuring device.

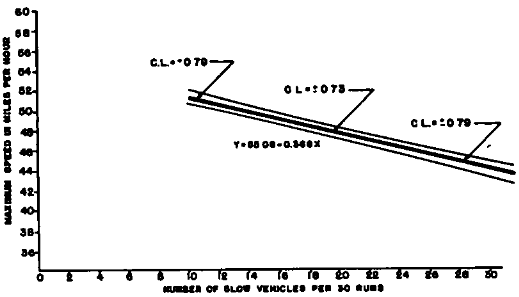


Figure 7. Effect of slow vehicles on maximum speed.

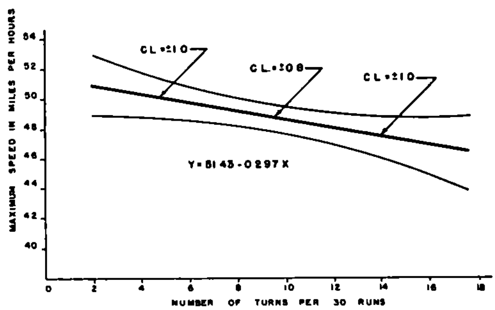


Figure 8. Effect of number of turns on maximum speed.

Certain modifications would be required to adjust for the effects of traffic signals and reduced speed limits. If the average-car data were employed as a supplement to the maximum-car data, it would be necessary to investigate the effect of the lower speed limit restrictions on average speed. The case might arise where the average speed might be above the speed limit. Such a condition would result in higher average-car speed than maximum-car speed. This negative differential might cause complications in the data analysis and should be investigated before using the test results.

Before-and-After Studies

The maximum-car method has proved itself most useful in before-and-after studies. In the spring of 1959, this method was used in a study performed on US 70 east of

Asheville, N.C. Prior to extensive revisions in the traffic control devices and pavement markings, an operational study was conducted on a 12-mi section of this highway.

After completion of the revisions and after time had elapsed for driver reorientation, an "after study" was conducted in an attempt to measure any changes in traffic operations. In this particular instance, passing opportunities and turning movements were recorded in detail. The results of this study were used to evaluate the feasibility of the continued use of these changes and their possible use in other sections of the state.

Route Selection Studies

The maximum-car method of travel-time measurement might be employed in route selection studies for both urban and rural conditions. The method would employ the theory that the maximum-car operation is a standard datum from which to measure varying effects caused by differing physical conditions. In addition to these physical effects recorded on the speed and delay recorder chart, valuable user-cost and user-convenience data also could be collected.

The resulting field data could be analyzed to determine the detrimental effects of the physical make-up of one route as compared with an alternate. With regard to its user benefits and physical characteristics measured by the maximum car, this factual comparison would result in an intelligent selection of the best route.

CONCLUSIONS

Probably the outstanding single feature of this maximum-car method in traffic analysis is its versatility. As previously discussed, the method can be adopted to a wide variety of investigations which are essential to the decision-making process of the traffic engineer. Without this method of travel-time measurement and analysis, an adequate investigation of the effects of intensive commercial roadside development would have been a more difficult task. Its use in before-and-after and congestion studies has been explored and found most useful. The adaptation of the maximum-car method to route selection and roadway geometry studies may become effective uses of this procedure. The conclusions drawn from the data collected by this method are based on physically measured facts and not mere hypothesis. It is hoped that this endeavor has laid a foundation on which to base a great variety of new and improved studies.

It is appropriate to re-emphasize the importance of the fact that the success of the maximum-car method is dependent on the mental attitude of the test-car operator. Care must be taken to insure that the operator of the maximum car does not traverse a particular site to the extent that he become over-familiar with the physical conditions of the roadway. Driver fatigue is another important consideration from the standpoint of both safety and driver consistency. The maximum-car observer must become familiar with the estabilished recording procedure, and it is desirable that one observer do all the recording in a study. Once the data is collected, a specific procedure must be established and adhered to in the reduction of the field data to an analytical form. Consistency is imperative in any measuring process, and the maximum-car method is essentially a measuring device.

The lack of basic background data is one of the major criticisms of present-day solutions to traffic problems. It is anticipated that the maximum-car method will be helpful in the collection of this much needed basic information; thereby guiding the traffic engineer to the proper solutions of particular problems.

An Analysis of Urban Travel Times and Traffic Volume Characteristics

C. DWIGHT HIXON, Traffic Plans Engineer, Nashville Metropolitan Area Transportation Study

● AS PART of the Nashville Metropolitan Area Transportation Study, several studies were carried out which were somewhat of a departure from normal urban practices. Travel time has been generally used for before-and-after studies at specific locations. Here the study included the entire urban area and is used as a basis to compare the "level of traffic service" of the various traffic carrying arteries.

In many cities there was not an abundance of information concerning traffic volumes. In others, Nashville included, there was considerable information on file. The intent of the volume study was to get the available information into use and establish a volume counting program to obtain additional information to round out the over-all picture.

This paper is intended to show some of the analyses and results of these two studies; not so much for the sake of specific values found in Nashville, but to discuss some of the general findings of the studies.

To date the Travel Time Study covers the major arterial system. Data were collected during the morning peak hour and off-peak periods for automobiles. The evening peak hour study also included transit.

The data collected covers approximately 150 mi of major arterial streets. All but the outlying control sections of this system have had around 20 travel time passes at various times of the day. With exceptions, the morning peak hour passes were inbound and the evening peak hour passes, including transit, were outbound. The exceptions were one-way streets, circumferential streets and radial streets which appeared to offer a poorer level of service in the off-peak direction of flow. The off-peak period passes were made in both directions on all streets.

During the data collection phase, several analyses were accomplished to determine sample adequacy. The National Committee on Urban Transportation study program prescribes an adequacy check for sample size for travel time studies. It is based on average deviation and assumes normality in the sample.

The National Committee on Urban Transportation check for adequacy states that the sum of the deviations from the calculated mean of six travel times shall be equal to or less than the mean for the same to be considered adequate.

For this discussion let:

T_1, T_2, etc. = travel time on any pass;
T_m = mean travel time;
$D_1 = T_1 - T_m$; $D_2 = T_2 - T_m$ etc.;
s = standard deviation;
S_m = standard error of the mean; and
N = number of passes on which the check is based, in this case 6.

Therefore:

$$D_1 + D_2 + D_3 --- + D_6 = T_m \tag{1}$$

The average deviation = $\frac{T_m}{6}$. In other words if Eq. 1 is true, then the average deviation in time on a given pass is equal to one-sixth the mean.

By definition average deviation equals 0.7979 standard deviations (0.7979s):

$$0.7979 = \frac{T_m}{6} \text{ or } s = 0.209\ T_m \tag{2}$$

Assuming that each of the six passes is an estimate of the true mean it is seen that the standard deviation of these sample means about the true mean (S_m) is:

$$S_m = \frac{s}{\sqrt{N}} = \frac{0.209\,T_m}{\sqrt{6}} = \frac{0.209T_m}{2.449} = 0.085\,T_m \qquad (3)$$

Therefore, six passes meeting the conditions of this check will yield accuracy within 8.5 percent in 67 percent of the cases.

The normality assumption was checked by making 51 passes in each direction over three control sections on one street. The street ran from downtown to the city limits and had characteristics comparable to other major streets. The chi square "goodness of fit" test was applied to the results. For the purpose of this test, there were six control sections (three control sections with passes in each direction). The results are given in Table 1 and, based on these results, it was assumed that normality could not be disproved.

The National Committee procedure states that on control sections where the travel times do not meet the conditions of the check for six passes, four additional passes shall be made. This occurred on 20 of the 149 control sections. A check was made to determine the difference between travel times calculated from six passes and that

TABLE 1

RESULTS OF CHI SQUARE "GOODNESS OF FIT" TEST[a]

Control Section		Inbound						Outbound					
					Freedom						Freedom		
Limits	Length (mi)	T_m (min)	s (min)	X^2	Deg.	Sign. Level	X^2	T_m (min)	s (min)	X^2	Deg	Sign. Level	X^2
Church St.- Lea Ave.	0.57	2.39	0.385	7 92	4	0.05	9.49	2.18	0.404	23.891	4	0.05	9.49
Lea Ave.- South St.	0.51	1.38	0.197	26.63	4	0.05	9.49	1.39	0.246	13.72	3	0.01	11.34
South St.- Bradford Ave.	1.25	2.81	0 292	10.07	4	0.025	11 14	2.76	0.240	2.32	4	0.5	3.357
Church St.- Bradford Ave.	2.33	6.36	0 429	5.14	3	0.10	6.25	6.22	0.646	10.63	4 4	0.05 0.025	9.49 11.14

[a]As applied to 51 travel time passes in each direction on 8th Ave. between Church St. and Bradford Ave.

calculated from ten passes. On 17 of the 20 sections, there was 1 mph or less difference. Only one section had as much as 3 mph difference and this section was somewhat of a special case. It was only about two blocks long and was called a control section only because of geometric characteristics substantially different than either of the adjoining ones. All this seems to indicate that even if the check is not met after six passes, that more passes will not necessarily alter the average travel time.

On completion of the data collection, the average speed for each control section was calculated for the various periods of the day. Figure 1 shows the speeds for each section for the evening peak hour. This presentation shows at a glance where the lower speeds are found during this period. The solid black lines represent average speeds of 5 to 10 mph. The clear lines show speeds of more than 40 mph. Similar figures were prepared for the morning peak hour and off-peak periods.

The time contour map is a well-known tool developed from travel time data (Fig. 2). There are two sets of time contours on this map and the picture they present is of interest. The solid lines were plotted from morning peak travel times which were inbound to the central business district. The dashed lines were plotted from evening peak hour times which were outbound from downtown. The same streets are involved but there is considerable difference in the two patterns. The morning pattern is shifted to the southwest relative to the evening pattern. Thus, some streets which offer rather poor service during the morning peak hour offer better service during the evening peak

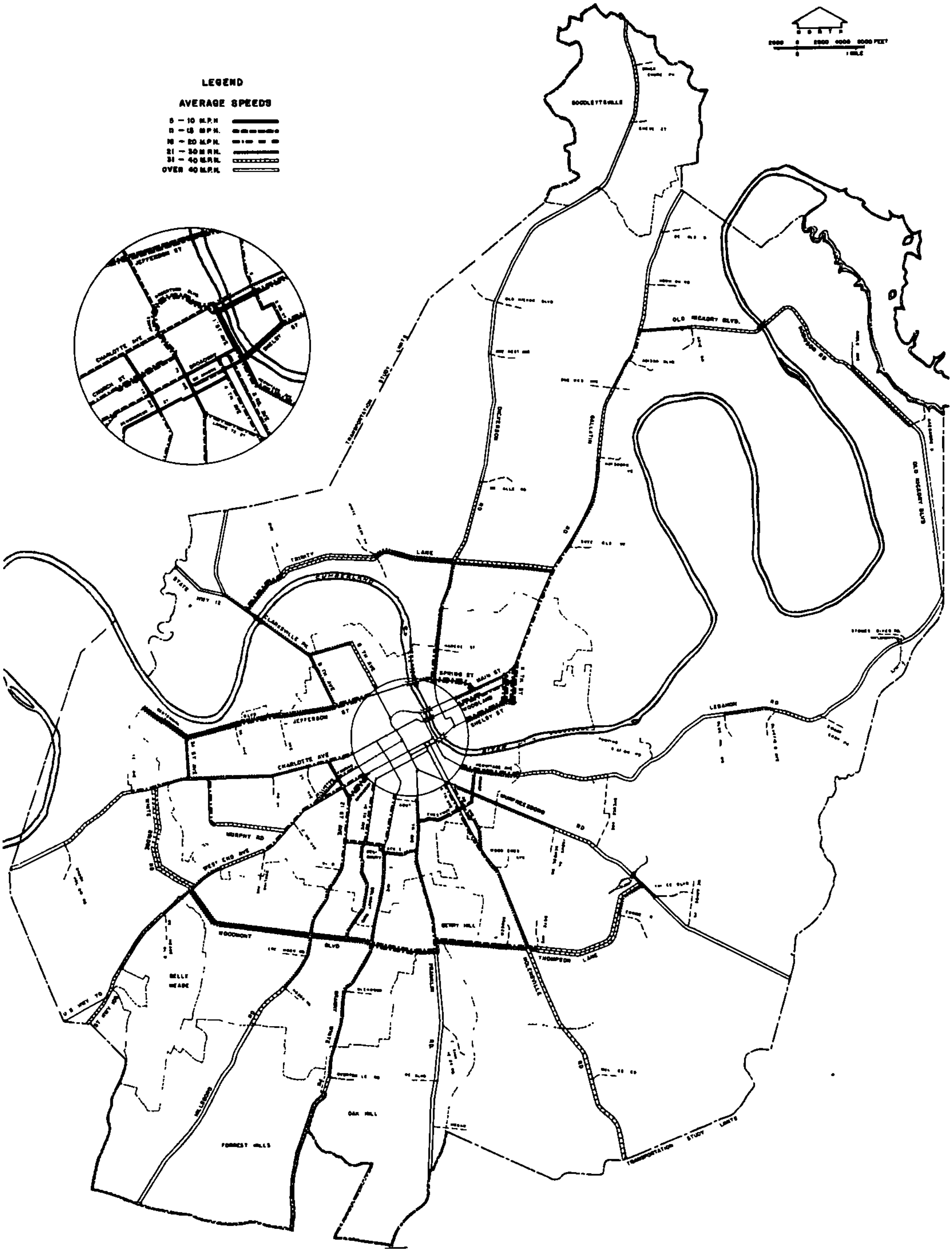


Figure 1. Average over-all speeds on major arterial streets during PM peak hour (winter 1958-1959).

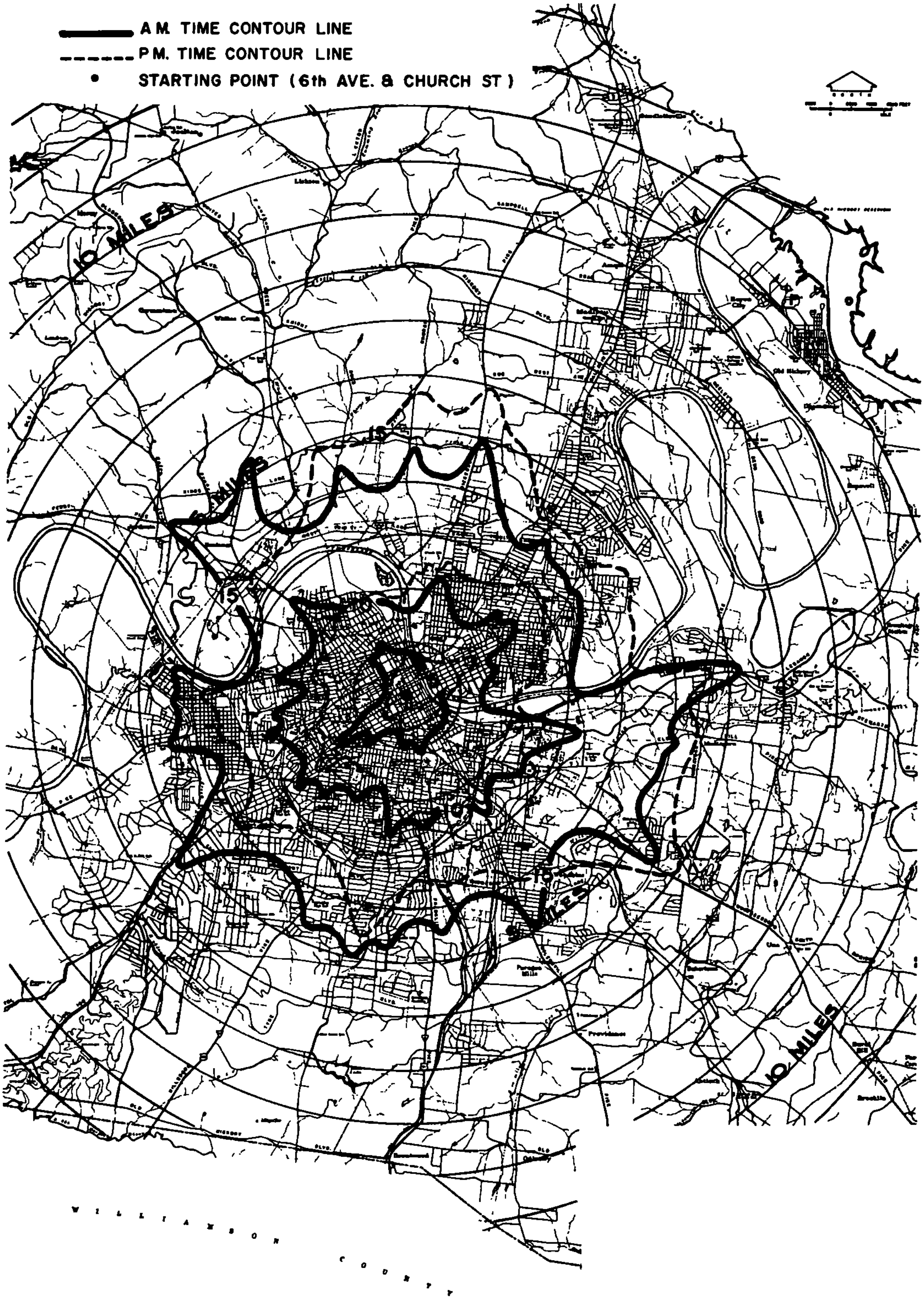


Figure 2. Time contour map (peak hour).

and vice versa. Figure 3 illustrates this in another manner. It is a cumulative frequency diagram and shows the evening peak hour speed for each control section as a percent of the morning peak hour speed. The variation is quite large. The PM speed varies from 0.5 to 3.5 times the AM speed. On only about 40 percent of the control sections is the difference less than 10 percent. This is significant in that is shows the need for data representative of both peak hours in order to say the level of service on this or that street is more deficient than that on another street.

The original intent of the Travel Time Study was to produce a measure of street deficiency. To accomplish this, the following desirable peak hour speed criteria were agreed on by the Technical Coordinating Committee: downtown — 15 mph, and more than 1 mi from the center of downtown — 25 mph. The speed on each control section was compared to the applicable criteria. The difference between the actual speed on a control section and the criterion speed constitutes a speed deficiency or sufficiency for the section. A deficiency is converted from miles per hour to minutes per mile and multiplied by the peak hour volume for the control section. The result is the delay experienced by traffic on the control section. It is expressed as vehicle minutes of delay per mile. Adding the delay experienced during both peak hours gives the total peak period delay.

Considerable delay was experienced on some control sections during the evening peak hour and on others in the morning peak. There was delay on still other sections during both peaks. Combining the delay for the two peaks gives a more nearly complete picture of the comparative level of service on each control section. This is in agreement with the fact that there was considerable variation in the speeds for the AM peak and the PM peak. Figure 4 shows this vehicle delay for the two peak hours. A glance at this figure shows where the more serious delays and congestion exist.

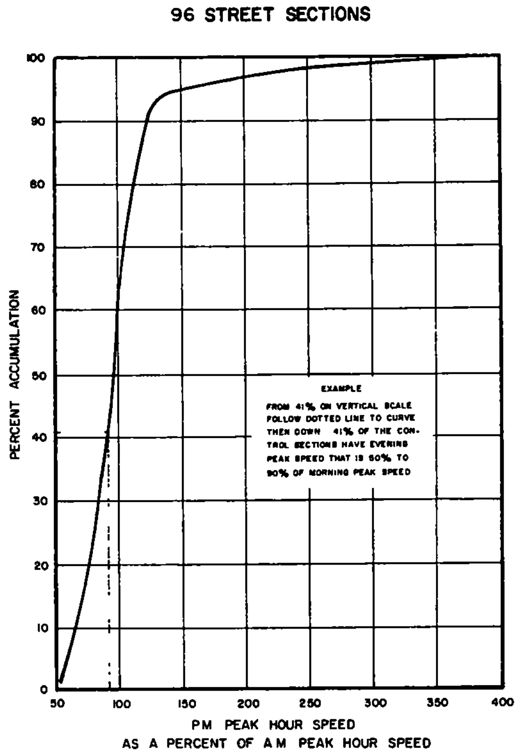


Figure 3. Cumulative frequency curve showing PM peak hour speed as a percent of AM peak hour speed.

Finally the travel times were combined to produce over-all times between various parts of the urban area. These were again compared to desirable or criterion travel times to determine deficiencies. The criteria used are as follows:

MINIMUM DESIRABLE AUTO TRAVEL TIME[a] FOR TRIPS OF VARIOUS LENGTHS
(Measured on the Basis of Airline Distance)

Average Weekday Peak Hour	
Length of Trip (mi)	Travel Time (min)
2	7
4	12
6	16
8	20
10	24
12	28

[a]"Better Transportation for your City." NCUT, Pub. Adminis, Service, Chicago, p. 49.

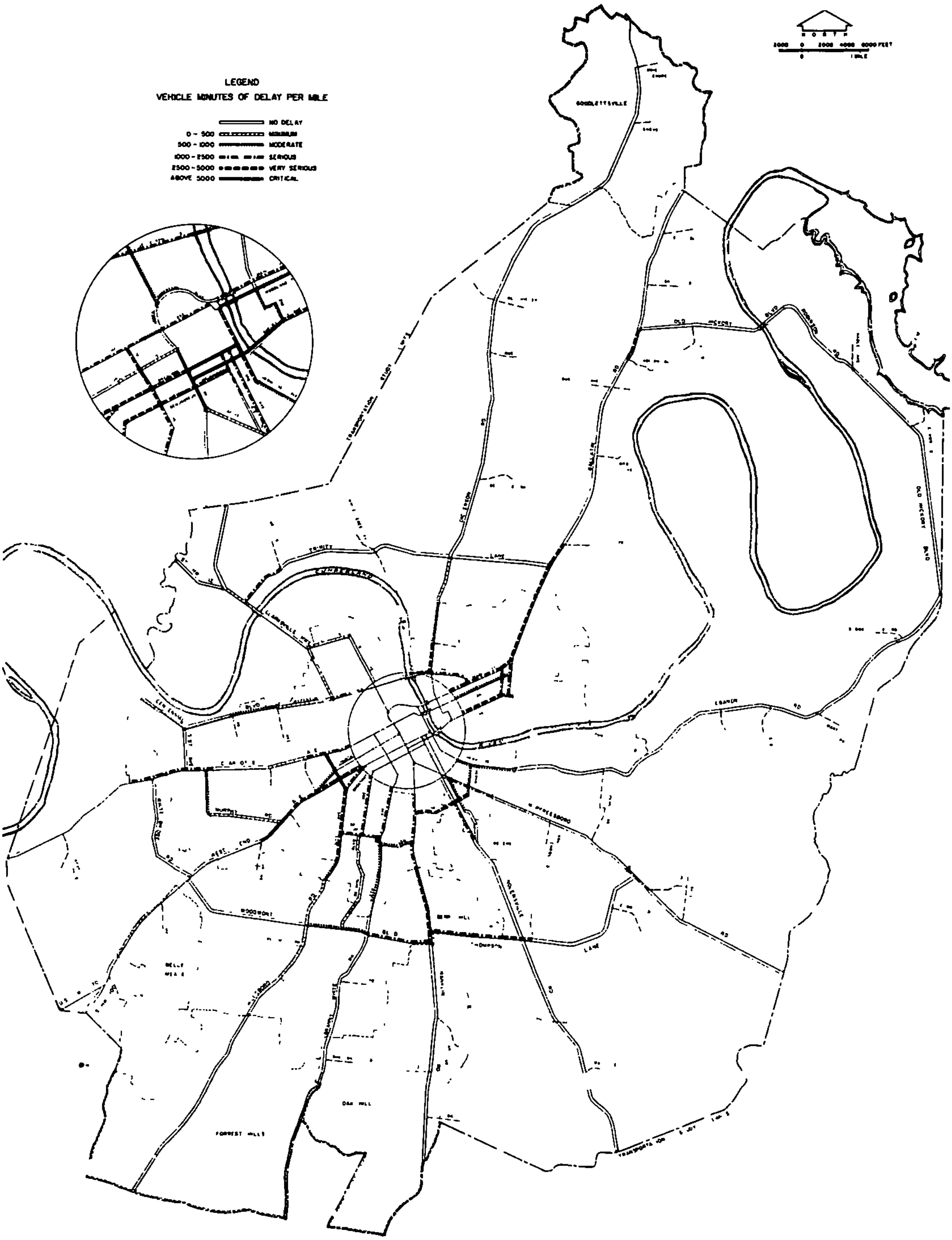


Figure 4. Vehicle minutes of delay per mile on major arterial streets during AM and PM peak hours (fall-winter 1959).

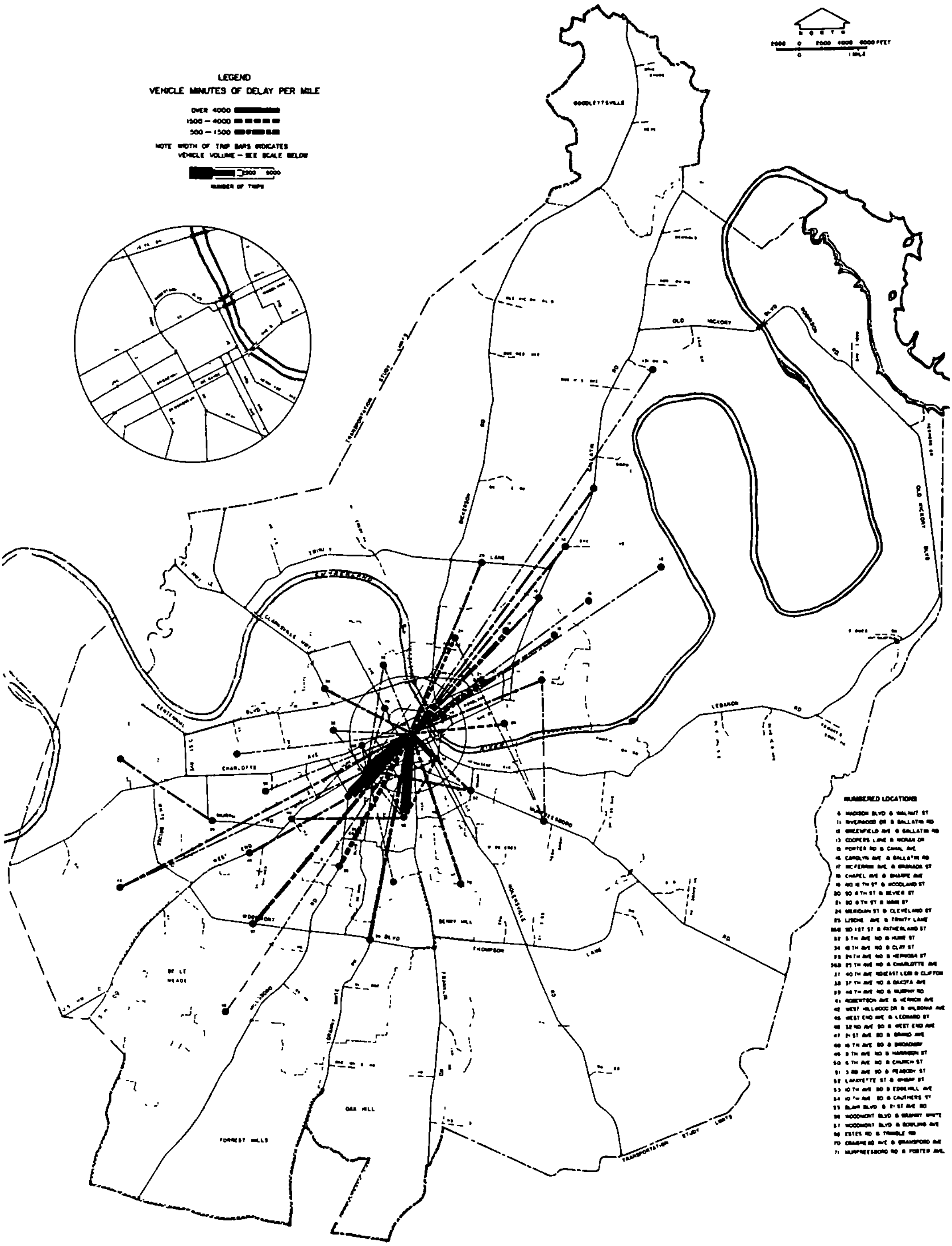


Figure 5. Vehicle minutes of delay per mile for trips between various parts of the community (based on an average of AM and PM peak hour travel times—fall-winter 1959).

The resulting deficiencies are relative, and show the level of service between any two areas as it compares to that between other areas.

The actual 24-hr volume of trips between the different areas was furnished by Wilbur Smith and Associates in the Origin-Destination Study. The vehicle delays obtained by multiplying these volumes by the average peak hour travel time deficiency are relative delays, but give a good picture of the comparative over-all deficiencies in the area.

Figure 5 shows the vehicle delay thus calculated. Each of the lines shown represents, by pattern, the amount of vehicle delay for the trip. The width of the line represents the number of vehicles making the trip in a 24-hr period. The lines are shown for only those trips having a delay of more than 500 vehicle minutes of delay per mile. It might

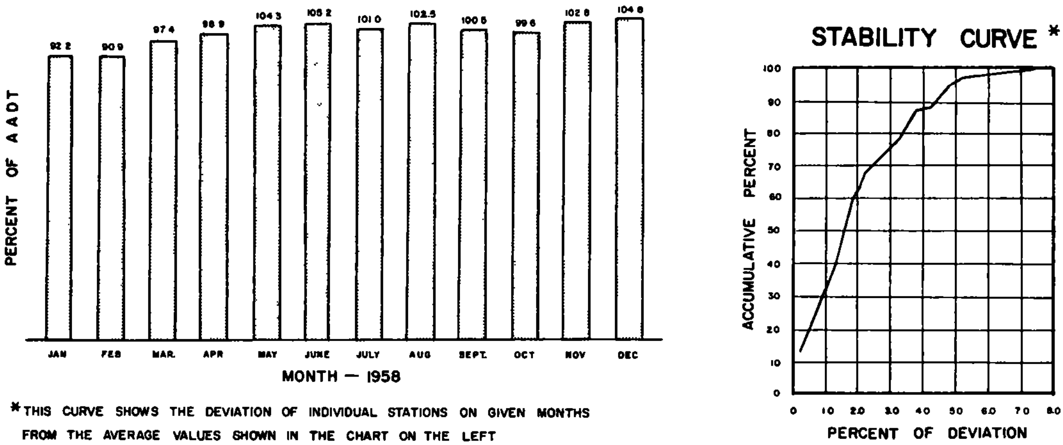
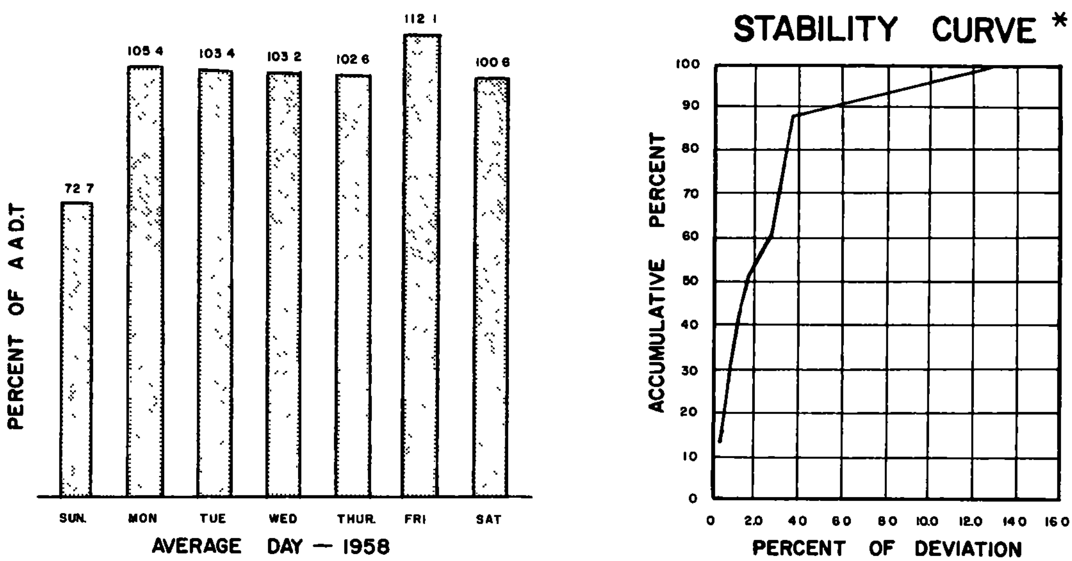


Figure 6. Average daily traffic for each month of the year as a percent of the annual average daily traffic (for 5 permanent counter stations in Nashville—major arterial streets).



* THIS CURVE SHOWS THE DEVIATION OF AVERAGES FOR INDIVIDUAL STATIONS FOR VARIOUS DAYS OF THE WEEK FROM THE AVERAGE VALUES SHOWN ON THE LEFT

Figure 7. Traffic for each day of the week as a percent of the annual average daily traffic (for 5 permanent counter stations in Nashville—major arterial streets).

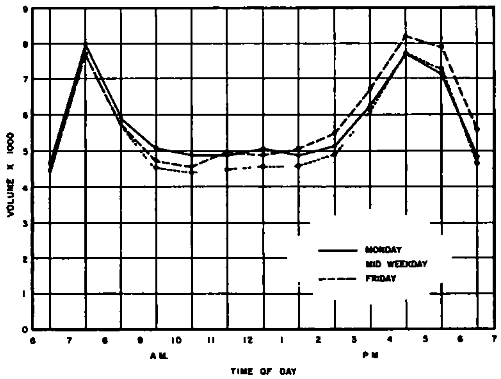


Figure 8. Comparison by hour of Monday and Friday daytime volume to other weekdays at all major arterial permanent counter stations.

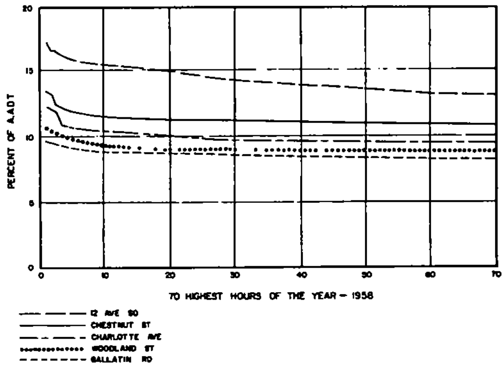


Figure 9. Seventy highest hours as a percent of annual average daily traffic (for arterial stations in Nashville).

be explained that some trips not shown had more delay than some that are shown. (In one instance, the actual time delay was over 11 min per mile.) However, the volumes were so low (zero in many cases) that the vehicle minutes of delay were negligible. Travel delay represents economic loss only to the extent that road users are affected. Thus with no one making the trip, it was of little immediate concern whether or not such a trip would have a travel time deficiency.

This sort of analysis shows very definitely where relative deficiences exist in the level of service offered by the major

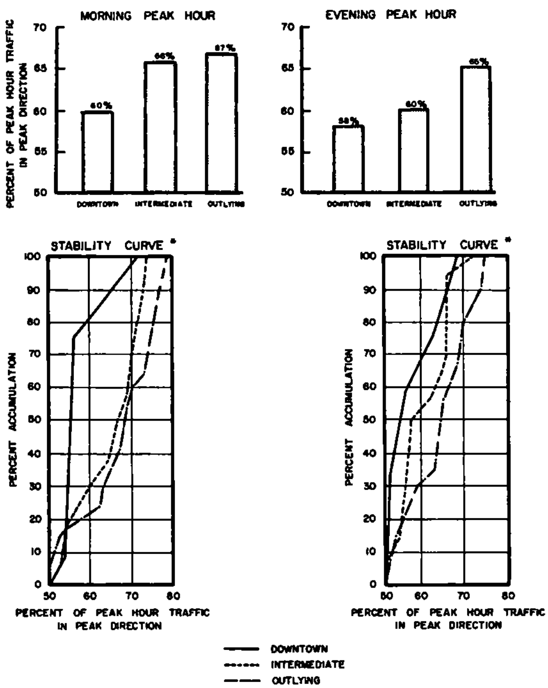


Figure 10. Percent of peak hour traffic moving in the peak direction (for 48 downtown, intermediate and outlying directional count stations).

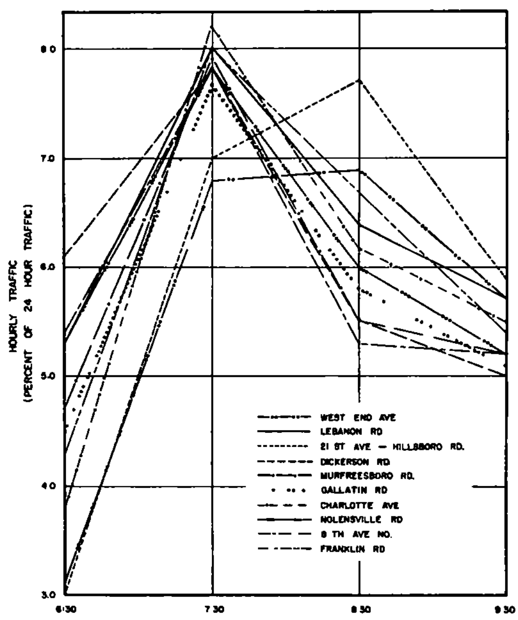


Figure 11. AM peak hour traffic as a percent of 24-hour traffic (for various traffic corridors).

TABLE 2

TIME OF OCCURRENCE OF THE 70 HIGHEST HOURLY VOLUMES OF THE YEAR AT 5 PERMANENT COUNTER STATIONS

Location	Percent of Highest 35 Hours That Occur During			Percent of Highest 70 Hours That Occur During		
	Weekday AM Peak (%)	Weekday PM Peak (%)	Other (%)	Weekday AM Peak (%)	Weekday PM Peak (%)	Other (%)
Gallatin Rd.	-	74	Sat. midday, 26	-	73	Sat. midday, 27
Charlotte Ave.	9	89	Mon. 3-4, 2	11	88	Mon. 3-4, 1
Woodland St.	91	9	-	89	11	-
12th Ave. So.	100	-	-	86	14	-
Chestnut St.	11	89	-	13	86	Sun. 5-6, 1
Average	42	52	6	40	54	6

street system. The analysis points up the areas connected by major arterials having the greatest travel delay.

When the Traffic Volume Study was begun, Nashville was perhaps more fortunate than some other cities. The State of Tennessee maintains five permanent count stations on the major arterial system. There were several years' data which, for the most part, were simply filed away. In addition, there were approximately 100 volume counts made at locations on the major arterial or collector street system. These counts were made by both Nashville and the State of Tennessee and were all less than 2 yr old. In many cases there were counts for 2 yr. These were compared to be sure that the one that was used was not an abnormal count. It was felt that they would be quite adequate for studying various volume patterns.

The permanent counter data were used first to determine the monthly factors. Data

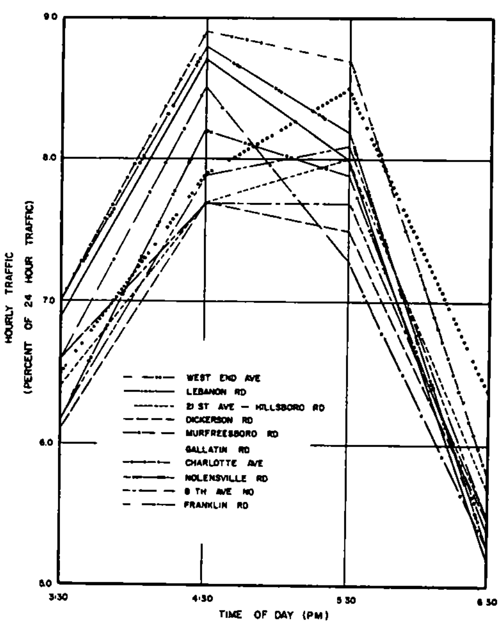


Figure 12. PM peak hour traffic as a percent of 24-hour traffic (for various traffic corridors).

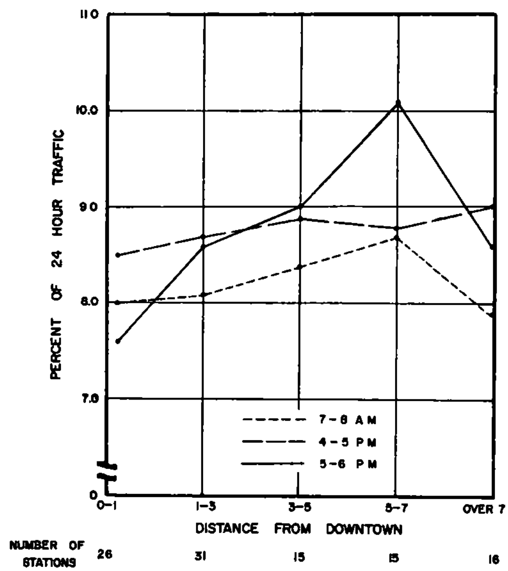


Figure 13. Peak hour traffic as a percent of 24-hour traffic at various distances from downtown (103 stations in the Nashville urban area—weekdays).

from the five permanent counters, mentioned earlier, were averaged for each month (Fig. 6). To determine the reliability or stability of these factors, the cumulative frequency curve was prepared. It shows the deviation for each station for each month from the average. The monthly average daily traffic at none of the locations deviated from the average more than 7.5 percent of the annual average daily traffic for any month and 75 percent of the time the deviation was less than 4.0 percent. This analysis indicates what sort of errors might be introduced by using these monthly factors. Actually, there are deviations between locations that are greater than the variations between the months.

The same type of analysis was used to arrive at Figure 7 which shows the traffic for each day of the week as a percent of the annual average daily traffic. As is normal, Friday has the highest traffic volumes. Monday's traffic is also higher than the other weekdays. The stability curve was prepared in the same manner as the previous one and shows that 88 percent of the time the volume, for any given day of the week, at one of the stations will deviate less than 4.0 percent of the average daily traffic from the average shown.

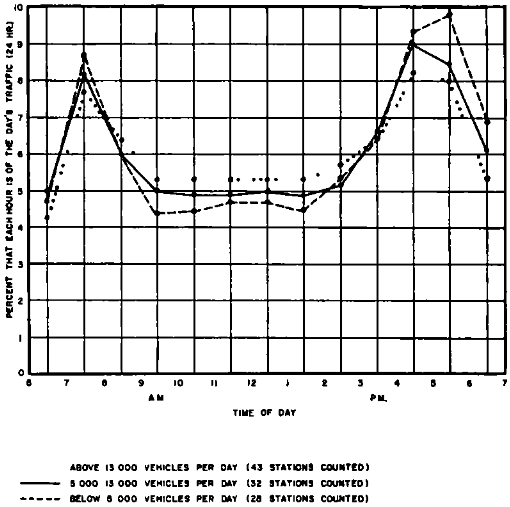


Figure 14. Hourly traffic as a percent of daily traffic for three volume groupings (103 stations in the Nashville urban area).

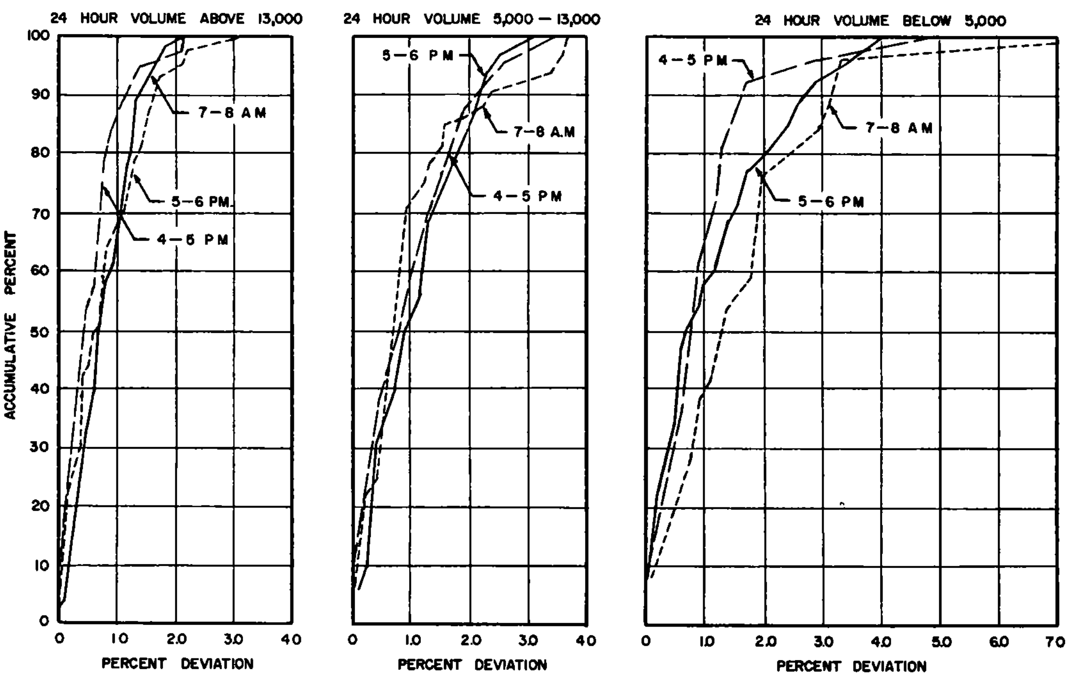


Figure 15. Stability of hourly variation curves—weekday (deviation of each of 103 stations for peak hours from the average shown in Figure 14).

The higher traffic volumes on Monday and Friday introduced the following questions: (a) Is this volume increase uniform throughout the day or does it occur during a certain part or parts of the day? and (b) Does this reason hold true for both Monday and Friday?

Data from the permanent counters were analyzed and the result is Figure 8. Actually there is little difference in the morning peak hour. Monday has a very slightly higher peak than the other weekdays. The higher volume results from greater volumes during the mid-day off-peak hours. Friday, on the other hand, has a very high evening peak hour volume. In fact, it is 10 percent higher than any other weekday. In addition, the Friday volumes from mid-morning on are considerably higher than those of mid-week days.

Figure 9 shows the 70 highest hours for each station as a percent of the average daily traffic for that station. In exploring the old 30th highest hour idea, it can be seen that in Nashville's case almost any hour between the 10th and the 70th would be usable. To say that 10 percent of the average daily traffic will be used as a design hour volume is not far afield. The street which is the exception here has one of the highest peak hour volumes relative to its daily traffic of any of the 103 locations studied. It serves a predominately residential area and has almost no commercial traffic and thus its resulting hourly pattern is not considered very representative of the major streets in Nashville.

Also of interest is the time of occurrence of the 70 highest hours on the various streets. Table 2 shows that 54 percent of them occurred during the evening peak hour, 40 percent during the morning peak and the other 6 percent at various other times. Actually three of the five stations had most of their high hours during the evening peak hour. The other two had their highest hours during the morning peak.

Forty-eight of the counts used in the Volume Study were directional counts.

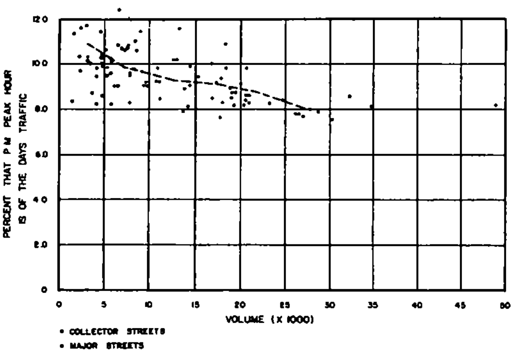


Figure 16. Peak hour traffic volume vs 24-hour volume (at counter stations within the Nashville urban area).

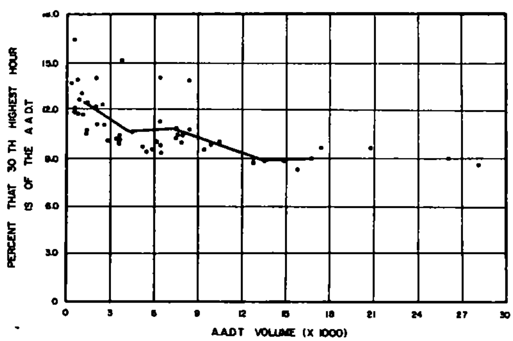


Figure 17. Thirtieth highest hour traffic volume vs annual average daily traffic (at urban permanent counter stations in Tennessee).

Past these locations, the percent of peak hour traffic in the peak direction ranged from 50 percent to 78 percent. Several groupings of data were tried to "narrow" this spread.

The following were tried without success:

1. Group by distance from downtown — four groups were used, but no pattern could be established.

2. Group by 24-hr volume. The data were split into two groups with the dividing point at 20,000 vehicles per day. The average peak direction volume (as a percent of the day's traffic) was the same in both groups.

Finally the counts were grouped according to whether the individual location was classified as downtown, intermediate or outlying, based on the Highway Capacity Manual definitions. Figure 10 shows the relationship found from this grouping. It should be mentioned that when an area, and all count locations within that area, were classed as downtown, intermediate, or outlying, no pattern could be established. Although the trend toward greater splits at outlying locations is not pronounced, it appears to definitely be present. The "stability" curves bear this out. These are simply cumulative frequency curves. The one for outlying stations is to the right of the intermediate one which is in turn to the right of the downtown curve. This indicates generally higher peak direction percentages at intermediate locations than downtown and still higher ones at outlying locations.

One of the more interesting analyses involved the hourly variation at the 103 locations. As is normally found, three patterns were noted. The normal weekday with work trips creating the peaks; Saturday, with a combination of commerce, shopping and recreation and finally Sunday, with its church and recreation traffic. The weekday traffic is the most significant because it occurs five days out of seven and about 95 percent of the highest traffic hours occur during these days.

The wide spread found in peak hour volumes (expressed as a percent of 24-hr traffic) was a cause of concern. A look at the data showed that evening peak hour volumes varied from about 6 percent of the day's traffic to over 11 percent. Obviously this does not tell very much about peak hour traffic.

The data were grouped in various fashions to find a pattern or group of patterns which would be more representative and allow more accurate estimation of day to day peak hour traffic.

These groupings are worthy of comment even though they did not produce the desired end result:

1. Corridor Analysis. — The urban area was divided into corridors, each one bisected by a major radial street. Several 24-hr counts made at different locations along the bisecting street were averaged to determine the hourly pattern for each corridor. The hourly volume patterns for the two peak periods are presented in Figures 11 and 12. In each corridor more variation was found between the various stations within a corridor than between the average patterns for the various corridors. It is noted in Figure 11 that two streets have a decidedly different pattern than all the other streets. These two streets serve residential areas in the higher economic brackets.

2. Street Classification Analysis. — The 103 volume counts referred to above were separated into major arterial and collector street groups. The collector street volume pattern in general had higher peak hour volumes percentagewise than major arterials. However, some major arterials had higher peak hour volumes percentagewise than many collectors. Again, there was as much as or more variation between individual stations within a group than between the groups.

3. Distance from the Center of the City (Ring Analysis). — The volume counts were grouped this time according to distance from the center of the city. The average hourly pattern was established for each group. Peak hour traffic for each distance group is shown in Figure 13. It appeared that there was possibly a pattern related to distance from the center of the city, because there is an indication that peak hour traffic, as a percent of 24-hr traffic, increases as distance from the central business district increases. This is especially true of the curve representing 4:00 PM to 5:00 PM. It was decided, however, that because traffic volumes in general decrease as distance from the central business district increases, this was another indication that the peak hour traffic, as a percent of the day's traffic, increases as the 24-hr volume decreases. Discussion of this follows.

4. Volume Grouping Analysis. — The 24-hr volume counts were separated into three volume groups, as follows: 0 - 5,000 vehicles per day, 28 counts; 5,000 - 13,000 vehicles per day, 32 counts; and 13,000 and above vehicles per day, 43 counts — total counts = 103. The resulting hourly volume patterns are shown in Figure 14. The deviation from these curves by the individual stations was calculated for the peak hours. It is the difference between the percent of the day's traffic in the peak hour at each station and the average percent of the day's traffic in the peak hour for that volume

group. Figure 15 shows these deviations as cumulative frequency curves. The use of this figure is illustrated by the following example:

Assume a given street falls into the 5,000 - 13,000 vehicles per day volume grouping. From Figure 14 it is seen that on the average, the traffic flow from 5:00 - 6:00 PM will be 8.5 percent of the 24-hr traffic. Figure 15 shows that the 8.5 percent value will be correct within 2.0 percent for 85 percent of the streets within this volume range; that is, the value will fall between 6.5 percent to 10.5 percent, 85 percent of the time based on existing data.

To add more support to the argument that peak hour volumes decrease percentage-wise as 24-hr volume increases, Figure 16 was prepared. It shows the percent of the day's traffic during the peak hour plotted against the 24-hr volume for each count station. Although there is considerable "scatter" to these data, there is a definite trend toward proportionately lower peak hour volumes with an increase in the 24-hr volume.

This same trend is in evidence in comparing the 30th highest hour volume as a percent of the average daily traffic to average daily traffic for urban counting stations in various parts of the state (Fig. 17). Stations having higher average daily traffic in general have lower 30th highest hour volumes when expressed as a percent of average daily traffic.

CONCLUSIONS

From these analyses there are certain conclusions which are indicated although admittedly the data are somewhat limited in some cases. These conclusions, as a result, are of a general nature.

1. Travel time can provide a good measure of the level of traffic service provided by a street. To present a complete picture of this service, data representing more than just the evening peak hour is necessary.
2. The procedures prescribed by the National Committee on Urban Transportation will give an adequate sample for travel time measurement.
3. Morning peak hour travel time passes may be made on any weekday. Evening passes may be made any weekday except Friday. Weekday morning peak hour traffic volumes are fairly constant and the same is true of evening peak hour volumes except that for Friday, which is about 10 percent higher.
4. As 24-hr traffic increases, the peak hour traffic as a percent of this 24-hr traffic decreases.

There were a few other conclusions drawn which applied specifically to Nashville and are not mentioned here for that reason. It is hoped that the discussions in this paper will add to the present knowledge of factors affecting urban transportation.

HRB:OR-456

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