308

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Operational Effects of Design And Traffic Engineering

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Operational Effects of Design And Traffic Engineering

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Effect of Traffic Improvements on Operation Of an Urban Arterial Street

EDMUND A. HODGKINS, Highway Engineer, U.S. Bureau of Public Roads

● IN 1959, a study was undertaken to evaluate methods for "Increasing the Traffic Carrying Capability of Urban Arterial Streets." This study, which was made on 4½-mi-long Wisconsin Avenue in Washington, D.C., became popularly known as the "Wisconsin Avenue Study" (1).

In 1960, the first of a proposed continuing series of Wisconsin Avenue "after" studies was made, the basic objective being to determine to what extent certain traffic engineering improvements adopted in the interval of time since the original study was made, had affected the capacity, the traffic volumes carried, the speeds, and the travel time. Essentially, these improvements on the arterial included removal of streetcars and loading platforms, establishment of new parking and turn regulations, and changes in signal timing.

The after study consisted of an inventory of the traffic engineering features as they existed on Wisconsin Avenue during the spring of 1960, of field studies of eight critical intersections during peak periods, and of test vehicle runs to determine current speeds and travel times. Speed data, traffic counts, and intersection studies supplied by the District of Columbia Department of Highways and Traffic also were used in evaluating the results. No attempts were made to restudy traffic friction events, to conduct further galvanic skin resistance (GSR) tests, or to repeat several related studies made in the original 1959 study. Figures 1 through 3 show the study street.

CHANGES IN THE STREET

The improvements recommended in the original Wisconsin Avenue report were classified into three phases, as follows:

Phase 1—those possible at relatively little or no cost;
Phase 2—those requiring moderate expenditures; and
Phase 3—those requiring major expenditures and construction.

An inventory, comprising a series of field checks complemented by information received from the District of Columbia Department of Highways and Traffic, was made to determine by phase the improvements that had been effected. Of the eleven improvements suggested for phase 1, only one, the conversion of the transit line from streetcar to bus operation, had been fully adopted at the time this study began in April 1960. Three other phase 1 improvements pertaining to turning movement controls, removal of parking, and one-way streets had been partially adopted. Lane markings, control of U-turns, controls on turning movements in midblock, and improvements to off-street parking area operation had not been effected. Except for the signal progression, none of the improvements recommended in phases 2 and 3 had been undertaken.

The specific changes revealed by the inventory were as follows:

1. The transit system had been converted from a streetcar to a bus operation for about 3 months. The loading platforms had been removed, but the streetcar tracks still remained.

2. All parking on Wisconsin Avenue in Georgetown had been prohibited during peak hours. From R Street to Massachusetts Avenue "no standing" regulations were in force from 7 to 9:30 a.m. on the west side and from 4 to 6:30 p.m. on the east side. "No parking" regulations were in force on the opposite side of the street during these

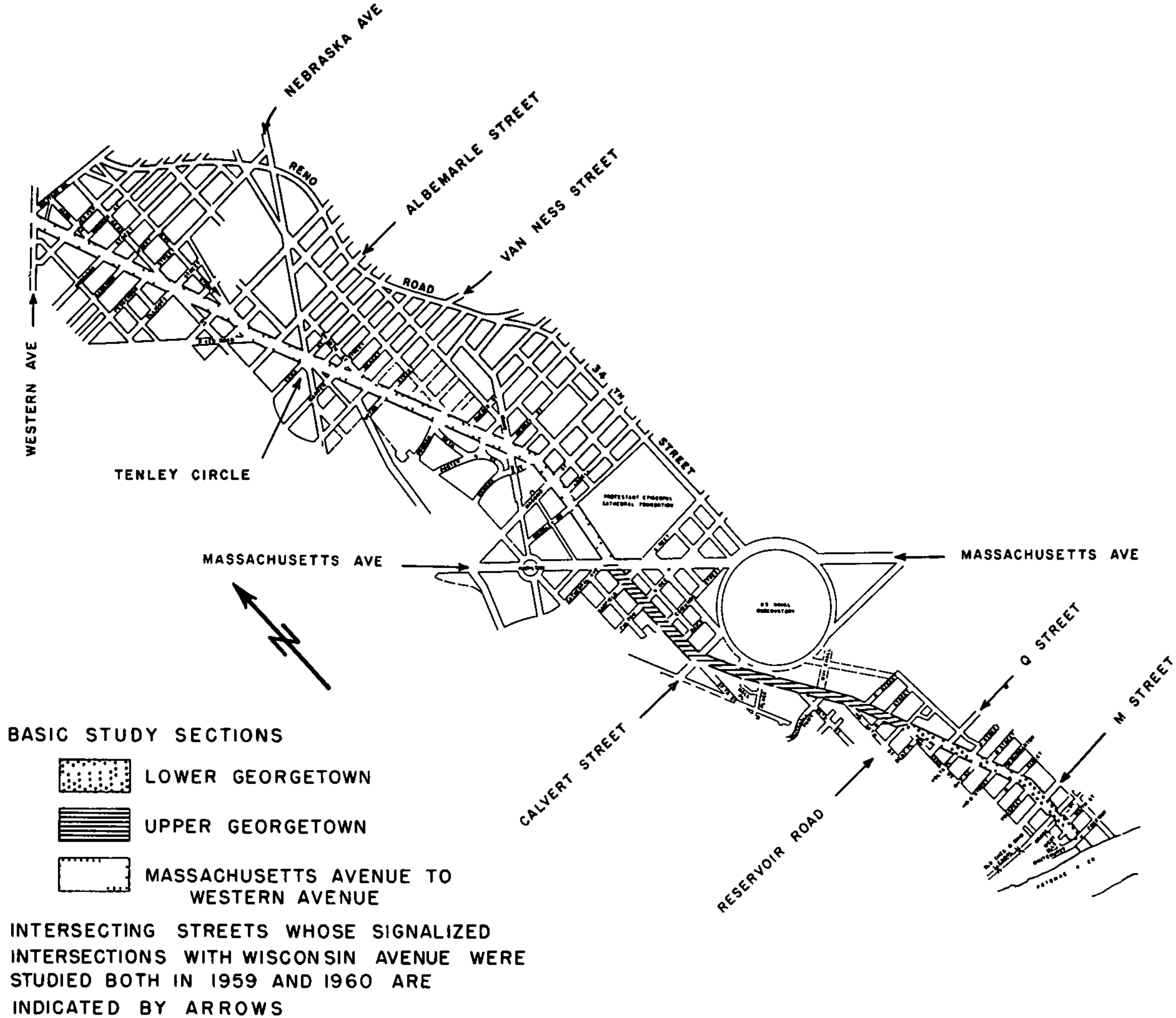

Figure 1. Wisconsin Avenue and vicinity, Washington, D. C.

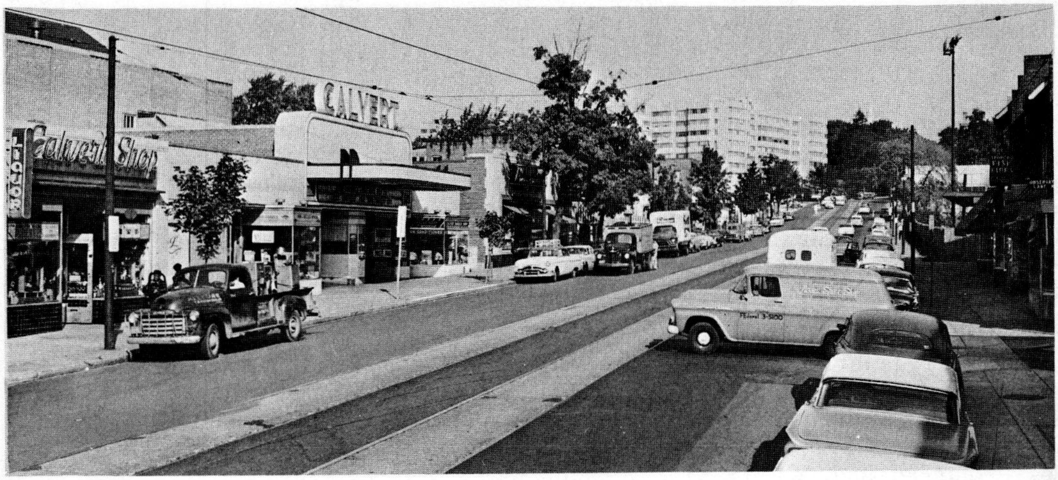

Figure 2. Looking north on Wisconsin Avenue toward the Calvert Street intersection, from Observatory Lane, during off-peak conditions.

periods, except that "no standing" regulations were in force between Garfield Street and Massachusetts Avenue from 4 to 6:30 p.m. on the west side.

3. There had been relatively little change in parking regulations north of upper Georgetown since the original study. Areas north of Massachusetts which were scheduled to be affected by similar new "no parking" and "no standing" regulations had not yet been signed, except at the intersections at Van Ness Street and at Western Avenue.

4. Left turns had been prohibited from Wisconsin Avenue into Massachusetts Avenue, Tenley Circle, and Western Avenue. Also traffic westbound on M Street and on Western Avenue, and traffic both eastbound and westbound on Massachusetts Avenue, had been denied left turns into Wisconsin Avenue. At Tenley Circle direct left turns into Wisconsin Avenue had been eliminated; instead, traffic was routed around the circle to turn right into the Wisconsin Avenue traffic stream.

5. Thirty-seventh Street had been made one-way southbound from Wisconsin Avenue to Tunlaw Road, and Thirty-ninth Street one-way northbound from Veazey Street to Albemarle Street.

6. A progressive signal system had been provided with progression set for 25 mph. It favored the northbound traffic during the afternoon peak period and the southbound traffic during the morning peak period and was radio-controlled from Q Street to Massachusetts Avenue. A 65-sec cycle was being used in the off-peak hours at all intersections except M Street and Western Avenue where a 90-sec cycle and an 80-sec cycle, respectively, were used at all times. The other intersections were on an 80-sec cycle during peak hours. Rodman Street was the only intersection which was signalized during the after study but not during the before study.

BASIC FACTORS CONSIDERED

For the purposes of comparison, Wisconsin Avenue was divided into three sections as in the original study. These are (a) lower Georgetown—K Street to Reservoir Road; (b) upper Georgetown—Reservoir Road to Massachusetts Avenue; and (c) north of Massachusetts—Massachusetts Avenue to Western Avenue. The factors to be compared for each section, as indicators of improved operation, were intersection traffic volumes, number of "loaded" (that is, fully utilized) signal cycles, and speeds, including average speeds, running speeds, and number of stops.

Careful consideration was given to the choice of factors to be used as indicators of improved operation. The following paragraphs contain brief justifications for the use of each factor as an indicator, and discuss the methods used for obtaining and using the data.

Volumes

Generally on a well-established arterial, only gradual increases in volume are noted, year after year. If larger than expected increases are noted, it is very likely that something has been done to attract this increased traffic. Any improvement in the traffic-carrying capability of the street would of course provide such an attraction.

Manual counts were used to obtain volumes on Wisconsin Avenue at selected intersections. Because it was not possible to study all of the intersections included in the before study, it was necessary to consider two methods of comparing the volume data obtained in the two studies, as follows:

1. Average directional peak-hour volumes for the selected intersections in the after study were compared with the corresponding average directional volumes for all of the intersections in the before study. Use of this method meant, for each of the three street sections, comparing a small number of intersections in the after study with the larger number included in the before study.
2. Average directional peak-hour volumes for the selected intersections in the after study were compared with the average directional volumes for only the same intersections in the before study. This method limits the before and after comparisons to the

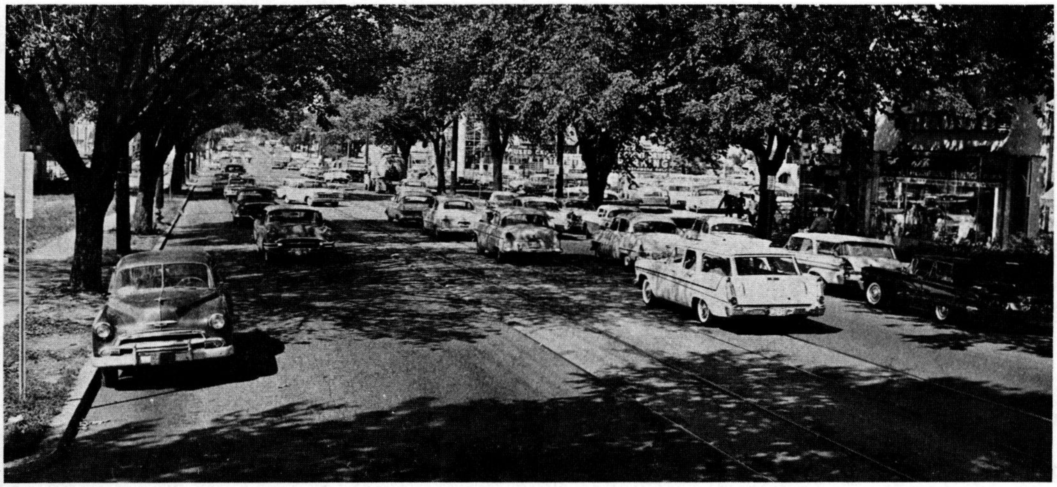

Figure 3. Looking north on Wisconsin Avenue toward the D. C.-Md. line at Western Avenue, Northern terminus of the study, from north of Jenifer Street, shortly before the P.M. peak period.

intersections studied in the after study. It was concluded that the second method, although not using all of the available 1959 data, gave the most valid comparisons. Consequently, this is the method on which the volume comparisons cited in the report were based, although both methods are treated in the tabular summaries of data.

Generally, approach volumes at the intersections were used in computing average volumes for each of the three sections of Wisconsin Avenue. Three exceptions were made at points where it was felt that a true picture of the actual volumes within the section being discussed required use of the volumes leaving the intersection, due to major turning movements occurring at the intersection. For the northbound traffic flow it was felt that volumes leaving the intersection at M Street were more representative of lower Georgetown as a whole than the approach volumes at the same point. Similar reasoning dictated that traffic leaving the intersection at Calvert Street should be used in computing southbound volumes for the upper Georgetown section, and that traffic leaving Massachusetts Avenue should be used in computing the northbound volumes for the section north of Massachusetts Avenue.

Loading

A good index of the congestion at a signalized intersection is the number of "loaded," or fully utilized, cycles at that intersection. Briefly, a loaded cycle on a specific intersection approach is one in which all lanes of that approach are fully utilized throughout the green period, either by a continuous flow of traffic or by traffic present and ready to move, but blocked by conditions within that intersection. For any given intersection, loading is usually expressed as the percentage of loaded cycles among the total number of cycles occurring during the peak hour.

In analyzing the intersection data it became necessary to relate loaded cycle time to intersection capacity. In theory it is possible to have 100 percent loaded cycle time when the intersection is operating at practical capacity. Again, according to theory, it is possible for the intersection to be operating very close to practical capacity and not have any loaded cycles during the period studied.

In actual practice the occurrence of either of these conditions is very rare. The following conclusions were reached based on field observations:

1. If the recorded data indicate intermittent cycle loading, the intersection is carrying volumes close to the practical capacity of that intersection.
2. If the recorded data show a sustained period of loaded cycles, the intersection, during that period, is carrying volumes greater than the practical capacity of that intersection.

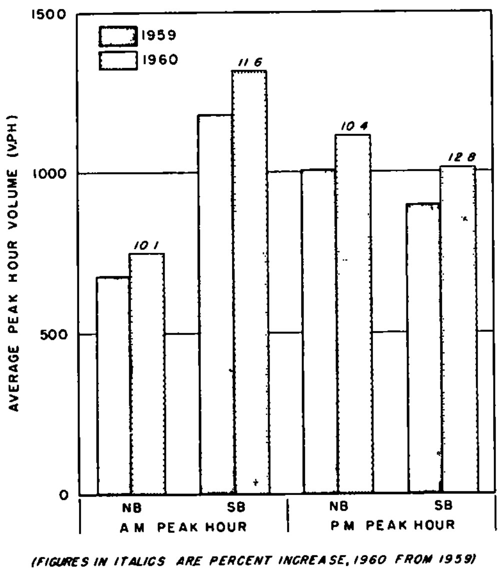

Figure 4. Average peak-hour traffic volumes at eight intersections on Wisconsin Avenue (K Street to Western Avenue), which were studied both in 1959 and 1960.

In the original Wisconsin Avenue study it was pointed out that, insofar as the study street was concerned, the capacity was governed more by frictional features throughout the street than by intersection considerations. It was further pointed out that the first step in increasing the capacity of the street was the elimination of the turbulence produced by frictional factors.

If intersection considerations are to become governing factors in the operation of the street, midblock sections must be capable of supplying the intersection with sufficient traffic to permit them to operate at least at practical capacity. If this happened, development of a small amount of loaded cycle time would be evident at those intersections previously operating below practical capacity.

However, to insure a smooth flow of traffic, it is necessary to insure that the intersections do not so limit the traffic that they become bottlenecks or metering points. This situation would be indicated by a large increase in loaded cycle time. Of course the most desirable objective is to produce an integral system in which the practical capacities of the street sections are equal to the practical capacities of the intervening intersections.

Loading conditions were recorded for each signal phase of each signal cycle by the field parties making the intersection studies. Direct comparisons were made of the before and after loading values of the eight intersections studied both times. In addition, a general picture was given by comparing loading within study sections, as a whole. The percent of loaded cycles reported for each of the three sections of the street is an average of the loading occurring at the selected intersections studied in that section.

TABLE 1

PEAK-HOUR TRAFFIC VOLUMES, APPROACHING AND LEAVING ALL INTERSECTIONS STUDIED ON WISCONSIN AVENUE (VEHICLES PER HOUR) 1959-1960

	Northbound A.M.				Southbound A.M.				Northbound P.M.				Southbound P.M.			
	Approaching		Leaving		Approaching		Leaving		Approaching		Leaving		Approaching		Leaving	
Intersection	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960
M Street	216	249	538	583	597	748	537	538	319	563	549	832	551	710	251	289
Prospect Street	527		537		572		619		607		554		551		565	
N Street	529		514		502		519		547		546		549		536	
P Street	605		517		756		558		634		717		721		772	
Q Street	475	577	508	667	737	861	608	748	572	812	647	936	573	705	492	639
Reservoir Rd.	511		405		523		734		668		465		504		458	
R Street	527		619		633		565		624		962		594		595	
34th Street	605		569		1,090		866		1,067		1,052		728		528	
Whitehaven St.	563		533		1,006		1,155		1,103		955		734		793	
35th Street	553		739		1,192		1,008		952		1,105		1,007		738	
W Place	755		749		985		1,101		1,122		1,079		931		963	
Hall Place	779		769		1,094		1,073		1,292		1,289		1,039		1,073	
Calvert Street	696	779	781	769	747	888	1,062	1,100	1,075	1,267	868	1,107	720	910	946	1,050
Edmunds Street	588		615		845		832		872		936		763		847	
Mass. Avenue	537	610	826	868	1,105	1,160	611	1,297	744	796	1,008	1,112	926	995	672	1,202
Woodley Road	790		797		1,124		1,161		945		1,004		976		1,011	
Macomb Street	800		886		1,160		1,194		1,020		1,064		931		963	
Newark Street	803		754		1,077		1,104		1,134		1,096		921		919	
Idaho and Ordway	746		688		1,221		1,152		1,081		1,042		1,026		1,035	
Porter Street	752		879		1,178		1,258		1,139		1,283		993		1,027	
Rodman Street	879		896		1,220		1,167		1,021		1,053		912		869	
Van Ness Street	851	938	816	881	1,426	1,561	1,506	1,674	1,395	1,376	1,366	1,357	1,019	1,105	1,130	1,201
Tenley Circle So.	677	786	740	844	1,219	1,603	1,269	1,641	1,089	1,169	1,193	1,244	1,014	1,112	1,075	1,159
Tenley Circle No.	740	844	758	851	1,338	1,607	1,219	1,603	1,193	1,244	1,221	1,284	1,045	1,250	1,014	1,112
Albemarle Street	781	817	821	878	1,514	1,689	1,561	1,698	1,192	1,197	1,328	1,337	1,118	1,148	1,106	1,111
River Road	770		662		1,189		1,180		1,281		1,044		831		818	
Fessenden Street	586		611		1,136		1,230		950		1,000		824		837	
Western Avenue	566	578	753	809	1,636	1,539	1,339	1,204	1,009	1,017	1,196	1,224	889	1,042	683	784

The average for a section was computed by totaling the number of loaded cycles occurring during the peak hour at the selected intersections within that section and dividing that total by the total number of cycles in that section for the peak hour.

For example, suppose there are six intersections in a section, five of which have 80-sec cycles and one has a 90-sec cycle. The intersections with an 80-sec cycle have 45 cycles during the peak hour and the intersection with the 90-sec cycle has 40 cycles during the peak hour. Thus, the base number of cycles is the total number of cycles occurring in the section during the peak hour. In this case, it is 265 total cycles. Assume that the total number of loaded cycles occurring in that section is 27, then the result is 27 divided by 265, or 10.2 percent loaded cycle time.

Speeds

An increase in average speeds through a section of highway indicates that traffic is encountering fewer stops, or that it is moving faster between stops, or both. Increases in running speeds (that is, speeds between stops) more directly reflect midblock performance because most of the effect of stops is eliminated. However, running speed values do reflect decelerations to and accelerations from stops.

The travel speed studies were made by means of the test car in exactly the same manner as in the original study, the study street again being divided into ten control sections. This being the case, the results are directly comparable and all of the data obtained in both studies have been used in making these comparisons. The speed data for the ten control sections were consolidated into the same three basic sections used for comparative purposes.

TABLE 2

AVERAGE PEAK-HOUR TRAFFIC VOLUMES ON WISCONSIN AVENUE (VPH)

		Volume 1959		Volume 1960	Percent Increase	
Location	Direction and Time	All Available Intersections	Intersections Studied Both Years	All Intersections Studied	All Available Intersections	Intersections Studied Both Years
Lower Georgetown	NB AM	531	506	580	+9.2	+14.6
	SB AM	615	667	805	+30.9	+20.9
	NB PM	596	561	822	+37.9	+46.5
	SB PM	575	562	708	+23.1	+26.0
Upper Georgetown	NB AM	633	696	779	+23.1	+11.9
	SB AM	988	1,062	1,100	+11.3	+3.6
	NB PM	1,013	1,075	1,267	+25.1	+17.9
	SB PM	843	946	1,050	+24.6	+11.0
North of Mass. Ave.	NB AM	755	740	805	+6.6	+8.8
	SB AM	1,253	1,373	1,526	+21.8	+11.2
	NB PM	1,104	1,148	1,186	+7.4	+3.3
	SB PM	959	1,002	1,109	+15.6	+10.7
Over-all	NB AM	672	683	752	+11.9	+10.1
	SB AM	1,041	1,182	1,319	+26.7	+11.6
	NB PM	969	1,009	1,114	+15.0	+10.4
	SB PM	843	898	1,013	+20.6	+12.8

The speeds obtained from these data are the average speed and the average running speed as determined by the test car method. The average speed is based on the total time needed to traverse a section and includes all stops. The average running speed is the average obtained when the test car was traveling at speeds above 2 mph when decelerating and above 5 mph when accelerating.

"BEFORE" AND "AFTER" RESULTS

Volumes

The intersection studies showed increases in traffic volumes on Wisconsin Avenue, as given in Tables 1 and 2. In considering the results, it must be stressed that at the time of the after study the streetcars and loading platforms had been removed for only 3 months, and the major changes in parking regulations below Massachusetts Avenue had been in effect for less than 2 months. Still, over-all increases for the entire street ranged from about 10 to 13 percent, whereas average traffic volumes in the District of Columbia as a whole showed no change from 1959 to 1960. Average peak-hour volumes in the after study, for the street as a whole (Fig. 4), ranged from a minimum of 752 vph northbound during the morning to a maximum of 1,319 vph southbound during the morning, as compared, respectively, to 683 vph and 1,182 vph in the before study. For the individual sections (Fig. 5), the increases in directional peak period volumes ranged from a maximum of 47 percent for p.m. northbound traffic in lower Georgetown to a minimum of only 3 percent for p.m. northbound traffic on the section of the street north of Massachusetts Avenue.

Northbound volumes in lower Georgetown during the afternoon peak period increased from an average of 561 vph in the before study to an average of 822 vph in the after study, or an increase of about 47 percent. Increases in the southbound direction amounted to 26 percent during the same period. During the morning peak hour in this section increases of 21 percent southbound and 15 percent northbound were recorded. In the heavy direction of flow, southbound in the morning, the increase was considerably less than the 50 percent predicted in phase 1 in the Wisconsin Avenue Report. Nevertheless, it is evident that, considering the short period of time elapsing since the improvements were effected and the fact that all of the improvements included in phase 1 had not yet been undertaken, the results are impressive.

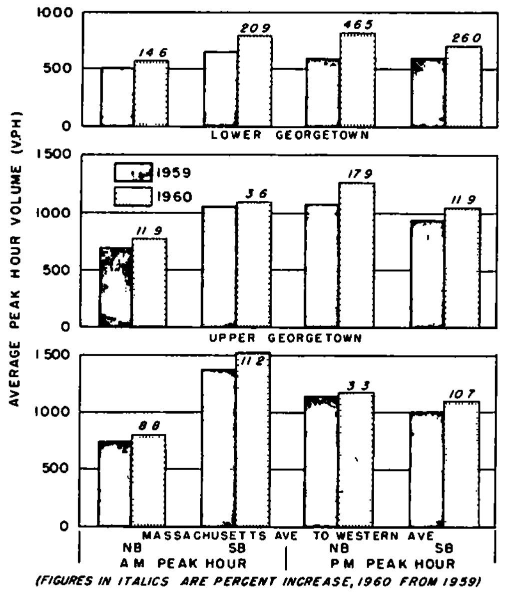

Figure 5. Average peak-hour traffic volumes, by section, at intersections on Wisconsin Avenue studied both in 1959 and 1960.

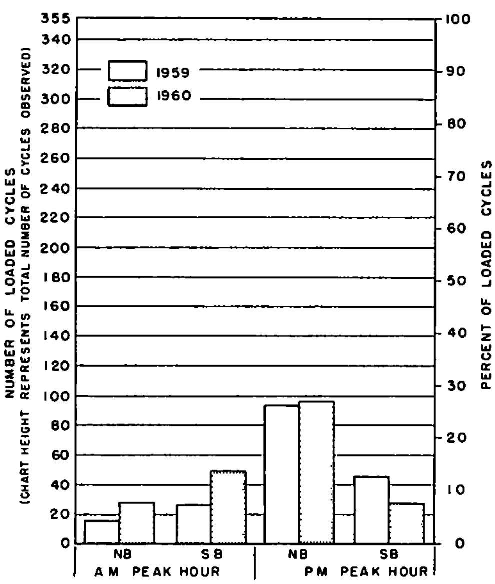

Figure 6. Number of loaded cycles at eight intersections on Wisconsin Avenue, (K Street to Western Avenue), studied both in 1959 and 1960.

Volumes in upper Georgetown, northbound in the afternoon, increased from an average of 1,075 vph in the before study to an average of 1,267 in the after study. This is an increase of about 18 percent. However, the morning peak showed an increase of only 4 percent in the southbound, heavy flow direction, from 1,062 vph to 1,100 vph. Traffic which formerly turned left from the east leg of Calvert Street into the southbound traffic flow on Wisconsin Avenue now has been diverted from the entire area. Most of this traffic turned left into Observatory Circle from westbound traffic on Massachusetts Avenue. These left turns at Observatory Circle are no longer allowed; thus a relatively large southbound traffic volume has been removed from Wisconsin Avenue in this area. It would be purely a matter of conjecture to try to define the new route of this traffic. However, a study of the available traffic data and of the geographical nature of the area in question indicates that the traffic which formerly entered this section via Observatory Circle and Calvert Street now does not enter Wisconsin Avenue anywhere in this neighborhood.

In the section north of Massachusetts Avenue, the volume southbound in the morning peak increased from 1,373 vph during the before study to 1,526 vph during the after study, or an increase of 11 percent. The increase in volume northbound in the afternoon amounted to 3 percent.

Observations of the traffic on Wisconsin Avenue indicate that the remaining streetcar tracks affect the capacity of the street to a greater degree than was anticipated, due, at least in part, to the absence of lane markings. It was noted that most drivers will not drive in the area containing the tracts if they can possibly avoid it. It is estimated that removal or covering of the car tracks combined with provision of lane markings will further increase the volumes by 10 to 15 percent.

Loaded Signal Cycles

Table 3 gives the signal cycle data, and Tables 4 and 5 the loaded cycle conditions, at the eight intersections along the entire length of Wisconsin Avenue which were studied, both in 1959 and 1960. Average before and after loading conditions at these intersections are shown in Figure 6. The total number of cycles studied, including all of the selected intersections, is 355 as indicated by the left-hand scale of the figure. Subsequent Figures 7 through 9 show loading conditions by section and by intersection, the base number of cycles studied being similarly shown by the left-hand scale. The right-hand scale in these four figures shows the percentage of loaded cycles.

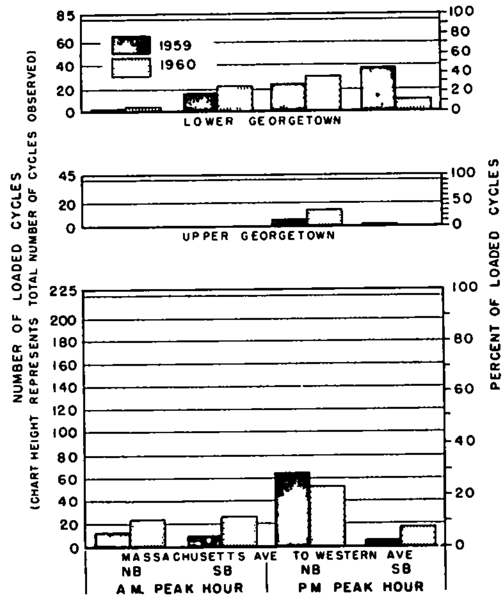

Figure 7. Number of loaded cycles by section, direction, and time period, at intersections studied both in 1959 and 1960.

For all practical purposes there was no change in average loading in the northbound traffic flow in the afternoon. Loaded cycle time increased by 4 percentage

TABLE 3

SIGNAL CYCLE DATA FOR PEAK PERIODS, AT INTERSECTIONS ON WISCONSIN AVENUE STUDIED BOTH YEARS

Intersecting Street	Cycle Length, Sec., (Same in Both A.M.&P M.)		Percent Green Time on Approach							
			N.B., A.M.		S.B., A.M.		N.B., P.M.		S B., P.M	
	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960
M St.	90	90	23.3	23.3	23.3	23.3	23.3	23 6	23.3	24.6
G St.	80	80	43.8	43.8	43.8	43.8	43.8	43.8	43.8	43.8
Calvert St.	80	80	42.5	58.8	37.5	48.8	42.5	56.2	37.5	38.8
Garfield St.[a]	—	80	—	22.5	—	70.0 T. 45.0 L.	—	40.0	—	72 5 T. 32.5 L.
Mass. Ave.	80	80	18.8	42.5	48.8 T. 30.0 L.	42.5	27.5	36.2	51.3 T. 23.8 L.	36.2
Van Ness St.	80	80	63.8	63.8	63.8	63.8	63.8	63.8	63.8	63.8
Tenley, So.	80	80	52.5	57.5	52.5	57.5	52.5	57 5	52.5	57.5
Tenley, No.	80	80	52.5	57.5	52.5	57.5	52.5	57 5	52.5	57.5
Albemarle St.	80	80	57.5	57.5	57.5	57.5	57.5	57 5	57.5	57.5
Western Ave.	80	80	40.0	37.5	57 5 T.&R. 12.5 L.	57.5 T.&R. 12.5 L.	40.0	57.5	57 5 T.&R. 12.5 L.	57.5 T.&R. 12.5 L.

[a] Garfield Street was not signalized in 1959. It is included in this table because it was an integral part of the Massachusetts-Wisconsin intersection "complex" in 1960.

points, northbound in the morning. A 7 percentage point increase in loaded cycle time was noted for the southbound traffic flow in the morning and a decrease of about 5 percentage points for the same direction in the afternoon.

TABLE 4

COMPARISON OF LOADED SIGNAL CYCLE CONDITIONS ON WISCONSIN AVENUE BY SECTIONS, 1959-1960

Section of Street	Total No of Cycles Studied in Peak Hour (same both yrs.)	No of Loaded Cycles 1959	No of Loaded Cycles 1960	Difference in No. of Loaded Cycles	Percent Loaded Cycles 1959	Percent Loaded Cycles 1960	Difference in Percent Loaded Cycle Time	No of Loaded Cycles 1959	No of Loaded Cycles 1960	Difference in No. of Loaded Cycles	Percent Loaded Cycles 1959	Percent Loaded Cycles 1960	Difference in Percent Loaded Cycle Time
(a) A M													
		Northbound						Southbound					
Lower Georgetown	85	2	4	+2	2.3	4.7	+2.4	16	22	+6	18.8	25.8	+7.0
Upper Georgetown	45	0	0	0	0	0	0	0	0	0	0	0	0
North of Mass	225	13	24	+11	5.8	10.7	+4.9	10	27	+17	4.4	12.0	+7.6
Over-all	355	15	28	+13	4.2	7.9	+3.7	26	49	+23	7.3	13.8	+6.5
(b) P M													
		Northbound						Southbound					
Lower Georgetown	85	24	30	+6	28.2	35.2	+7.0	37	10	-27	43.5	11.7	-31.8
Upper Georgetown	45	5	13	+8	11.1	28.9	+17.8	2	0	-2	4.4	0	-4.4
North of Mass.	225	65	53	-12	28.9	23.6	-5.3	6	17	+11	2.7	7.6	+4.9
Over-all	355	94	96	+2	26.4	27.0	-0.6	45	27	-18	12.7	7.6	-5.1
(c) A.M. and P.M. Combined													
Entire Street[a]	1,420	180	200	+20	12.7	14.1	+1.4	-	-	-	-	-	-

[a] Northbound and Southbound

When each section is examined individually, however, sizeable changes in percent of loaded cycle time become evident, ranging from a decrease of 32 percentage points for the southbound traffic flow in the afternoon in lower Georgetown to an increase of 18 percentage points for the afternoon peak traffic northbound in upper Georgetown. Figure 7 compares the number of loaded cycles for each of the three sections for the before and after studies.

Table 4, which summarizes the differences in percent of loaded time by seconds, shows that in lower Georgetown there have been 7 percentage point increases in loaded cycle time in the direction of heavy flow during both the A. M. and P. M. peaks, from 19 percent to 26 percent in the morning and from 28 percent to 35 percent in the afternoon. This increase in loading indicates that intersections are governing capacity to a

TABLE 5

LOADED CYCLE CONDITIONS FOR PEAK PERIODS, AT INTERSECTIONS ON WISCONSIN AVENUE STUDIED BOTH YEARS

Intersecting Street	Total No. of Cycles (Same in A.M. & P.M.) 1959	Total No. of Cycles (Same in A.M. & P.M.) 1960	N.B., A.M. 1959	N.B., A.M. 1960	S.B., A.M. 1959	S.B., A.M. 1960	N.B., P.M. 1959	N.B., P.M. 1960	S.B., P.M. 1959	S.B., P.M. 1960
			No. of Loaded Cycles on Approach							
M St.	40	40	0	1	13	3	0	26	17	1
Q St.	45	45	2	3	3	19	24	4	20	9
Calvert St.	45	45	0	0	0	0	5	13	2	0
Garfield St.[a]	—	45	—	0	—	6	—	0	—	5
Mass. Ave.	45	45	7	0	0	0	15	0	5	11
Van Ness St.	45	45	6	9	5	6	42	12	1	0
Tenley, So.	45	45	0	0	—	—	8	0	—	—
Tenley, No.	45	45	—	—	5	17	—	—	0	6
Albemarle St.	45	45	0	15	0	1	0	32	0	0
Western Ave.	45	45	0	0	0	3	0	9	0	0

[a] Garfield Street was not signalized in 1959. It is included in this table because it was an integral part of the Massachusetts-Wisconsin intersection "complex" in 1960.

greater degree than before. In upper Georgetown the only significant change is an increase of 18 percentage points northbound in the afternoon.

North of Massachusetts Avenue changes in loaded cycle time ranged from a decrease of 5 percentage points northbound in the afternoon to an increase of 8 percentage points southbound in the morning. This indicates an increase in volumes in the morning greater than proportional to the increase in capacity developed by the traffic engineering improvements. Conversely the decrease in loading during the afternoon peak period indicates that the present increase in capacity is not being fully used for this period.

When each intersection is examined individually, as shown in Table 5 and in Figures 8 and 9, a greater range of change is apparent. The largest increase in percent loaded cycle time occurred northbound in the afternoon at Albemarle Street (Fig. 9). In 1959 at this point, there was no loaded cycle time; in 1960 the intersection was loaded at this point, 71 percent of the time. The largest decrease in loading occurred at Van Ness Street, also in the northbound afternoon peak period. In 1959, loading occurred during 93 percent of the peak hour; in 1960, loaded time dropped to 25 percent of the peak hour.

In the original study Van Ness Street was pointed to as a prime example of an intersection becoming a bottleneck. In the period of time between the original study and the after study, improvements were made to alleviate this condition, mainly parking restrictions. The after study shows a large reduction in loaded cycle time and a change in a portion of the remaining loaded cycle time from the "sustained" type of loading to the "intermittent" type of loading. Thus, it can be seen that this intersection is no longer operating at or close to its possible capacity as it was originally; it now is operating near its new practical capacity.

The after study shows that in the northbound direction Albemarle Street is assuming the characteristics of a bottleneck although not to the extent that Van Ness Street was originally. Q Street and Tenley Circle appear to be potential trouble spots, southbound.

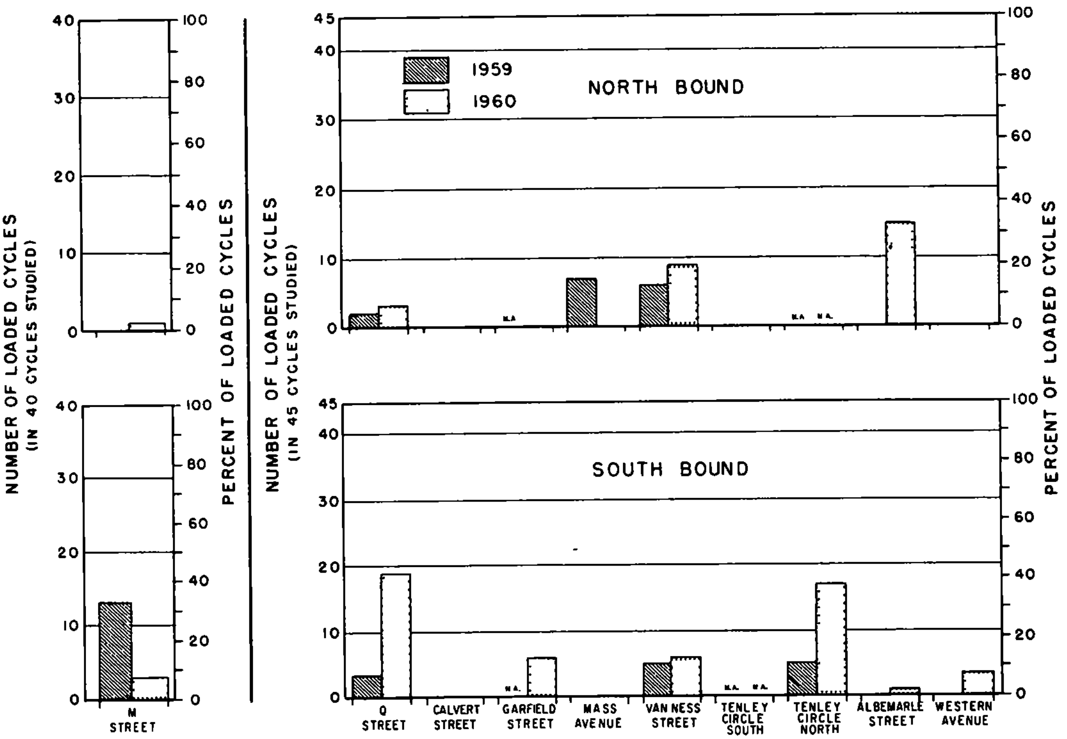

Figure 8. Number of loaded cycles at those intersections studied both in 1959 and 1960-A.M. peak.

Further examination of Figures 6 to 9, and especially of Figures 8 and 9, shows that generally an equalizing influence has been effected on Wisconsin Avenue. Those intersections which showed little or no loading in the original study, generally show a small increase in loading in the after study, indicating an increase in the capacity of the mid-block sections. Conversely, intersection improvements at Van Ness removed a bottleneck at that point.

The development of increased loading northbound in the afternoon on Wisconsin Avenue at Albemarle Street is not surprising, because improved operation at Van Ness Street and at Tenley Circle permits more traffic to reach this point. Neither is the Q Street problem unexpected. Such transfers of problem points were predicted in the original report where it was pointed out that there will be a continuous need for study and correction of new spot bottlenecks as the improvement program is carried out.

Speeds

Average Speeds.—In the after study, average speeds ranged from 18 to 21 mph when classified by direction and time periods for the entire Wisconsin Avenue test section. The average speed of all runs showed a gain of 8 percent, from 17.8 mph in the before study to 19.2 mph in the after study. As shown in Figure 10, there was an over-all increase in speed for all the periods studied except for the northbound traffic in the morning. The northbound A.M. off-peak shows no change in speed since last year; the northbound morning peak period shows an over-all decrease in speed of about 1 mph. Table 6 compares the average speeds for the before study and after study by section, direction, and time.

Figure 11 compares average speeds, by sections. In lower Georgetown the over-all average speed increased 8 percent. It increased about 1½ mph, from 13 mph to about 14½ mph, in the southbound morning peak hour. The largest change in speed in this section was a 2½-mph increase in the southbound morning off-peak hours. The only decrease in average speed in this section occurred in the northbound morning off-peak hour, and amounted to 1 mph. The northbound peak traffic in the afternoon averaged only 0.2 mph faster than in the before study, 12.3 mph, an insignificant increase.

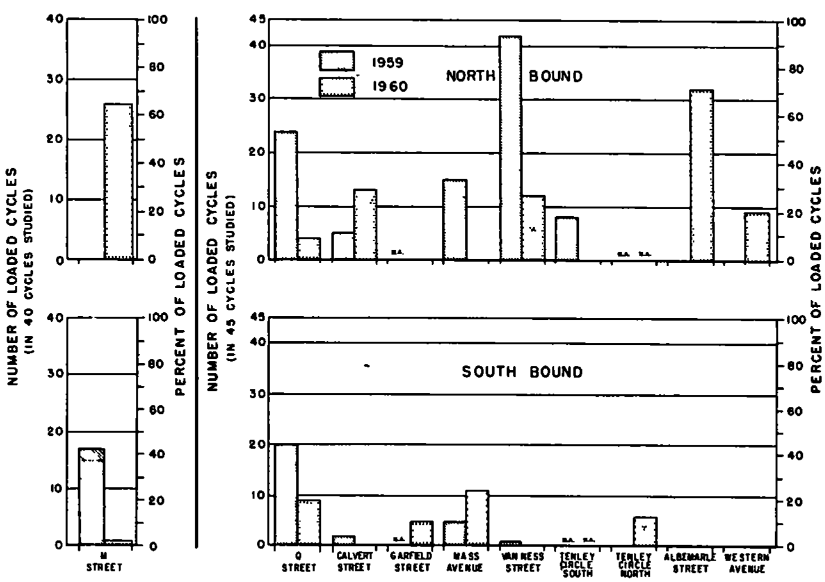

Figure 9. Number of loaded cycles at those intersections studied both in 1959 and 1960-P.M. peak.

Increases in average speed in upper Georgetown ranged from 0.5 mph to 6 mph. The over-all average speed gain was 11 percent. Southbound in the morning peak the speed changed from an average of 21 mph during the before study to 23 mph in the after study. The increase in speed northbound in the afternoon peak amounted to about 4 mph, from 15 mph to 19 mph.

At the time of the after study, average speeds in the section north of Massachusetts Avenue ranged from 18 to 23 mph, registering an over-all average speed gain of 7 percent. Both peak hour heavy traffic flows in this section showed speed increases of

TABLE 6

COMPARISON OF AVERAGE SPEEDS ON WISCONSIN AVENUE, 1959-1960

	Average speeds														
	A.M. Peak			A.M. Off-Peak			P.M. Off-Peak			P.M. Peak			All Time Periods, Combined		
Location	1959	1960	% Change	1959	1960	% Change	1959	1960	% Change	1959	1960	% Change	1959	1960	% Change
(a) Northbound															
Lower Georgetown	14.4	15.0	+4.2	13.0	11.7	-10.0	11.9	12.9	+8.4	12.3	12.5	+1.6	13.1	13.5	+3.1
Upper Georgetown	19.0	19.4	+2.1	18.1	19.7	+8.8	16.4	19.2	+17.1	15.1	18.9	+25.2	17.3	19.2	+11.0
Mass.-Western Ave.	23.0	21.4	-7.0	20.8	20.8	0	18.5	18.8	+1.6	16.1	19.8	+23.0	19.9	20.5	+3.0
Over-all	20.4	19.4	-4.9	18.2	18.2	0	16.8	17.6	+4.8	15.1	17.8	+17.9	19.9	18.5	+3.3
(b) Southbound															
Lower Georgetown	13.0	14.3	+10.0	11.0	13.3	+20.9	10.7	11.3	+5.6	11.3	12.9	+14.2	11.7	13.2	+12.8
Upper Georgetown	21.1	23.0	+9.0	19.3	25.1	+30.1	19.6	22.9	+16.8	19.0	20.4	+7.4	19.9	22.3	+12.1
Mass.-Western Ave.	19.2	22.9	+19.3	19.7	22.4	+13.7	20.9	20.1	-3.8	18.3	21.0	+14.8	19.5	21.7	+11.3
Over-all	18.6	21.2	+14.0	17.5	20.2	+15.4	17.9	18.5	+3.4	17.0	19.1	+12.4	17.8	19.9	+11.8
(c) Average Speed															
Over-all street[a]	-	-	-	-	-	-	-	-	-	-	-	-	17.8	19.2	+7.8

[a]Both directions combined.

about 4 mph, and most of the remaining time periods in both directions showed increases in speed. However, the speeds in this section were decreased 1½ mph for the northbound traffic during the morning peak period. This probably represents the slight adverse effect of the new progression, favoring peak direction traffic, on the opposing flows.

Running Speeds.—The changes in running speeds consistently amounted to two-thirds of the corresponding changes in average speeds in all cases. Running speeds increased from an average of 21.8 mph in the before study to 22.8 mph in the after study.

Stops

Data concerning the number of stops on the street are presented in Table 7. For Wisconsin Avenue as a whole in the before study, as shown in Figure 12, the average number of stops per run ranged from 13

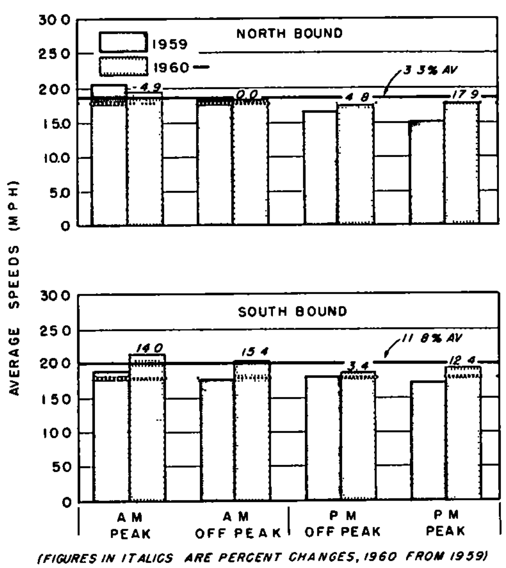

Figure 10. Comparison of average speeds on Wisconsin Avenue.

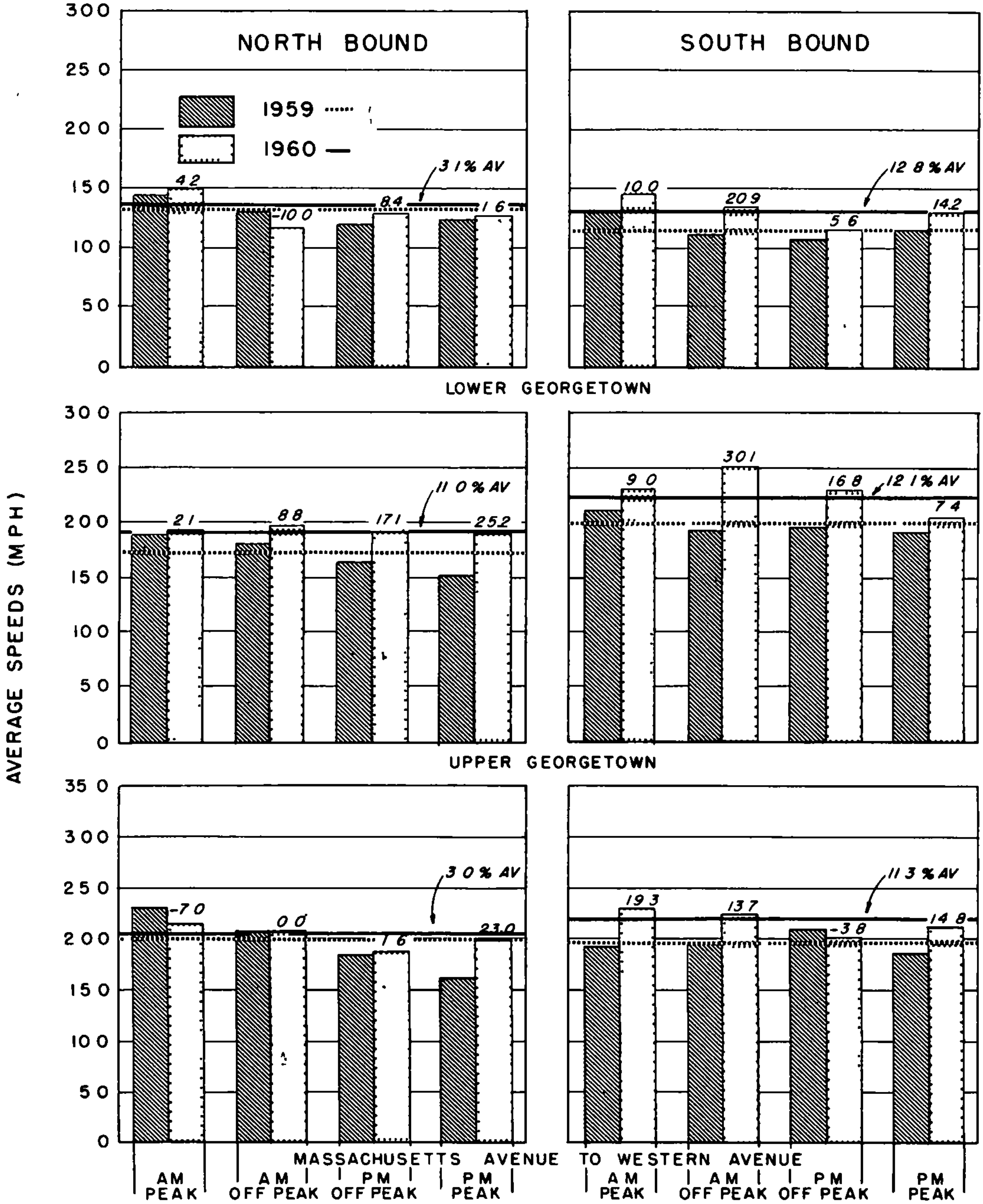

Figure 11. Comparison of average speeds by section and time of day.

to 25 depending on the time of day. In the after study the average number of stops per run ranged from 4 to 8. The average number of stops per run for all periods combined was reduced by 65 percent. During peak periods the reduction has been even greater, amounting to 76 percent southbound in the morning and 70 percent northbound in the afternoon.

TABLE 7

COMPARISON OF AVERAGE NUMBER OF STOPS PER RUN ON WISCONSIN AVENUE, 1959-1960

Location	Average No. of Stops per Run														
	A.M. Peak			A.M. Off-Peak			P.M. Off-Peak			P.M. Peak			All Time Periods, Combined		
	1959	1960	% Change	1959	1960	% Change	1959	1950	% Change	1959	1960	% Change	1959	1960	% Change
(a) Northbound															
Lower Georgetown	2.7	0.6	-77.8	2.2	1.4	-36.4	2.8	0.8	-71.4	3.2	1.0	-68.8	2.7	0.9	-66.7
Upper Georgetown	3.9	1.7	-56.4	3.8	1.4	-63.2	4.9	1.5	-69.4	6.5	1.3	-80.0	4.7	1.1	-76.6
Mass.-Western Ave.	3.9	3.3	-15.4	5.4	3.8	-29.6	9.5	4.3	-54.7	13.1	4.1	-68.7	7.6	3.8	-50.0
Total	12.5	6.5	-48.0	14.5	7.8	-46.2	18.5	8.2	-50.3	24.5	7.3	-70.2	17.2	7.1	-59.1
(b) Southbound															
Lower Georgetown	4.5	1.6	-64.4	7.3	2.0	-72.6	7.4	2.4	-67.6	5.5	1.6	-70.9	6.0	1.8	-70.0
Upper Georgetown	4.2	0.6	-85.7	4.9	0.3	-93.9	4.9	1.0	-79.6	4.7	1.7	-63.8	4.6	1.0	-78.3
Mass.-Western Ave.	9.4	2.0	-78.7	9.0	2.0	-77.8	6.4	4.0	-37.5	10.2	3.2	-68.6	8.8	2.7	-69.3
Total	18.5	4.4	-76.2	21.0	5.0	-76.2	19.0	7.6	-60.0	20.5	6.8	-66.8	19.6	5.9	-69.9
(c) Average No. of Stops per Run															
Over-all street[a]	-	-	-	-	-	-	-	-	-	-	-	-	18.4	6.5	-64.7

[a] Both directions combined.

Data for the individual sections are shown in Figure 13. The average number of stops per run for lower Georgetown has decreased 75 percent since the before study. Then, stops per run ranged from two to seven. In the after study the range was one to two stops per run. The number of stops decreased 65 percent southbound in the morning and 69 percent northbound in the afternoon.

The average number of stops per run in upper Georgetown, which ranged from four to seven in the before study, decreased to a range of zero to two in the after study. This over-all decrease amounts to 75 percent. The number of stops per run has decreased by 80 percent for the northbound afternoon peak period traffic flow and 86 percent for the southbound morning peak period flow.

For the section north of Massachusetts Avenue, the average number of stops per run in the before study ranged from 4 to 13; in the after study the average number of stops per run ranged from two to four. This is an over-all decrease of 60 percent. The peak period heavy traffic flows showed decreases of 69 percent northbound and 79 percent southbound.

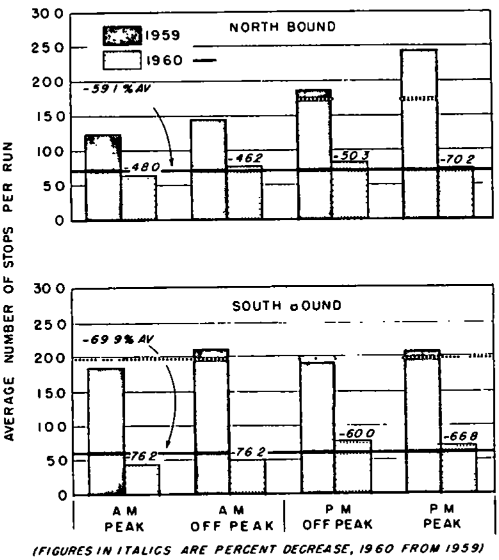

Figure 12. Average number of stops by time of day—K Street to Western Avenue (deceleration to 2 mph or below was identified as a stop).

Significance of Speed Data

The over-all increase in average speed can be attributed to the smoother traffic flow provided by the new traffic signal progression and to reduction in midblock friction. Reduction in delay time at signalized intersections contributed approximately 40 percent of the northbound average speed increase and 10 percent of the southbound average speed increase. Reduction in midblock friction helped increase both average and running speeds. The marked decrease in all kinds of stops reduced the time lost in deceleration and acceleration with resultant benefits to average speed and running speed.

Capacity

The over-all increases in volume indicate that, on the whole, the capacity of Wisconsin Avenue has been improved. Development of slight loading at several intersections which previously showed none indicate that the volume at those intersections has increased to the point where efficiently full use is being obtained; that is, practical capa-

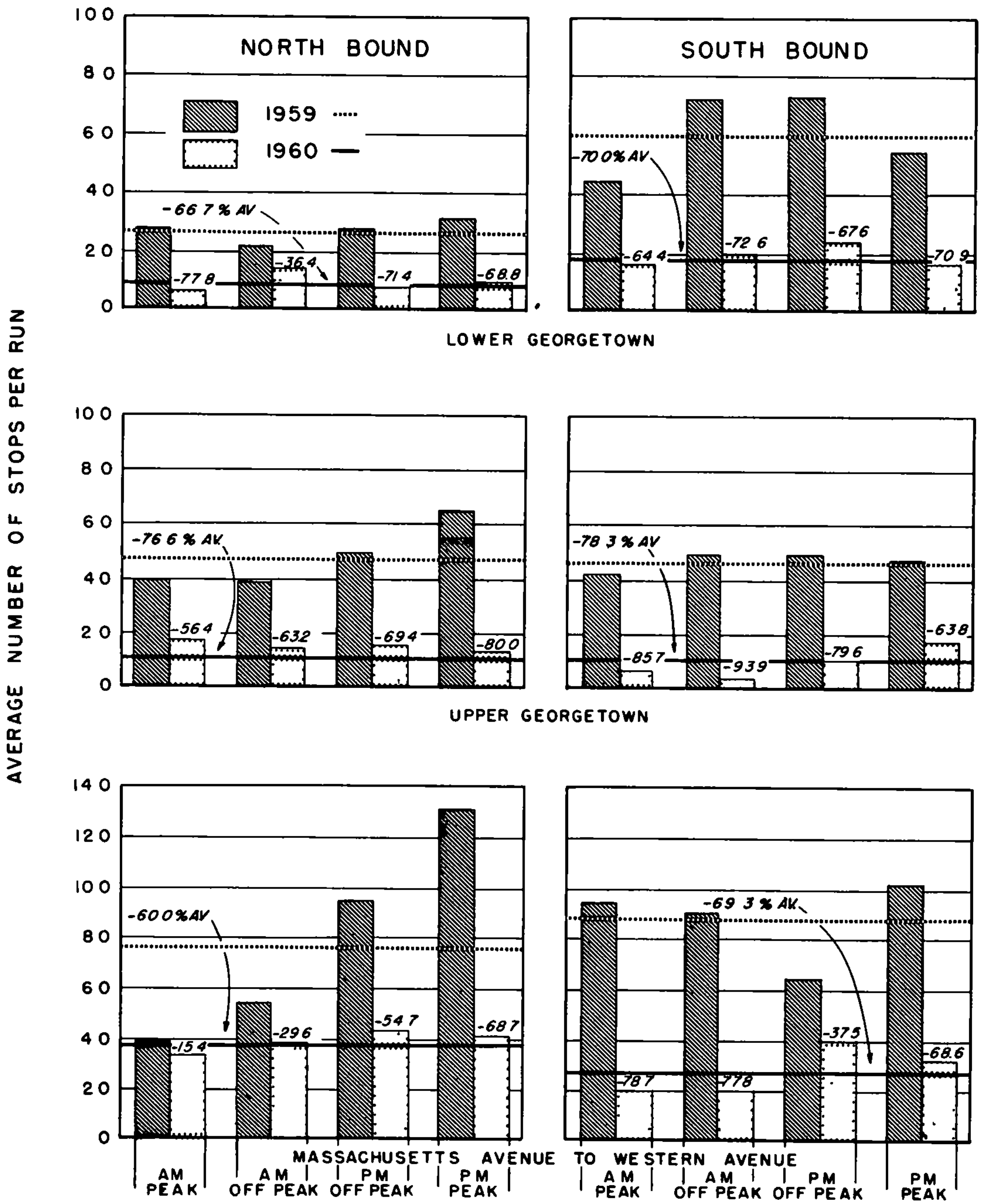

Figure 13. Average number of stops by section and time of day (deceleration to 2 mph or below was identified as a stop).

city has been attained. Conversely, decreases in loading at other intersections indicate that their capacity has been improved to such an extent that they no longer are being as fully used. Over-all increases in speed and the large reductions in the average number of stops per run also indicate a substantial improvement in the traffic carrying capability of the street.

It is interesting to note that the foot of Wisconsin Avenue between K and M Streets, a steep grade which in the before study had the characteristics of a local feeder street, seems to be in the process of assuming the urban arterial characteristics of the remainder of Wisconsin Avenue. This is borne out by a 76 percent increase in volume and a 65 percent increase in loading northbound during the afternoon peak hour. The volume between K and M was 319 vph in the before study and 563 vph in the after study. Thus, although this section is not yet operating at practical capacity it is rapidly approaching that condition and could prove to be troublesome in the future if the present trend continues. This situation has not yet appreciably affected the general traffic conditions for lower Georgetown as a whole. It is, nevertheless, an example of unforeseen changes in the habits of the driving public which frequently occur, disrupting carefully-thought plans.

The fairly large increases in volumes in the heavy flows during peak hours, the increase in speed during the southbound peak period, and the large decrease in the number of stops are all indications that the capacity of the lower Georgetown section has been greatly improved, probably primarily through elimination of midblock friction produced by the streetcar transit operation.

For the upper Georgetown section it was noted that for the northbound traffic flow in the afternoon peak period the volume had increased 25 percent, the over-all speed had increased 4 mph, and the number of stops per run had decreased by 80 percent. These figures illustrate the effect that removal of the streetcars and loading platforms, the new parking regulations, and the changes in signal timing have had in greatly increasing the capacity of this section. Undoubtedly, the street is currently capable of carrying even greater volumes. The volumes will probably continue to increase as more drivers discover the improved conditions on the street.

The changes in the traffic engineering features north of Massachusetts Avenue were the same as the changes south of that street, except that the new parking regulations were in effect only at the Van Ness Street and Western Avenue intersections. It was previously mentioned that up to a 67 percent decrease in loaded cycle time occurred at the Van Ness Street intersection where the signs pertaining to the new "no parking" and "no standing" regulations had been installed. The substantial increase in the capacity of this intersection which this reduction represents has not yet been matched by the section as a whole. In fact, it cannot be expected that when the new parking regulation signs are finally erected throughout this section, the capacity of this section will be increased proportionately because Van Ness was previously a known "bottleneck." However, it is believed that substantial increases for this section as a whole can be realized.

Generally, however, the increases in volumes, combined with fairly large increases in speed for the heavy flow directions during the peak periods, the large decreases in the number of stops, and the reduction in loading in the northbound afternoon peak hour traffic flow seem to indicate an increased capacity which is not being fully used. This being the case, the practical capacity under prevailing conditions will not be reached until further volume increases occur.

SUMMARY OF RESULTS

The following results are evident from the comparison of the data obtained in the before and after studies:

1. Over-all peak hour volumes for Wisconsin as a whole increased by an average of 11 percent. The increases in lower Georgetown amounted to 27 percent, in upper Georgetown 11 percent, and north of Massachusetts Avenue 9 percent.
2. For Wisconsin Avenue as a whole and for the three sections as well, the trend was toward a small increase in percent loaded cycle time.

3. Average speed for all runs increased from 17.8 mph to 19.2 mph. The largest peak-hour increases in speed, amounting to 4 mph, occurred north of Massachusetts Avenue in the heavy flow directions. Southbound in the morning the speed increased from 19 to 23 mph, and northbound in the evening the increase was from 16 to 20 mph. In general, lesser increases were noted at other times, and in Georgetown at all times. However, soutbound in the A.M. off-peak in upper Georgetown, a 6-mph increase occurred.

4. Changes in average running speeds consistently averaged two thirds of the corresponding changes in average speeds. Running speeds increased from an average of 21.8 mph to 22.8 mph.

5. For Wisconsin Avenue as a whole, the number of stops per run has been reduced on the average by 65 percent. In both lower and upper Georgetown the average number of stops per run for all periods combined has been reduced by 75 percent. North of Massachusetts Avenue the reduction in the average number of stops per run for off-peak and peak periods combined amounted to 60 percent. Generally the reduction in the average number of stops per run was greater in the heavy flow directions during the peak periods.

6. The concurrent increases in speed and volume coupled with the decrease in number of stops indicate that the capacity of Wisconsin Avenue as a whole has been substantially increased. The increases in capacity vary within the three sections studied. Use of the increased capacity also varies within the three sections, but this use was greatest in lower Georgetown.

7. In upper Georgetown and north of Massachusetts Avenue the street is currently capable of carrying greater volumes. Further increases in volumes must occur before these sections reach their capacity under prevailing conditions.

REFERENCE

1. "Increasing Traffic Capacity of Arterial Streets." HRB Bull. 271 (1960).

A New Technique for Predicting Vehicle Operating Cost

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This paper discusses the development of a new approach to the prediction of the highway user costs associated with different highway alignments. This approach is built around a digital computer program which simulates the physical operation of a sample vehicle or vehicles. Although intended to produce the same sort of information as the AASHO Road User Benefit Analysis Manual, this new technique will permit a far more detailed analysis of alternatives in highway design.

● THE basic objective underlying the development of this new approach to the vehicle operating cost problem was to provide the highway design engineer with a capability commensurate in its sophistication with the capability he already possesses for estimating construction and other such highway costs. This involved finding some easily applied technique for determining the effect of relatively minor changes in alignment and grade on operating cost. It became apparent at an early stage in the research that the only practicable way for the design engineer to do this would be for him to employ an electronic computer to analyze design alternatives.

Accordingly, an experimental program was developed for an IBM 650 EDPM which would determine the effect of design changes on the performance of a vehicle by simulating its operation over the alignments in question. This initial program was used to test the mathematical expressions which describe motor vehicle operation and to evaluate the over-all feasibility of a computer approach to the vehicle operating cost problem. As a result of this early work the decision was made to go ahead with the project, and a set of two entirely new computer programs was written to replace the original program. These new programs are more sophisticated in their simulation of vehicle operation and much more flexible insofar as their use by the highway design engineer is concerned.

The next step in the research program was to test the predictive ability of the programs by comparing the performance of actual vehicles in the field to computer simulations which assumed identical conditions of alignment and speed and identical vehicle characteristics. Although some data were available from previous field testing performed for other purposes, it was necessary to supplement these with a special series of field tests. These were carried out in September and October of 1960 in the Washington, D. C., area and are reported herein. The results of this test series indicate that with some modifications the computer programs can be expected to give satisfactory results for both the travel time and fuel consumption of any sample vehicle.

This paper is intended principally as a general discussion of the research performed to date and of the computer programs on which this research has focused. It does not treat in detail the question of precisely how the computer programs are to be operated. That is left for a Program Manual now in preparation. Similarly, the paper gives only a brief discussion (Appendix) of the mathematics of the programs. This topic is covered in detail in a somewhat more lengthy research report which is also in preparation.

Finally, the paper does not attempt a complete discussion of the way in which the results of the programs would be applied to a highway location or design problem. This phase of the research effort is not yet complete. The problems raised in this connection will therefore be touched on here only in enough detail to make clear the nature of the analyses which the computer programs will make possible.

DESCRIPTION OF THE PROGRAMS

The basic set of computer programs which has been developed consists of a Vehicle Parameter Computation Program and a Vehicle (Operation) Simulation Program (designated EA1-T1 and EA1-T2, respectively, in the DTM Program Series). A third program designed to plot the simulation output graphically is also available, but it is of no particular interest here.

Parameter Computation Program

The Parameter Computation Program converts the basic data which describe the vehicle into a set of summary parameters for use by the Vehicle Simulation Program. The engineer uses as input to the Parameter Computation Program such things as vehicle weight, frontal area, and tire size, engine bore, stroke, displacement, and number of cylinders, the full throttle torque curve, and transmission and rear axle gear ratios.

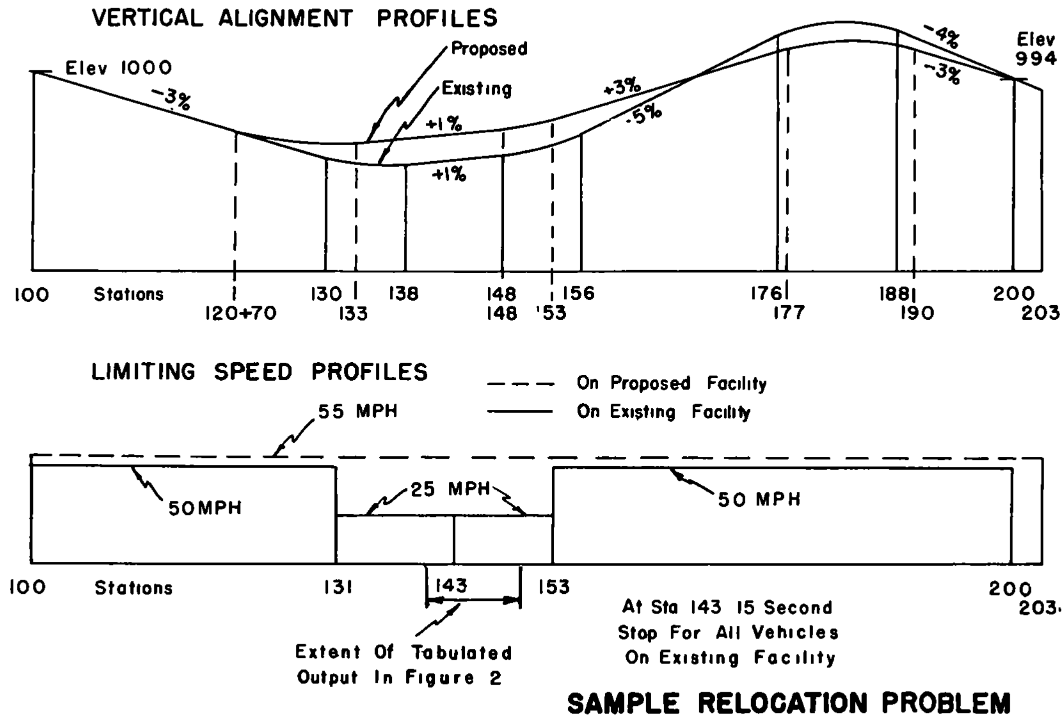

Figure 1. Vertical alignment and speed profile input to the Vehicle Simulation Program.

These are all readily available manufacturer's data. The engineer must also prepare some other information which will be supplied him in the Program Manual. This includes such things as air and rolling resistance coefficients, brake specific fuel consumption as a function of piston speed, and engine mass equivalent constants.

Using this information the program performs the following sort of operations: consolidates all the tractive resistance coefficients, computes the coefficients of the engine torque curve (by the method of least squares), prepares a table of fuel rates in terms of engine rpm and brake horsepower, and computes a table of vehicle vs engine speed parameters for each year. Using a computer for this task permits a more sophisticated analysis of the basic data than the highway design engineer would normally undertake. More importantly, it makes it possible to set up the basic programs so they can use the vehicle manufacturer's regularly published data as input to describe vehicle characteristics. This removes what would otherwise be a major problem (if not a major delay) for the engineer who wants to use the over-all analysis technique on only an occasional basis, and who does not therefore have the special knowledge required to derive summary parameters in the proper form.

The Parameter Computation Program need be used only when a new class or type of vehicle is to be simulated. This contrasts with the Simulation Program which would be run one or more times for each class of vehicle for every one of the alternatives in what might be a large number of different design problems.

Vehicle Simulation Program

The Vehicle Simulation Program itself computes travel time and fuel consumption as a function of vehicle characteristics, highway alignment, and driving speed restrictions. It does this by predicting the vehicle's motion from the basic laws of physics and calculating fuel consumption on the basis of the power thus required from the engine. As the computer follows the vehicle's motion along the alignment, it punches out the resulting information at regular increments of time as specified by the engineer.

The program as presently coded does not compute any other vehicle operating results such as oil consumption, tire wear, or maintenance costs. Too little is known about the relationships between these costs and the physics of vehicle operation to warrant their computation on anything other than a simple distance-traveled basis. Computation on that basis is done more readily by hand than with a computer. It will be a simple matter, however, to add these capabilities to the program whenever the necessary mathemetical relationships have been developed.

The program works with five sets of input information. The first of these describes the horizontal alignment of the highway—the PC, PT, radius, and superelevation of each curve. The second set describes the vertical alignment of the highway—the VPC and VPT of each vertical curve and the grades which connect these curves. The third set of input data describes the maximum speeds at which the vehicle is to be operated on different sections of the alignment under study. It also lists the stationing of any stops which the vehicle is to make and length of time it is to wait at each stop. Finally, it specifies the maximum acceleration and deceleration rates at which the vehicle is to be operated. The fourth set of input describes the mechanical and physical characteristics of the vehicle. (It is simply the output from the Parameter Computation Program previously described.) The fifth and final set of input is the control information by which the engineer tells the computer where to begin and end the simulation run, at what intervals to compute the vehicle's performance, and how often to punch out its answers.

The driving speed specifications mentioned as the third set of input data warrant further explanation. These are not intended simply to be either posted speed limits or engineering design speeds. They are intended rather to reflect the preference of the average vehicle operator. As such they must be specified by the engineer. (If he wants, an engineer can even use these restrictions to reflect a certain amount of traffic interference.) If a vehicle is physically incapable of reaching whatever speed limit the engineer has specified, the program will compute only what the vehicle can actually do.

Figure 1 shows how the speed change input might look. The case illustrated shows a slowdown first from 50 mph to 25 mph and then from 25 mph to a stop (at station 143) with a 15-sec wait. From that point the vehicle accelerates (at the specified rate if possible) up to 25 mph and continues at that speed to station 153. At Sta. 153 it begins another acceleration to 50 mph and proceeds at that speed to the final station (Sta. 200). The speed change input in this case consists of the speeds as shown and the stationing of the changes.

Given these and the other types of input data previously described, the program can compute the resulting motion profile and the fuel consumption associated with it. Figure 2 shows the tabulated output of a vehicle simulation run for the sample alignment in

Figure 1. The logic used by the program for such a run is explained subsequently. The mathematics of the computations are outlined in the Appendix.

Vehicle Simulation Logic

To simplify the mathematical relationships the Vehicle Simulation Program uses to describe vehicle dynamics, it was assumed that a vehicle would always be operating under one of the following five conditions: (1) moving at a constant speed, (2) accelerating at a constant rate, (3) decelerating at a constant rate, (4) standing with the engine idling, or (5) moving with the engine at full throttle, and thus at either maximum possi-

EA1 T2

VEHICLE SIMULATION AND OPERATING COST PROGRAM

IDENT	STA FT	VEL MPH	TIME SEC	FUEL POUNDS
318 03 200 00	140+67.097+	25.000+	70.000+	0.11755+
318 03 200 00	141+03.764+	25.000+	71.000+	0.11964+
318 03 200 00	141+40.431+	25.000+	72.000+	0.12173+
318 03 200 00	141+77.098+	25.000+	73.000+	0.12382+
318 03 200 00	142+13.765+	25.000+	74.000+	0.12591+
318 03 200 00	142+46.535+	19.685+	75.000+	0.12662+
318 03 200 00	142+71.509+	14.370+	76.000+	0.12716+
318 03 200 00	142+88.688+	9.055+	77.000+	0.12780+
318 03 200 00	142+98.072+	3.740+	78.000+	0.12828+
318 03 200 00	143+00.000+		79.000+	0.12876+
318 03 200 00	143+00.000+		94.000+	0.14542+
318 03 200 00	143+01.833+	2.500+	95.000+	0.14862+
318 03 200 00	143+07.333+	5.000+	96.000+	0.15183+
318 03 200 00	143+16.499+	7.500+	97.000+	0.15504+
318 03 200 00	143+29.332+	10.000+	98.000+	0.15826+
318 03 200 00	143+45.832+	12.500+	99.000+	0.16149+
318 03 200 00	143+65.998+	15.000+	100.000+	0.16489+
318 03 200 00	143+89.831+	17.500+	101.000+	0.16887+
318 03 200 00	144+17.331+	20.000+	102.000+	0.17349+
318 03 200 00	144+46.107+	19.772+	102.300+	0.17382+
318 03 200 00	144+76.939+	22.272+	103.300+	0.17894+
318 03 200 00	145+11.438+	24.772+	104.300+	0.18430+
318 03 200 00	145+47.937+	25.000+	105.300+	0.18667+
318 03 200 00	145+84.604+	25.000+	106.300+	0.18877+
318 03 200 00	146+21.271+	25.000+	107.300+	0.19087+
318 03 200 00	146+57.938+	25.000+	108.300+	0.19297+
318 03 200 00	146+94.605+	25.000+	109.300+	0.19507+
318 03 200 00	147+31.272+	25.000+	110.300+	0.19717+
318 03 200 00	147+67.939+	25.000+	111.300+	0.19927+
318 03 200 00	148+04.606+	25.000+	112.300+	0.20134+
318 03 200 00	148+41.273+	25.000+	113.300+	0.20348+
318 03 200 00	148+77.940+	25.000+	114.300+	0.20569+
318 03 200 00	149+14.607+	25.000+	115.300+	0.20797+
318 03 200 00	149+51.274+	25.000+	116.300+	0.21032+
318 03 200 00	149+87.941+	25.000+	117.300+	0.21274+
318 03 200 00	150+24.608+	25.000+	118.300+	0.21523+
318 03 200 00	150+61.275+	25.000+	119.300+	0.21779+
318 03 200 00	150+97.942+	25.000+	120.300+	0.22042+

Figure 2. Sample output for a portion of the Sample Relocation Problem.

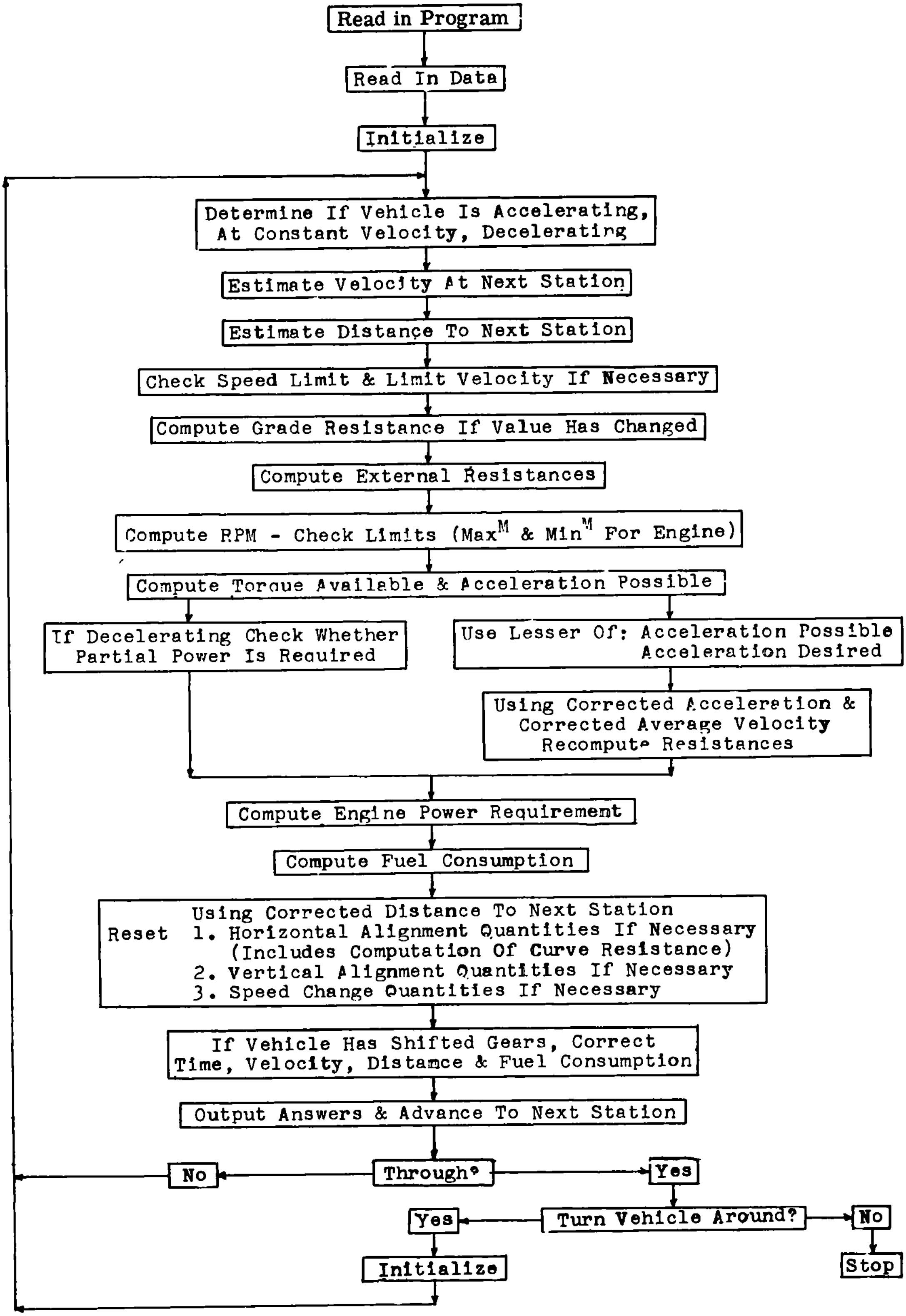

Figure 3. Explanatory flow chart showing the logical and computational steps associated with an acceleration condition.

ble acceleration or maximum sustained speed. Conditions 1 through 4 would be as specified in the input data. Condition 5 would be the result of the vehicle's performance as dictated by the alignment.

The logic of the program is best understood by looking first at the computations required when a vehicle is accelerating (which might produce either Conditions 2 or 5). Figure 3 outlines the logical steps associated with each cycle of computations for the simulation of an accelerating vehicle. The first step of such a cycle computes an estimated average vehicle speed over the next time increment as a function of the performance determined by the immediately previous computation cycle. The estimated distance traveled is computed from the estimated speed over the time increment. The estimated speed is then checked against the limiting speed at the new alignment station and is reduced to the limiting speed if it exceeds it.

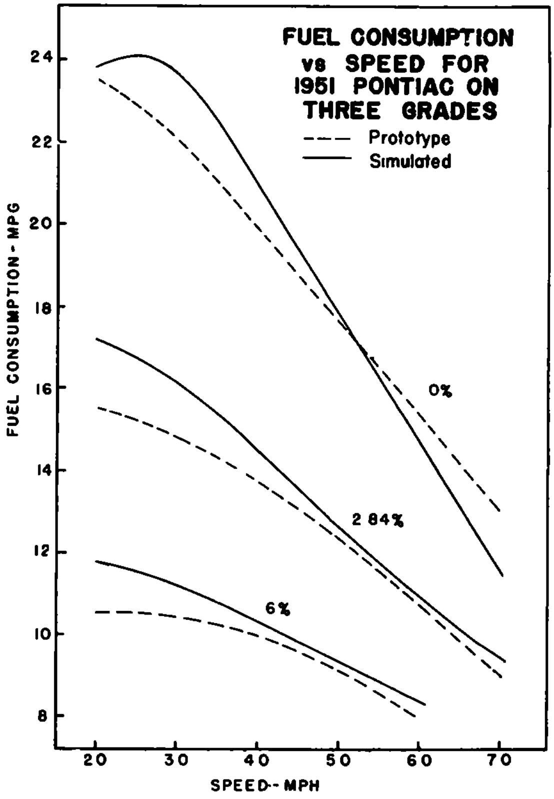

Figure 4. Comparison of simulated and prototype fuel consumptions at constant velocities for the vehicle used in HRB Bulletin 107 (1).

In the next step the program computes the external and internal resistances to vehicle motion on the basis of the average estimated speed over the time increment. Knowing these resistances as well as the maximum torque available from the engine, it computes the possible acceleration. The program then takes as the actual acceleration the lesser of either the possible acceleration or the allowable maximum acceleration. If the resulting actual acceleration is different from the acceleration rate on which the estimated speed was based, the speed, distance, and resistances are corrected. (Because the error converges rapidly on zero, this iterative procedure need be repeated only once.)

Knowing the total tractive resistance and the actual acceleration rate, it is possible to compute the total engine power requirement. The program then takes a fuel consumption rate from a table of values describing fuel consumption as a function of total power requirement and engine rpm. (This table is one of the items prepared by the Parameter Computation Program and used as input to the Vehicle Simulation Program.) The fuel consumption over the time interval is computed from this fuel consumption rate. Finally the program checks the alignment and speed profiles for any changes and resets the necessary quantities for the next computation cycle. If the vehicle exceeds the speed specified in the input as the maximum for the particular gear in which the vehicle is operating, a subroutine computes the performance profile during the gear shift before proceeding to the next computation cycle. This is shown in Figure 2 where the automatic transmission shifts at 20 mph.

In the case where a vehicle is moving at constant speed, the program will omit many of these steps. If the vehicle was moving at the limiting specified speed during the preceding cycle, for example, the program will simply check for any changes in the motion or alignment profiles and if it finds none will simply add an equal increment of distance, time, and fuel to the previous answers.

In the case where a vehicle is decelerating, the computations are also simplified. The program computes in advance the deceleration profile required to bring the vehicle down to the proper speed (which may be a complete stop) at the proper station. When the vehicle reaches a point on that profile, it merely decelerates at the prescribed rate. Fuel consumption is then taken either from the fuel table or from an idling fuel rate, depending on whether or not the engine is still furnishing power.

The program can perform all of these operations for any type and size of highway vehicle and over any type of alignment. Although the storage limitations of the computer restrict the amount of alignment data that can be read in at one time, flexible control features in the program permit getting around this problem. By specifying certain digits in an input control word the engineer can instruct the computer to read in more data when it has finished with the first batch and then to continue the motion of the vehicle where it left off. In the usual case of a road with a common alignment in both directions, the program can turn the vehicle around and run it back over the same alignment in the opposite direction.

TESTING THE PROGRAM

A basic assumption underlying this research effort is that if the Vehicle Simulation Program can demonstrate satisfactory prediction of the performance of a few representative vehicles, it can also satisfactorily predict any other vehicles whose performance would normally be of interest to the highway design engineer. An important part of the research was thus to check actual vehicle performance data against the results of computer simulations for the same vehicles and alignments. Some data collected by other researchers were available in the published literature. In general, however, these data were not sufficiently explicit with regard to either highway alignment or vehicle operating conditions to afford a thorough check on the computer program capabilities. As a result it was necessary to run additional field tests designed specifically to check the Simulation Program.

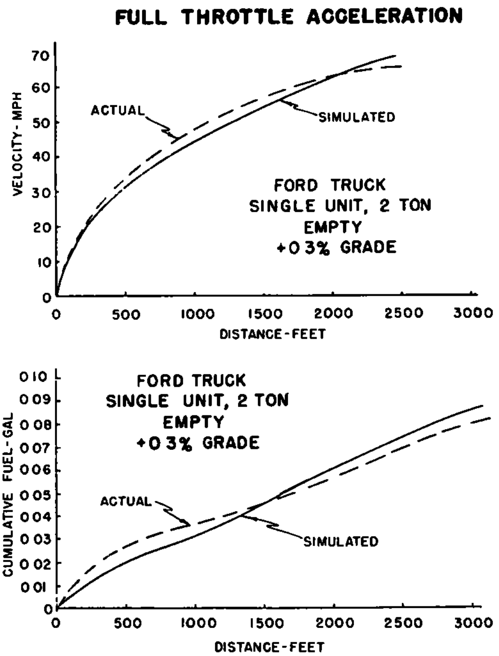

Figure 5. Performance and fuel consumption comparisons of simulated and actual runs for the single-unit test truck under full throttle conditions.

Of those data already available, the most useful were those reported on by Saal in connection with a series of fuel consumption runs made with a 1951 Pontiac sedan (1). In particular, it was possible to check computer predictions of fuel consumption for different grades and speeds against similar data collected by Saal. Figure 4 shows this comparison. In general the simulation results are quite close to the actual fuel consumption (the maximum discrepancies are approximately 7, 10 and 11 percent for the 0, 2.84 and 6 percent grades, respectively). Most of the discrepancy is probably due, moreover, to the fact that the program takes no account of the age, condition or adjustment of the engine.

Description of Field Tests

The field tests run in connection with the present research effort were designed to supplement the very limited sort of data previously discussed. These tests were run

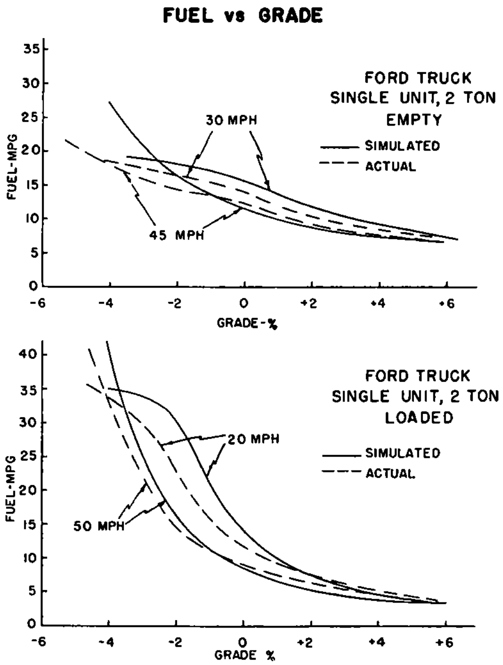

Figure 6. Fuel consumption comparisons of simulated and actual runs for the single-unit test truck empty and loaded as a function of grade.

during September and October of 1960 with the cooperation of personnel from the Division of Traffic Operations in the Office of Research of the Bureau of Public Roads. Three sites in the Washington, D.C., area were used: Dulles International Airport (where the runways, taxiways, and parking aprons had already been paved and so could be used for testing); and the Shirley Freeway in Virginia just south of Washington.

Three different vehicles were used for the tests: a light suburban-type car (not a compact model) with automatic transmission; a 2-ton single-unit truck; and a 50,000-lb GCW tractor-trailer. The two trucks were run in both an empty and a loaded condition.

The test series included four general types of runs: (a) constant speed runs in which the drivers maintained the same speed over an entire test section; (b) acceleration runs either at full throttle or at some constant rate as measured by an accelerometer; (c) deceleration runs where the vehicle was allowed either to coast to a stop or to decelerate at a constant rate; and (d) curve tests in which the vehicles were operated at constant speeds around circles of 150- and 300-ft diameter laid out on a level airport taxiway.

The measurement system employed in these tests included a number of elements. A fifth wheel mounted behind each vehicle measured speed and distance. A fanbelt-mounted tachometer provided data on engine rpm. An adjustable-precision volumetric fuel meter on loan from the Ford Motor Company measured fuel consumption. These units were connected in turn to a set of two digital recording devices provided and maintained by the Instrumentation Branch of the Division of Traffic Operations. These consisted of electronic counters and paper tape printers to output rpm, speed, distance, time, and cumulative fuel consumption (2). A series of identical constant velocity runs over 8800- and 1000-ft test sections yielded a standard deviation of only 1.5 and 3.9 percent, respectively, in the results from the fuel portion of the over-all measurement system.

Comparisons with Field Test Data

Although considerable difficulty was encountered in keeping the total measurement system operative, it was possible to gather data on a wide variety of runs with different vehicles moving over different alignments at various speeds and rates of acceleration and deceleration as previously outlined. For comparison purposes, computer runs were then made with the Simulation Program using these same vehicle, alignment, and speed conditions. Figure 5 shows the results of a comparison of actual speed profiles with those predicted for the same conditions by the computer. Figures 6 and 7 show comparisons of actual fuel consumption with that predicted by the computer. The computer results shown here are actually for the program as revised somewhat in light of the field test data. In particular, it was possible to infer from these data that the mathematical form of the chassis resistance equation, which had always been suspect, was applicable only to heavy vehicles. Changing this relation to a more generally applicable form (see Appendix) brought the simulation results much closer to the actual.

In general, the computer simulation results agree quite well with those of the field tests. Such error as does exist can be explained in part on the basis of at least three factors. The first of these is the problem of engine adjustment already mentioned. The computer programs use theoretical torque and fuel consumption performance standards which the actual engines will not generally meet, particularly in the middle speed ranges, where theoretical fuel consumption is a minimum. Adjusting the program input to account for this problem properly is difficult at best. A second problem is that in the lower speed and horsepower ranges the fuel performance map (Fig. 10) tends to be unreliable. Without a larger number of test results than were available, any comparison at low speed is thus open to some question. This unreliability is particularly noticeable in the downhill comparisons (Fig. 6). The fuel consumption rate under this condition although relatively low, is evidently somewhat erratic. A third problem stems from the uncertainty of the coefficient of air resistance which was used in simulating the performance of the test vehicles. This can produce particularly large errors in the fuel consumption at high speeds, where air resistance accounts for the bulk of the total resistances to vehicle motion.

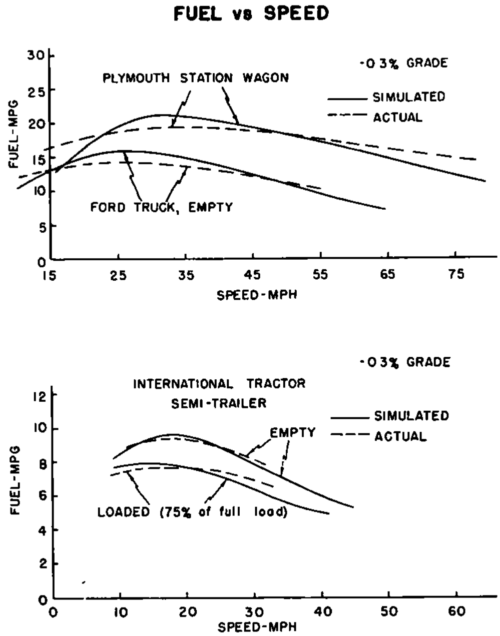

Figure 7. Fuel consumption comparisons of simulated and actual runs for all three test vehicles as a function of speed.

Such lack of agreement as remains is probably not serious. One reason is that the Simulation Program will normally be used in estimating relative rather than absolute vehicle performance and operating costs. Some inaccuracy is inevitable in the entire estimating process in any case, because of the difficulty involved in choosing the "average" vehicles which are to be taken as representative of the entire vehicle fleet using the highway under study. The conclusion is thus that the programs can provide the highway design engineer with an analysis technique of sufficient accuracy.

USING THE PROGRAMS

Possible Areas of Application

Probably the most important application of this new analysis technique will be in connection with the preliminary engineering phase of highway route location problems. In this phase the design engineer is considering alignment alternatives which could produce major differences in vehicle operating costs. A typical problem might involve determining the additional construction expense justified by the savings in vehicle operating costs which

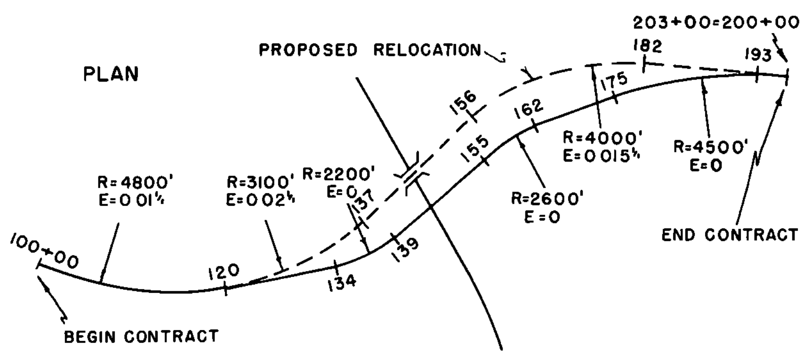

Figure 8. Plan of Sample Relocation Problem showing existing and proposed locations.

would result from a reduction in grade. Another might involve the choice between a long bridge on a direct line and a shorter and less costly bridge on a more circuitous line.

Another class of problems amenable to analysis by these computer programs is that of intersection and interchange design. The increased vehicle user costs associated with the stops required by an at-grade intersection, for instance, may in themselves justify construction of a grade separation. Further than that, the question of whether to carry a main road over or under a secondary road may even turn on the resulting difference in user costs. The decision between a directional and a cloverleaf type of interchange is still another application. Finally, the whole question of interchange spacing and location can be answered rationally only on the basis of an analysis of user costs. Because the cost per vehicle must be multiplied by thousands of vehicles, moreover, this analysis must be sufficiently detailed to detect the differences between small segments of new and old highway routes.

The computer analysis technique is able to detect just such differences in detail. The AASHO Road User Benefit Analysis Manual generally is not, based as it is on average alignment conditions only. In the few instances where more detailed analytical studies of vehicle performance on alternative alignments have been made, the results have indicated that the design engineer needs such an analysis to aid him in reaching a decision (3).

A Sample Problem

A sample highway design problem can best illustrate the use of the computer analysis technique. Figure 8 shows the plan of an existing section of highway and its proposed relocation. Figure 9 shows the vertical alignment profile and speed restrictions for the two locations. (Figure 1 showed the speed and alignment profiles for this same problem.) The proposed alignment would (a) reduce the grades and eliminate some rise and fall, (b) eliminate a stop (and 15-sec average wait) at an at-grade intersection, (c) provide better geometric design so that drivers would raise their maximum speed slightly, but (d) increase the length of the line a bit. The problem is to determine the effect of these changes on vehicle operating costs.

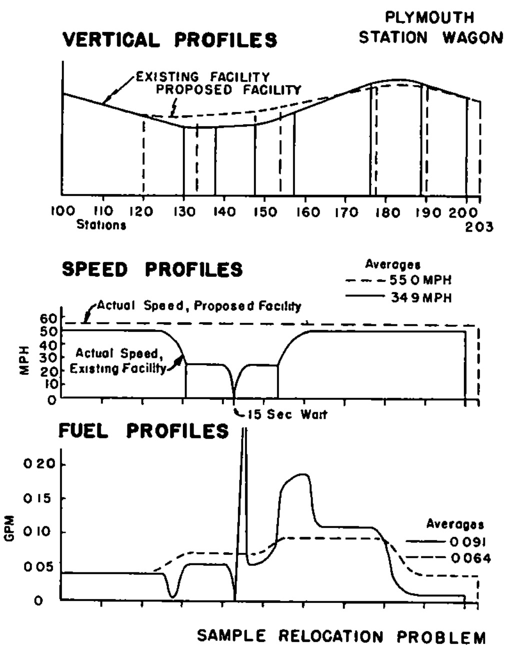

Figure 9. Speed and fuel profiles plotted from the output of the Vehicle Simulation Program (vertical profile is repeated here for reference).

Let it be further assumed that the highway in question has an ADT of 3,000 vpd, split evenly in each direction, and composed of 90 percent automobiles and 10 percent heavy trucks. For purposes of simplicity, the example considers only two classes of vehicles. In the computer simulation runs these two classes were represented, respectively, by the Plymouth station wagon and the (loaded) 50,000-lb GCW tractor-trailer used in the program of field tests previously described. Again, these vehicles were chosen for simplicity, not because they are necessarily "average" vehicles. (The determination of "average" vehicles is a problem mentioned briefly in the concluding part of this paper.) Assume also that all vehicles go straight through (that is, there are no turns on and off the intersecting road) and that traffic volumes will remain constant over the life of the facility.

Using this basic information it is possible to compute the difference in vehicle operating costs with the help of the computer programs. This requires simulation of eight trips: one for each type of vehicle in each direction (forward and return) over each alignment (existing and proposed). Table 1 gives the results of these runs. In addition, Figure 9 shows the actual speed profile and the variation in rate of fuel consumption for the station wagon when running in the "forward" direction on both the existing and proposed alignments. (These runs correspond to those entries in Table 1 marked with an asterisk.)

TABLE 1

		Average Speed (mph)			Avg. Fuel Consumption (gal/mi)		
Vehicle	Alignment	Fwd.	Ret.	Avg.	Fwd.	Ret.	Avg.
Automobile	Ex.	35.4*	35.3	35.4	0.0652*	0.0678	0.0665
	Prop.	55.0*	55.0	55.0	0.0574*	0.0592	0.0583
Heavy truck	Ex.	20.9	26.2	23.2	0.2932	0.2870	0.2901
	Prop.	45.6	48.4	47.0	0.2283	0.2242	0.2263

Note: Data on existing alignment include the stop.

Several things are interesting about these computer simulation results. First, the forward and return trips are virtually identical for the station wagon, because the alignment does not limit vehicle performance. This is not true of the heavy truck. Second, the deceleration and acceleration required for the low speed zones and the stop on the existing alignment have a serious effect on average speed, as would be expected. (The waiting time at the stop itself is relatively unimportant.) Third, the average fuel consumption rate is clearly reduced for both vehicles despite the higher speed on the proposed alignment.

Multiplying these results (expressed in average seconds and gallons per one-way trip) by the appropriate unit user costs and extending them by the estimated traffic volumes will yield the difference in total user costs between the existing and the proposed alignments. Using the cost of fuel as $0.30 per gallon and the cost of time as $2.00 per automobile hour and $3.00 per truck hour (neither unit cost is necessarily correct; the value of time must be left to the engineer's discretion) and using the traffic volumes previously suggested, the user cost savings would be as follows (no attempt has been made to include maintenance, tire wear, and oil costs, which would in any case be only slightly affected by the change in alignment suggested):

Fuel savings—auto 0.0126 gal at $0.30 =	$0.00378
Time savings—auto 65.3 sec at $2.00/3600 =	0.03628
Total savings—auto (psi 1-way trip) =	0.04006
Fuel savings—truck 0.110 gal at $0.30 =	$0.03300
Time savings—truck 145 sec at $3.00/3600 =	0.12083
Total savings—truck =	0.15383

Annual Savings

Automobiles = (3000)(0.90)(365) at $0.04006 =	$39,479
Trucks = (3000)(0.10)(365) at $0.15383 =	16,844
Total all vehicles =	$56,323

This final figure when combined with other costs and cost savings can then be used in a benefit cost, annual cost, or rate of return analysis.

OTHER CONSIDERATIONS

Discussion of Analysis Technique

One of the main objectives in setting up the computer programs was to make them simple to use. Thus, the programs are designed so that the highway design engineer can make an analysis of vehicle operating costs without any specialized knowledge of vehicle performance. In addition, all of the required vehicle parameters will either be readily available to him from the manufacturers' published data, or they will be tabulated in the Program Operating Manual. An even greater simplification can be made, however, in that it will be possible merely to furnish design offices with a deck of data cards already prepared for a set of representative vehicles by the Parameter Computation Program. This would obviate the need for an engineer to run anything but the Vehicle Simulation Program or to prepare anything other than alignment and speed restriction data as input to it.

This procedure could have the added advantage of centralizing in the hands of a more specialized group the very difficult problem of choosing precisely those vehicles which can best represent a small number of vehicle classes whose composition is almost hopelessly heterogeneous. Present thinking on this problem is that the entire fleet of vehicles using the highways should be broken into three classes: automobiles and four-wheeled light trucks, medium single-unit trucks, and heavy combination trucks. A representative vehicle would be chosen from a cross-section of the total population of each class in such a way that its performance would be average for its class. Ideally the population of vehicles sampled should, moreover, be the average population in use during the (future) life of the highway in question, a factor which makes the choice still more difficult. (Taking this problem out of the hands of the design engineer still leaves him, of course, with the job of estimating future traffic volumes by vehicle class.)

Once the necessary vehicle and alignment data are in hand, the analysis can go very fast. The computing speed of the Simulation Program is such that in the case of a detailed analysis the vehicle trip miles covered per computer hour is about three times that of the average simulated speed. As an example, three classes of vehicles moving at an average speed of 40 mph could be run in both directions over 5 mi of highway in 15 min of computer time. In a normal route location study with a good deal of constant speed running and no need to punch out intermediate answers, this time could be even further reduced.

These performance figures apply to the use of the program on the IBM 650 EDPM. The programs are presently coded for this machine and the Program Operating Manual will be written for it as well. A FORTRAN version of the programs will also be available, however, so that even better performance can be achieved on larger computers.

The brief description of the program logic and mathematics included here shows that the models used are not particularly sophisticated. The Simulation Program will not handle torque converter transmissions in quite the right way. A sacrifice was made in this respect to eliminate the need for two different simulation programs, although the discrepancy in the answers is probably not serious in any case. The program also has difficulty in predicting downhill fuel consumption. This is shown in Figure 6 by the lack of agreement between the actual and predicted fuel curves for large negative gradients. Again, this weakness may not be serious in this case because fuel consumption is low under these conditions anyway.

As pointed out, the results of the field tests indicate that notwithstanding these problems the programs can give answers which are wholly satisfactory for the highway design engineer's purposes. Certainly the answers are far better with respect to detail than anything which is otherwise available.

Future Research

The development of a satisfactory set of computer programs removes the major obstacle to the evolution of the more sophisticated technique for the analysis of highway user costs which is the ultimate objective of this research. It is clear, however, that much work remains to be done before the full benefits of this technique can be realized in highway design engineering practice. A continuing program of research contemplates work in the following areas:

1. Inquiring further into the possibilities for and desirability of increasing the ac-

curacy of the simulation model. This will involve a closer look both at the mathematics of the model as well as at some of the parameters presently being used in the equations. Part of this will be based on further analysis of the field test data and part on the experience gained by using the programs on actual test projects. As pointed out, accuracy no longer seems a problem for the bulk of the uses to which the program will be put. If minor changes can broaden the applications for which the program is suitable, however, they probably should be made.

2. Increasing the flexibility of the program. A number of things might be done in this regard. The most important is an investigation into ways of modifying the programs so that they can simulate the effect of traffic interference on vehicle performance and thus on operating costs. This is a particularly difficult problem, however, on which no early progress is in sight.

3. Applying the programs to actual location and design problems so as to study the extent to which operating costs are sensitive to design changes. This is really a three-part effort. It will, first of all, provide insight into the reasonableness of highway design criteria. Second, it will help to identify weaknesses in the analysis technique as well as the programs themselves. Finally, it will help to make more clear the ways in which the analysis techniques can be used on a practical basis.

4. Preparing summary information to relate vehicle performance and operating costs to highway alignment characteristics. The computer programs provide a very efficient way to develop this sort of data. It is envisioned, therefore, that they might be used to prepare tables and graphs similar to those in the AASHO Manual but in greater detail. These might then be used rather than the computer to analyze all but the most detailed differences between design alternatives.

5. Documentation. Although it does not properly come under the heading of further research, this phase of the work is extremely important at the present time. As mentioned at the outset of this paper, a Program Operating Manual is in the final stages of preparation. Its publication will immediately make the analysis technique available to those who want to use it. This manual will also be supplemented, however, by a detailed research report. This report will contain the detail needed as a basis for further research into and refinement of this and other techniques for the analysis of vehicle operation and costs.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the Massachusetts Department of Public Works and the U.S. Bureau of Public Roads who sponsored the research described in this paper. Thanks also are due to Professor C.L. Miller of the Massachusetts Institute of Technology under whose over-all supervision the work is being carried out.

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Appendix

Mathematics of Vehicle Motion Model

Referring to the description of the acceleration computation cycle, the following equations show the relationships used for estimating vehicle speed and distance over a given increment of time and subsequently correcting these in the iteration routine.

$$VE = VO + (AO)(DT) \tag{1}$$

in which
VO = speed at start of cycle,
AO = average acceleration used in previous cycle,
DT = specified time increment, and
VN = estimated speed at end of cycle (or corrected speed at end of cycle).

$$SN = SO + (VO)(DT) + (0.5)(AO)(DT)^2 \tag{2}$$

in which
SO = station at beginning of cycle, and
SN = station at end of cycle.

Grade resistance is taken as simply the tangential component of the weight of the vehicle and is computed by Eq. 3.

$$GR = (G)(W) \tag{3}$$

in which
G = grade (in feet of rise per horizontal foot),
W = weight gross vehicle weight, and
GR = approximate grade resistance in pounds of tractive effort.

According to the literature, the most accurate expression for both air and rolling resistances is an exponential equation. The expression used in the program, however, is the operationally simpler second order polynomial shown in Eq. 4.

$$RR = A + B(V) + C(V)^2 \tag{4}$$

in which
RR = sum of air and rolling resistances,
A, B, C = constants, and
V = speed in mph.

Constants A and B are derived by the Parameter Computation Program from the conventional rolling resistance parameters determined by other researchers (4). Constant C is derived from the coefficient of air resistance multiplied by the projected frontal area of the vehicle. The program does take into consideration any increase in vehicle resistance due to wind effects, but only by head or tail winds. Because none of these parameters is available from the vehicle manufacturer's data, they will be tabulated in the Program Manual. All of these parameters are based on the rolling resistance for a high-type concrete surface. Any surface of poorer quality would effect a linear increase in the rolling resistance. By increasing the value of A (again as outlined in the Program Manual), the program can account for this additional pavement resistance.

A percent efficiency is used to account for the loss in power between the engine flywheel and the drive wheels of the vehicle. This constant (actually an input parameter, EFF) is applied either to reduce the total torque available from the engine at the drive wheels as in the case of a maximum acceleration or to increase the sum of all the external resistances on the vehicle as in the case of constant velocity. Because torque is proportional to power at any given rpm, Eq. 5 can be used to account for the loss of power in the drive line. A variable factor should be used for transmissions with torque converters, but at present the program uses a constant similar to that for direct drive transmissions.

$$TQ = (TK)(EFF) \tag{5}$$

in which TK = engine torque (less accessory losses), and
TQ = torque corrected for drive train losses.

The efficiency factors used in the program vary from 90 to 95 percent for light vehicles operating in smaller gear ratios and from 85 to 90 percent for heavy vehicles with large gear reductions.

The rpm of the engine is computed by a set of rpm/V ratios for each gear as shown in in Eq. 6.

$$\frac{rpm}{V} = TF \times GRA \tag{6}$$

in which TF = tire factor (which includes a constant amount of slip), and
GRA = the total gear reduction.

Under average driving conditions a driver does not allow the curve resistance to be very large, because he either reduces his speed to that at which he can negotiate the curve comfortably, or he takes up more room in negotiating the curve so as to increase its effective radius, or both. Nevertheless, to handle certain situations where curve resistance is significant, the computations are set up to account for it. Eq. 7 and 8 illustrate this.

$$F = \frac{V^2}{15R} - E \tag{7}$$

in which F = coefficient of side friction,
V = speed in mph,
R = radius in feet, and
E = superelevation in ft/ft.

$$CR = (W)(CCR)(F)^2 \tag{8}$$

in which CCR = coefficient of curve resistance, and
CR = curve resistance

Eq. 7 uses a dimensionless parameter, F, to express unbalanced side friction force. Eq. 8 uses the second power of this parameter because of the parabolic nature of the slip angle versus cornering force relationship. Fiala (5) points out that the resistance to forward motion due to curvature is a function of the centrifugal force and the slip angle and further that the slip angle itself varies as the unbalanced centrifugal force. Thus, curve resistance can be considered as a function only of the weight of the vehicle, a coefficient to account for weight distribution, tire inflation pressure, etc., and the square of the unbalanced centrifugal force. The coefficient of curve resistance, CCR, in Eq. 8 is a function of the dimensions and weight distribution of the vehicle, the tire inflation pressure, and the friction between the tires and the pavement surface.

The rotating components of the engine, transmission, and wheel assemblies contribute an additional resistance when they are undergoing a change in angular velocity. This inertial resistance is accounted for by increasing the effective mass of the vehicle as per Eq. 9.

$$EM = M + K_1 + K_2 (GRA)^2 \tag{9}$$

in which K_1 and K_2 = mass equivalent constants (6, 7), and
EM = equivalent mass.

The program uses this equivalent mass rather than the true mass of the vehicle in computing acceleration.

The full throttle torque at any given rpm is computed by Eq. 10.

$$TK = E + F(rpm) + G(rpm)^2 \tag{10}$$

The coefficients E, F, and G are computed by the Parameter Computation Program.

The maximum possible acceleration can then be computed by Eq. 11.

$$AO = \frac{(TQ)\,(d)\,(EFF) - TR}{EM} \tag{11}$$

in which $d = (GRA)\,(TF)\,(d')$,
d' = dimensional constant, and
$TR = GR + RR + CR$.

As pointed out in the section on simulation logic, the program selects the lesser of either the specified maximum acceleration or the possible acceleration. It then back-figures the total tractive effort required (including the drive line resistances) and computes the power requirement by Eq. 12.

$$BHP = \frac{(AO)\,(EM) + TR}{EFF}\,\frac{V}{d''} \tag{12}$$

in which d'' = a dimensional constant.

On steep grades it is necessary to give special consideration to the effect of gear shifting on both the speed and fuel consumption of trucks. The program makes an approximation to the real case by allowing the vehicle to coast during each gear shift for an amount of time equal to the average shift time for that type of vehicle. In the case of an automatic transmission this time interval is very small; with trucks it is the average shift time, regardless of the particular shift maneuver required. In calculating the speed lost during the shift time, resistances are computed on the basis of the initial speed for the time period involved. This produces a second order systematic error, because that speed is always larger than the average speed over the coasting interval. This error, however, helps to offset the additional amount of fuel that the driver uses in double clutching. In any case, the approximate method reduces the error which would otherwise be introduced by ignoring the gear shift.

Mathematics of Fuel Consumption Model

Fuel consumption is computed on the basis of a consolidated engine performance map (8). Performance maps for particular engines are likely to show fairly wide variations due to eccentricities in the adjustment of the engines. A consolidated map of the performance of many engines in same range of compression ratios exhibits a much more uniform behavior. Because, in the last analysis, this program deals with a large number of vehicles rather than with a single one, such a consolidation is an acceptable step. Exactly what sort of errors are introduced by using such a map is not entirely clear, but the test results suggest that they are not serious. The original experimental simulation program was used to test other much simpler approaches to the fuel consumption problem. These proved entirely inadequate from the standpoint of accuracy. The performance map approach emerged as the only feasible alternative.

Figure 10 shows a typical gasoline engine performance map. The Parameter Computation Program transforms the data on this map into specific fuel consumption values on the basis of the bore, stroke, and number of cylinders of the engine in question. The resulting values of specific fuel consumption are arranged in 20 x 20 matrix in such a way that the values in the columns are proportional to the rpm and the values in the rows are proportional to the brake horsepower. This table of values (along with the other summary vehicle parameters) is punched out on cards to be used directly as input to the Vehicle Simulation Program. It is then possible for the Simulation Program to compute the fuel consumed in any time increment by looking on the table for the fuel rate as a function of the rpm and the required horsepower and multiplying that value by the time increment itself.

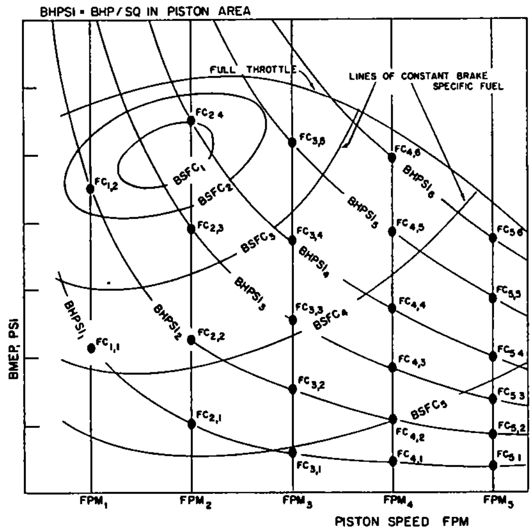

Figure 10. Typical gasoline engine performance map. Data stored in program are FPM$_1$, FPM$_2$...; BHPSI$_2$...; and FC$_{1,1}$, FC$_{1,2}$...; FC$_{2,1}$, FC$_{2,2}$....

Other input parameters are necessary to specify the rate of fuel consumption in pounds per hour for the vehicle when it is operating at closed throttle (that is, with the engine idling). One parameter specifies the rate when the vehicle is coasting; another specifies the rate when it is standing still. Carburetors are not normally designed to operate efficiently at closed throttle. A vehicle can thus have an extremely unpredictable fuel consumption curve under closed throttle conditions when the engine is turning over at high rpm. (An example of this would be coasting down a grade.) Evidently, a certain amount of additional fuel escapes past the idling jets as a result of the manifold pressure being different from the idling case. It is this additional fuel requirement that makes it necessary for the program to use one idling parameter when the vehicle is in motion and another one when it is standing still. Although the coasting rate is apt to be extremely erratic in any given vehicle, it is assumed to be consistent over a large number of vehicles.

Effect of Parkway Medians on Driver Behavior—Westchester County Parkways

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● THE Saw Mill River, Bronx River, and Hutchinson River Parkways are north-south passenger car express routes located in southern Westchester County, New York. These Parkways and the New York Thruway are interconnected by the Cross County Parkway in an east-west direction about 2 mi north of and parallel to the New York City line.

The four Parkways form a 65-mi network system of multilane limited-access highways under the jurisdiction of the Westchester County Park Authority and Commission for operation, maintenance and policing. (The Bronx River and part of the Saw Mill River Parkway are under the jurisdiction of the Commission.) These through traffic arteries provide inter-communication for the densely populated areas of Westchester County, southwestern Connecticut and New York City. This Westchester County Parkway System is shown in Figure 1.

Because of the large number of accidents, many of them fatal, that had occurred on the Cross County and Hutchinson River Parkways, the county officials, who had long foreseen the need for their improvement, decided in 1952, that it was time for them to take action. Accordingly, preliminary engineering reports were contracted for and prepared in 1954 with respect to traffic inadequacies and capacities to be provided for in reconstruction (1).

The consultant's report pointed out that the 4-lane Cross County Parkway, 6 mi in length and 40 ft wide, was constructed in stages between 1920 and 1941 with but 1.2 mi with separators. They also considered the Parkway inefficient and hazardous as a traffic artery because to quote: "(a) Its design is insufficient when compared to that of other modern parkways which provide continuous high speed operations; (b) its low design standards (viz lack of medial division and shoulders, and excessive horizontal curvature) produce a quality of route flow below that considered desirable by parkway users; and (c) its accident experience and particularly its fatality rate are higher than those of other parkways, and occur at lower speed levels". (2)

From 1953 to 1955 reconstruction contracts added about 1 mi of divided parkway with 24-ft roadways separated by pipe-type barrier or curbed mall. Then in 1958 the county officials took the interim expediency of installing double-faced steel beam-type guide rails along the center of the pavement on the easterly 2.2 mi of the Parkway replacing the paint line surface divider.

The Hutchinson River Parkway, 15 mi in length and with 23-ft wide roadways in each direction for most of its length and separated by a raised curbed median, usually 4 ft wide, was constructed between 1925 and 1930. Quoting from the consultant's report, "It is now inadequate for the large amount of traffic it serves because: (a) Its design is quite insufficient compared with that of modern parkways providing safe, continuous high speed operations; (b) its low design standards produce speeds lower than those considered desirable by parkway traffic; (c) its accident experience is higher than that of other parkways when operating speeds are taken into consideration; and (d) traffic volumes now exceed the practical capacity of the parkway over most of its length". (2)

In August 1957, the county installed 1,217 ft (0.23 mi) of steel double-faced beam-type guide rail in the center of the 4-ft wide median on the northerly end of the Parkway.

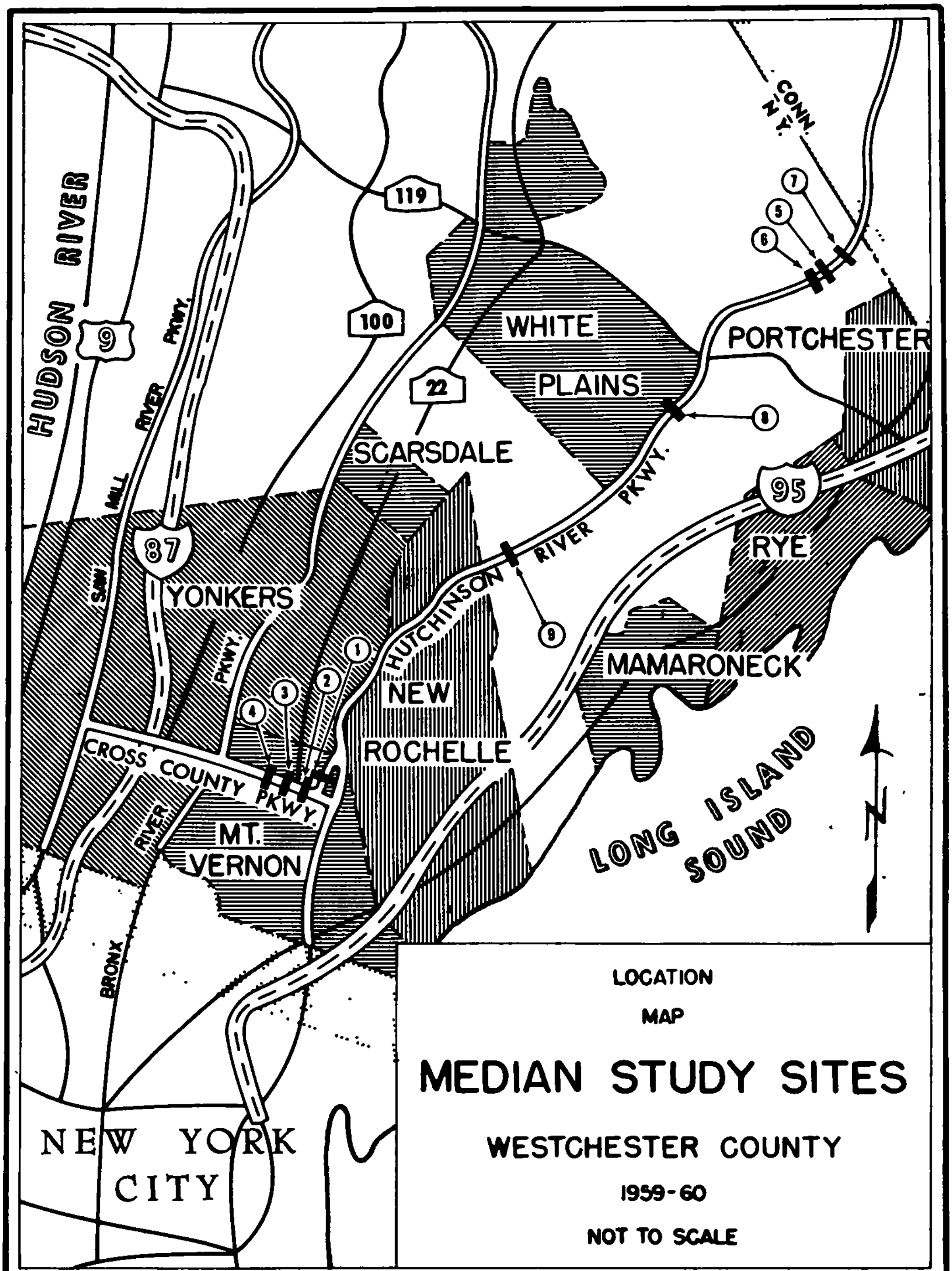


Figure 1. Location of study sites.

This installation was made on a compound series of relatively sharp horizontal curves in an effort to reduce cross median accidents at this location. The legal speed limits on the Cross County and Hutchinson River Parkways are 35 and 40 mph, respectively.

Based on observed traffic operations of the Cross County Parkway with the guide rail median, questions arose as to the feasibility of installing that type of median on the longer Hutchinson River Parkway. Thus, in the spring of 1959 the State was solicited to conduct a study to evaluate the effectiveness of the median on driver behavior. Consequently the study proposal included, in addition to those on the Cross County Parkway, sites typical of the concrete paved and grass with and without median guide rail types of medians, all 4 ft wide and curbed, on the Hutchinson River Parkway.

SCOPE OF STUDY

For the Cross County Parkway, four study sites were decided on for data collection, two on the travel sections with guide rail median and two on the travel sections with surface paint line divider. One site in each group was representative of the horizontal curve travel sections and the other of the tangent travel sections of the Parkway.

For the Hutchinson River Parkway, five study sites were decided on for data collection. All sites had 4-ft wide medians. Raised grass medians without guide rail were studied at one tangent and one horizontal curve, and the third study site was on a horizontal curve with guide rail in the median. The fourth and fifth study sites were on a tangent and a horizontal curve, respectively, with raised and paved (with portland cement concrete) medians.

Data were to be collected for both day and night traffic. Figure 1 shows the general location of the nine study sites.

PURPOSE

The purpose of the study was to evaluate by means of a day and night study of traffic performance the effectiveness of (a) the guide rail median on the 40-ft wide Cross County Parkway as compared to the surface paint line divider, and (b) the 4-ft wide raised and curbed grass compared with the 4-ft wide raised and curbed paved and the 4-ft wide raised and curbed grass with guide rail types of medians on the Hutchinson River Parkway. The effects to be ascertained would be those manifested in drivers' actions such as speeds, lateral placement and lane use, and in accident experience.

DESCRIPTION OF STUDY SITES

Sites 1 through 4, located on the easterly end of the Cross County Parkway, provided sites to permit comparison of data for the guide rail (sites 1 and 2) with the painted centerline divider (sites 3 and 4) types of median. The painted centerline consisted of two 4-in. wide solid white lines 4 in. apart on both tangents and curves. Figure 2 shows the details of sites 1 and 2 with the guide rail and Figure 3 those for sites 3 and 4 with the painted center divider. Table 1 contains a description of these four study sites.

Sites 5 through 9 were located on the Hutchinson River Parkway between its junction with the Cross County Parkway and the Connecticut State line. The three northern sites (sites 5, 6 and 7) were located on the section having a raised grass median 4 ft wide with 3-in. high concrete curbs. Site 5 was on a tangent; site 6 was on a horizontal curve and site 7 was on a curve with a guide rail in the center of the median. This was the only section of the Parkway with a guide rail in the median. Figure 4 shows the details of these three sites. The two southern sites (sites 8 and 9) were on the section having a 4-ft wide portland cement concrete median, 6 in. high. Site 8 was on a tangent and site 9 was on a horizontal curve. Figure 5 shows the details of these two sites and Table 1 contains a description of the five sites.

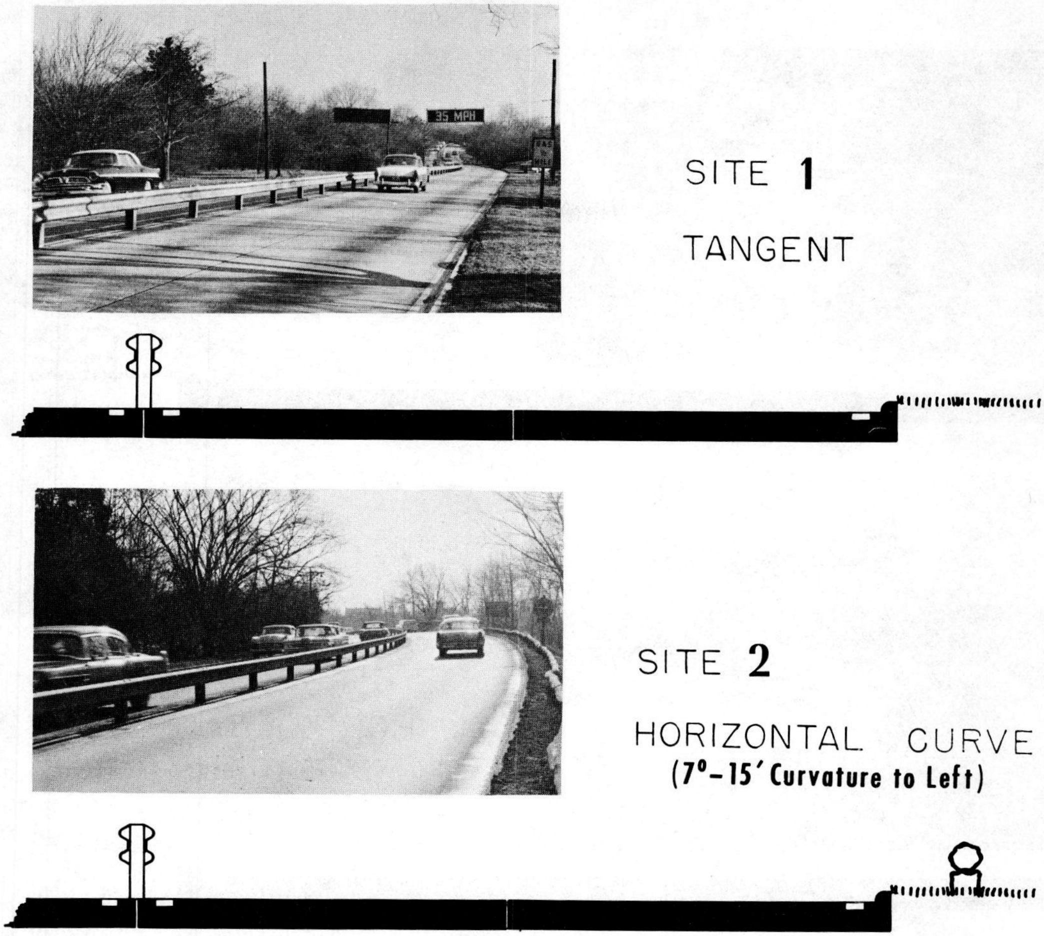


Figure 2. Sites with guide rail medians on the Cross County Parkway.

DATA RECORDED

The Bureau of Public Roads' Traffic Analyzer (3) was used to record the speed and placement of each vehicle as it passed the points of observation during day- and nighttimes. The dates of study, duration, number of cars observed and the hourly traffic volumes are given in Table 2. More than 42,000 passenger cars were studied. The multitude of data were processed by high-speed electronic equipment.

ANALYSIS

Comparison Between Day and Night Operations

It was preconceived that there might be a difference in driver behavior during the daylight and darkness hours. For this reason the analysis was directed first toward a comparison of speed and placement data during these periods. As given in Table 3, the average daylight speeds at all sites were generally within 1 mph of the speeds observed during darkness. Taking into consideration that at all sites both the day and night speeds were higher in the median than the curb lanes, the percent of cars exceeding the legal speed limits was also generally comparable for day and night.

A comparison of the total placement data, for day and night operations as given in Table 4, revealed that the placements were substantially the same for the curb and median lanes of the Cross County and the curb lane of the Hutchinson. The average

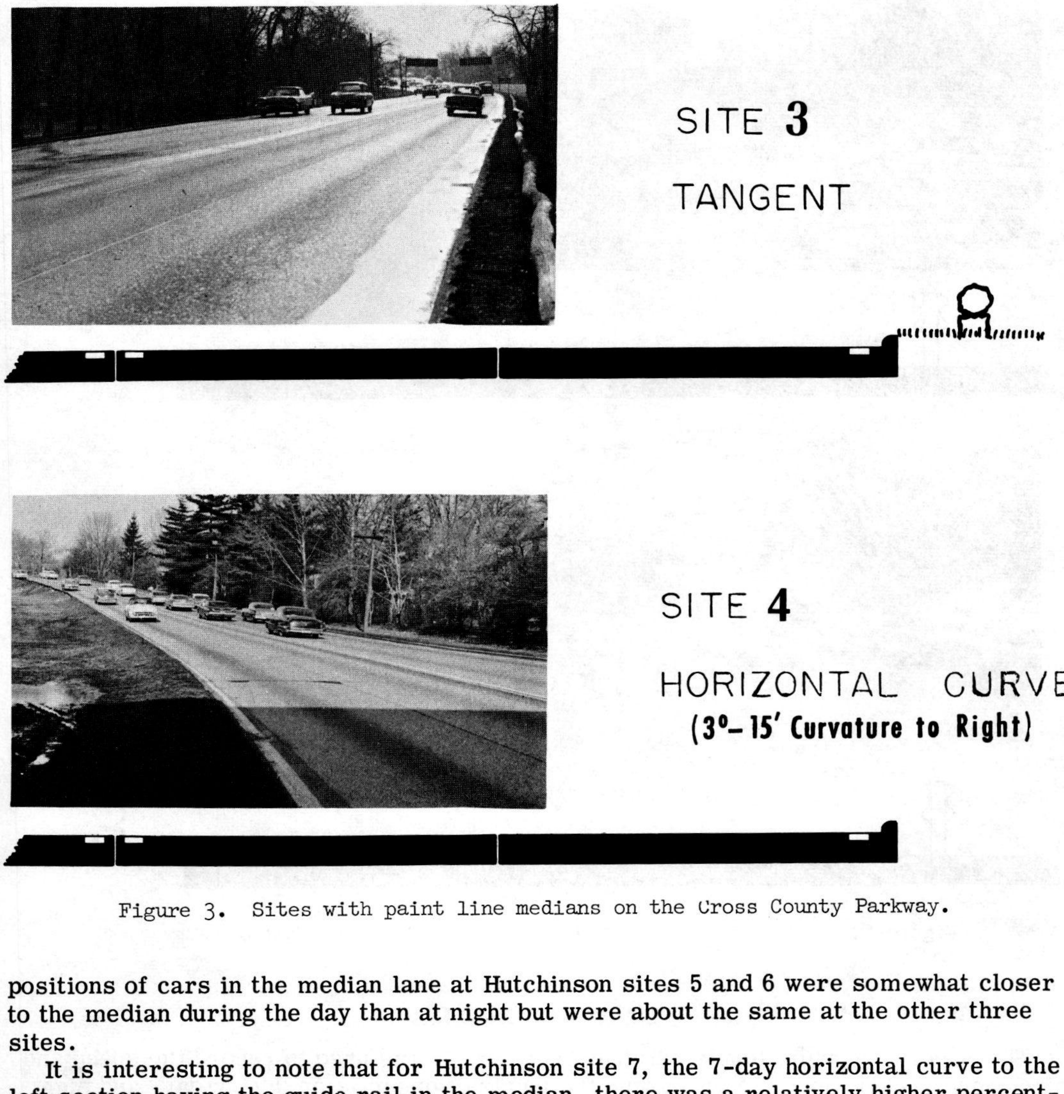


Figure 3. Sites with paint line medians on the Cross County Parkway.

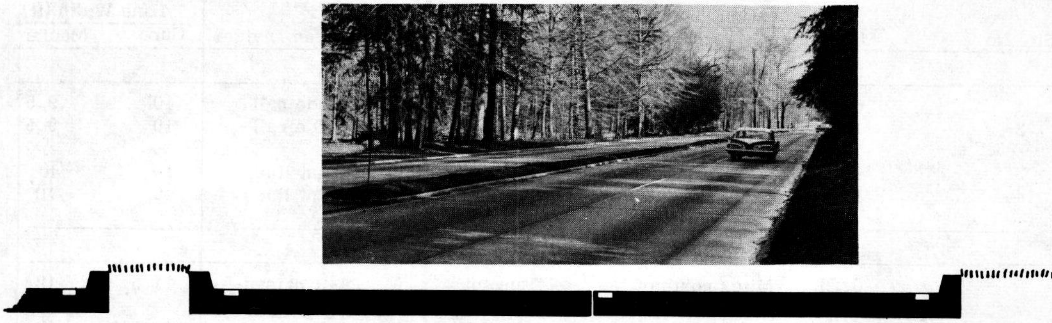
positions of cars in the median lane at Hutchinson sites 5 and 6 were somewhat closer to the median during the day than at night but were about the same at the other three sites.

It is interesting to note that for Hutchinson site 7, the 7-day horizontal curve to the left section having the guide rail in the median, there was a relatively higher percentage (33) of the cars straddling the lane line (Table 4) during the night than day (20). Sharp curves to the left and lighter traffic at night tend to produce these straddling results, both conditions being present at this site.

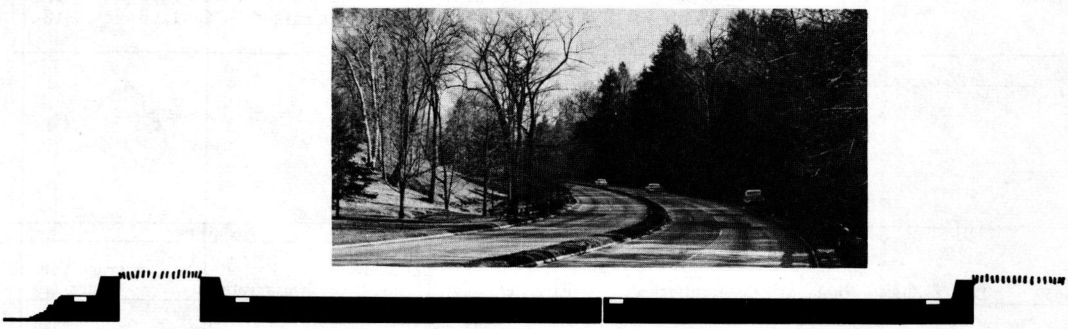
The driver behavior was generally about the same during daytime as during nighttime. Therefore, the ensuing analysis will be limited to the data recorded during the daylight hours only.

Relation of Volume to Speed and Lateral Placement

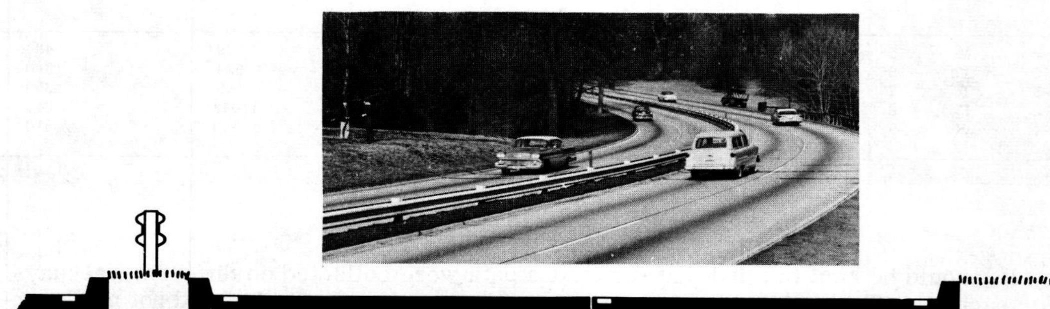
Of the nine sites studied, only the Cross County sites (sites 1-4) carried a range in traffic volume which permitted an analysis to determine the effect of volume on speeds and lateral placements. At these sites the volumes based on 6-min periods ranged from 800 cars per hour to approximately 2,800 vehicles per hour for one direction of travel. Table 5 gives the average speeds and lateral placements by the several volume groups for the four sites studied on the Cross County Parkway. Although the average speed decreased somewhat (generally less than 1 mph between volume groups) as the volume increased, the decrease in speed was not of a magnitude that seems to be worth consideration for further analysis.



SITE **5** TANGENT



SITE **6** HORIZONTAL CURVE
(4°–15′ Curvature to Left)



SITE **7** HORIZONTAL CURVE
(7°–00′ Curvature to Left)

Figure 4. Northern Hutchinson River Parkway sites.

TABLE 1

DESCRIPTION OF STUDY SITES

Site No.	Location: Miles	Location: From	Alignment	Median Divider	Lane Width (ft): Curb	Lane Width (ft): Median
(a) Cross County Parkwav						
1	0.8	South of Northerly intersection with Hutchinson River Parkway	Tangent	Guide rail	10	9.5[a]
2	1.1		Curve (7° - 15' left)	Guide rail	10	9.5[a]
3	1.4		Tangent	Paint line	10	10
4	1.8		Curve (3° - 15' right)	Paint line	10	10
(b) Hutchinson River Parkway						
5	0.25	Miles north of intersection with Lincoln Avenue	Tangent	4-ft raised grass	11.5	12
6	0.15		Curve (4° - 15' left)	4-ft raised grass	11.5	12
7	0.90		Curve (7° left)	4-ft raised grass with guide rail	11.5	12
8	1.1	South of intersection with Mamaroneck Ave.	Tangent	4-ft raised PCC	11.5	12
9	1.5		Curve (2° - 30' left)	4-ft raised PCC	11.5	12

[a]Guide rail occupies 0.5 ft of median lane.

TABLE 2

TRAFFIC DATA

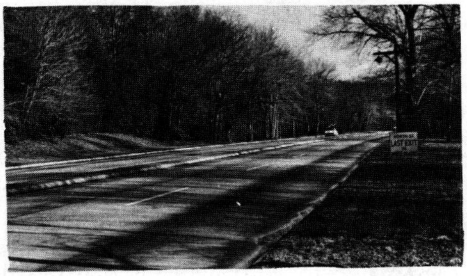
Date 1959	Day	Site	Daytime: Duration of Study (hr)	Daytime: No. of Observations	Daytime: Average Volume[a] (cars/hr)	Nighttime: Duration of Study (hr)	Nighttime: No of Observations	Nighttime: Average Volume (cars/hr)
(a) Cross County Parkway								
2-26	Th	1	2.5	2,871	1,150	2 0	2,355	1,180
2-27	F	2	2 5	4,547	1,820	2.0	3,688	1,840
2-25	W	3	2.5	4,148	1,660	2 0	3,257	1,630
3-2	M	4	2.5 (3-5 30pm)	4,046	1,620	2.0 (6-8pm)	3,041	1,520
(b) Hutchinson River Parkway								
3-9	M	5	2 8	1,489	530	1.7	679	400
3-5	Th	6	2.8	1,803	640	1 7	524	310
3-3	T	7	2 8	1,689	600	1.7	586	350
3-6	F	8	2.8	2,151	770	1 7	1,042	610
3-4	W	9	2.8 (3-5 30pm)	3,015	1,080	1 7 (6 18-8pm)	1,370	810

[a]Northbound traffic was studied at site 1 and eastbound at all other sites.

It should be kept in mind that the traffic data were collected on different weekdays for each site (Table 2) and furthermore the volumes given in Table 5 did not necessarily occur at the same time of day for each site. Therefore, the comparisons of data between sites by volume group do not reflect simultaneous volumes. Also, only at site 2 were the speeds for the same volume groups consistently lower than at the other sites. This leads to the conjecture that the larger volumes recorded at site 2 may be attributed to the lower speed.

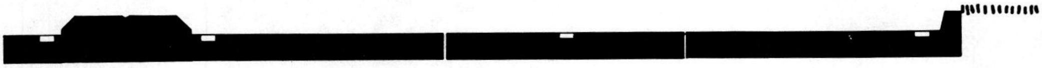
Discussion

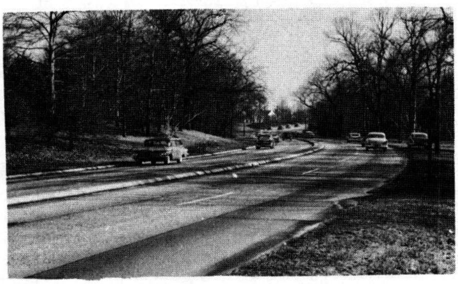
It would seem that the curve at site 2 (with guide rail divider) being 7°-15' may have had some effect in reducing the average speeds but on the other hand the 3°-15' curve at site 4 (with paint line divider) did not appear to have the effect of reducing speeds when compared to the speeds for tangent site 3 (with paint line divider). In



SITE **8**

TANGENT





SITE **9**

HORIZONTAL CURVE

(2°–30′ Curvature to Left)

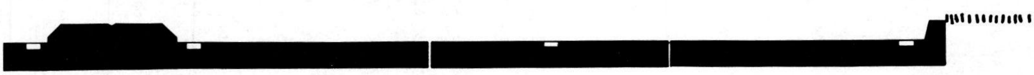


Figure 5. Southern Hutchinson River Parkway sites.

TABLE 3

COMPARISON OF DAY AND NIGHT SPEEDS BY LANE

Site No.	Curb Lane: Average Speed (mph) Day	Curb Lane: Average Speed (mph) Night	Curb Lane: Exceeding Speed Limit[a] (%) Day	Curb Lane: Exceeding Speed Limit[a] (%) Night	Median Lane: Average Speed (mph) Day	Median Lane: Average Speed (mph) Night	Median Lane: Exceeding Speed Limit (%) Day	Median Lane: Exceeding Speed Limit (%) Night
(a) Cross County Parkway								
1	40.4	39.7	90.7	86.3	44.0	42.9	96.3	96.6
2	35.9	34.9	56.9	48.1	37.6	35.8	72.4	58.4
3	38.0	36.3	75.3	58.8	43.6	41.9	97.4	96.0
4	42.0	41.5	97.2	96.1	46.0	45.3	98.5	98.6
(b) Hutchinson River Parkway								
5	47.2	47.8	89.0	93.6	51.0	51.8	99.0	98.5
6	45.5	45.6	83.5	84.0	48.8	49.7	94.7	97.8
7	42.2	42.8	66.0	45.8	45.8	46.8	89.1	95.7
8	43.8	44.4	87.1	48.9	48.9	49.6	95.8	97.7
9	44.7	43.7	78.1	68.4	49.2	47.8	97.2	96.6

[a]Legal speed limit on the Cross County Parkway (sites 1-4) was 35 mph, and 40 mph on the Hutchinson River Parkway (sites 5-9).

fact, the average speeds on the curve (site 4) were considerably higher (more than 3 mph) than on the tangent (site 3). Therefore, it would appear that the guide rail divider contributed, to some extent, to the lower average speeds at site 2.

This apparent effect of the guide rail divider on reducing speed on curves was further explored by applying the effect of degree of curvature on average speeds found by Taragin in some earlier driver performance research in New York and other States to quote, "The average speed is lower by 3 mph for each 4 deg that the curvature increase..." (4). Coincidentally there was a 4-deg difference in curvature between sites 2 and 4 (7°-15' and 3°-15'). Thus with all other variables being equal, the difference between the average speeds at these two sites would be 3 mph. The actual difference (Table 5) averaged more than 7 mph. Therefore, it appears that the guide rail divider at site 2 had the effect of reducing the average speeds more than 4 mph at all volumes and was somewhat greater than the effect of the degree of curvature.

TABLE 4

COMPARISON OF DAY AND NIGHT LATERAL POSITIONS OF CARS BY LANE AND RELATED DATA

Site No.	Average Placement[a] (ft) Curb Lane Day	Curb Lane Night	Median Lane Day	Median Lane Night	Cars Straddling Lane Line (%) Day	Night	Clearance Between Bodies of Adjacent Cars (ft) Day	Night
(a) Cross County Parkway								
1	5.8	5.9	13.7	13.8	10.3	13.2	2.3	2.3
2	6.1	6.1	13.9	13.8	17.8	18.0	2.1	2.1
3	6.0	5.8	15.1	15.1	13.0	11.0	3.5	3.6
4	5.5	5.5	14.4	14.4	6.7	6.8	3.2	3.3
(b) Hutchinson River Parkway								
5	7.6	7.7	17.1	16.5	9.7	13.8	4.2	3.7
6	8.2	8.2	17.6	16.7	25.3	29.4	4.0	3.8
7	8.1	8.5	17.1	17.1	20.5	32.6	3.6	3.5
8	7.0	7.0	16.4	16.4	8.0	8.1	3.8	3.8
9	7.4	7.6	17.0	17.0	6.8	8.8	4.1	4.1

[a]Distance to center of cars from right edge of pavement. Placement detector was at center of curve at sites 2, 4, 6, 7 and 9.

A comparison of the average speeds for sites 1 and 3 (Table 5) showed that the guide rail divider (site 1) is associated with somewhat higher average speeds (less than 1 mph) on tangents as the volume increases.

An examination of the average lateral placement data for the Cross County study sites (sites 1-4, Table 5) shows that there is practically no relation between volume and the lateral placement of cars for the tangent sites 1 and 3 and the 3°-15' curve site 4. This is also generally true for the 7°-15' curve site 2 except at the higher volumes the average placement of cars in the curb lane are somewhat closer to the curb (0.3-0.6 ft) than at the lower volumes. This is to be expected with the somewhat lower speeds.

Based on these examinations it can be assumed, where appropriate in the ensuing analyses, that the effect of volume on either speed or lateral placement is negligible. It should be noted that in some detailed analyses, data are presented for all volumes recorded, and include the five Hutchinson study sites (sites 5-9) which carried traffic volumes usually less than 1,200 cars per hour in one direction.

TABLE 5

AVERAGE SPEED AND LATERAL POSITIONS OF CARS BY LANE FOR VOLUME GROUPS ON THE CROSS COUNTY PARKWAY

	Guide Rail Median						Solid Double Line Median					
	Site 1—Tangent			Site 2—Curve (7°-15')			Site 3—Tangent			Site 4—Curve (3°-15')		
	Average Lateral Placement[a] (ft)			Average Lateral Placement (ft)			Average Lateral Placement (ft)			Average Lateral Placement (ft)		
Volume Group (car/hr)	Median Lane	Curb Lane	Average Speed (mph)	Median Lane	Curb Lane	Average Speed (mph)	Median Lane	Curb Lane	Average Speed (mph)	Median Lane	Curb Lane	Average Speed (mph)
800-1,199	13 6	5 8	41 4	13 7	6 4	36 5	15 2	6 0	42 9	14 2	5 6	43 9
1,200-1,599	13 7	5 8	41 4	13 7	6 3	36 2	15 2	6.1	40 5	14 4	5 6	43 7
1,600-1,999	13 7	5 8	40 6	13 8	6 3	36 4	15 2	6 0	40 0	14 4	5 5	43 4
2,000-2,399	-	-	-	13 9	6 0	36 2	15 1	5 9	39 4	14 4	5 4	43.0
2,400-2,799	-	-	-	13 9	5 8	35 2	-	-	-	-	-	-

[a]Lateral Placement is measured in feet from right edge of pavement to center of cars

Analysis of Speed Data

Figure 6 shows the average speeds of cars by lane for all volume groups at the nine sites. The data are for daytime operations only. The bars to the left of this figure show the speeds on the tangent sections and those to the right show the speeds on the horizontal curves. It is interesting to note that on the tangents the speed in the curb lane is somewhat higher (3 mph) on the section (site 1) having the guide rail than on the section (site 3) having the paint line divider. The reverse is true (7 mph) for the horizontal curve sites (site 2 vs 4) even when the expected reduction in speed (3 mph), due to difference in curvature, is considered. This applies to the Cross County Parkway where the lanes are only 10 ft in width except at the sections with the guide rail, (sites 1 and 2) which makes a divided highway out of an undivided highway, but by so doing, lane width was sacrificed. Here the median lane was only 9½ ft in width. The guide rail in the 4-ft wide raised grass median on the Hutchinson was installed on only a horizontal curve (site 7). This was the only test site available with guide rail. The speed on this section was approximately 3-4 mph lower than on the horizontal curve sections without the guide rail (sites 6 and 9). This difference in speed is to be expected due to the difference in curvature (7° vs 2°-30' and 4°-15').

Figure 7 shows the average speed for a common volume of 800 to 1,200 cars per hour in one direction, a traffic volume common to all study sites. It will be noted that the speeds shown for this volume are very similar to those shown for all volumes in Figure 6.

The percentage of cars exceeding the legal posted speed limit was examined in an effort to measure the influence of the median dividers on operating speeds. The posted legal speed limit was 35 mph on the Cross County and 40 mph on the Hutchinson River Parkway. Figure 8 shows the percentage of all cars that exceeded these speed limits at all volumes. Here again, fewer cars exceeded the speed limit on the horizontal curve sections (site 2—62 percent, and site 7—68 percent) with the guide rail than on the comparable sections (site 4—96 percent, and sites 6 and 9—84 percent) not having the guide rail. On the tangents, however, a greater percentage—8 percent—of cars exceeded the speed limit when the guide rail was present (site 1 vs 3).

Because drivers usually seem to assume that about a 5-mph tolerance above the speed limit is permissible, and to pinpoint the influence of the dividers on operating speeds, the percentage of cars by lane exceeding the speed limit plus the 5-mph tolerance was compiled (Fig. 9). In this case the percentages are those exceeding 40 mph on the Cross County and 45 mph on the Hutchinson Parkway. These data are for a volume of 800 to 1,200 vehcicles per hour in one direction of travel for each of the nine study sites. For the Cross County the results are quite similar to those shown in Figure 8 except that the effect of the guide rail on horizontal curves is more pronounced; for example, at site 2, with the guide rail, only 24 percent of the cars in the median lane exceeded 40 mph, whereas for the median lane of site 4 with the paint line divider more than 94 percent exceeded the speed. On the other hand, for the median lane on the tangent sections a greater percentage—14 percent—exceeded the speed criterion with the guide rail than without it (site 1 compared with site 3).

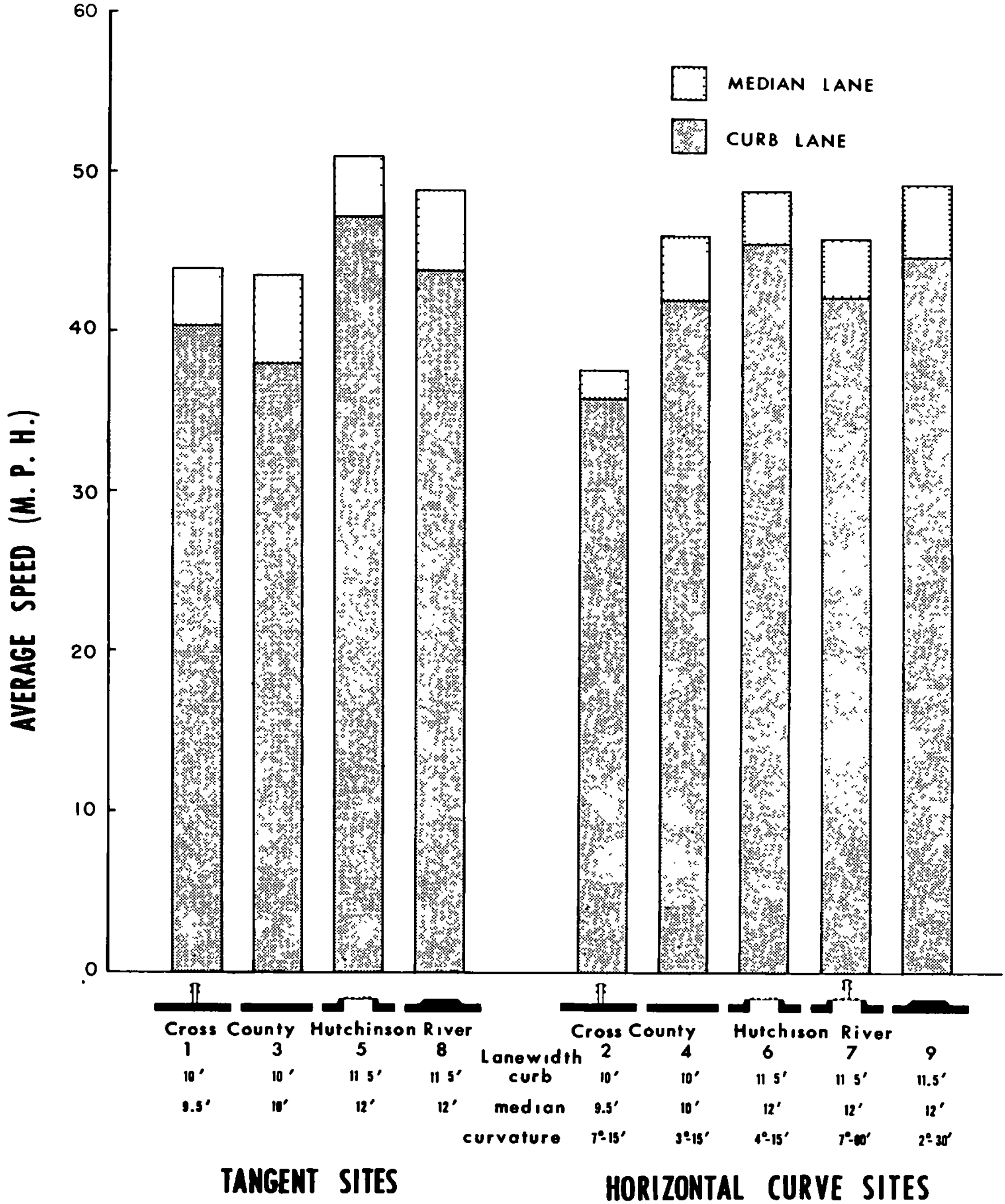


Figure 6. Average speed by lane for all volume groups during daytime.

Of interest here also is the fact that for the curb lane of the tangent sections of the Hutchinson (sites 5 and 8), the percentage of cars exceeding the speed limit plus 5-mph tolerance is considerably lower—33 percent—in the curb lane for the 4-ft wide portland cement concrete median (site 8) than for the grass median (site 5). This comparison for the median lane is rather small—10 percent.

From this study of speeds and speed distribution it appears that on a 40-ft wide section of pavement the guide rail median is effective in reducing speeds on horizontal curves. The speeds were reduced to the point where fewer drivers exceeded the speed limit. On tangent sections, however, the guide rail appears to be associated with an increase in speed. From the foregoing, it is believed that the guide rail median is an effective traffic control device on horizontal curves of narrow roadways for restraining drivers from using excessive and probably unsafe speeds. Also, on tangents, drivers

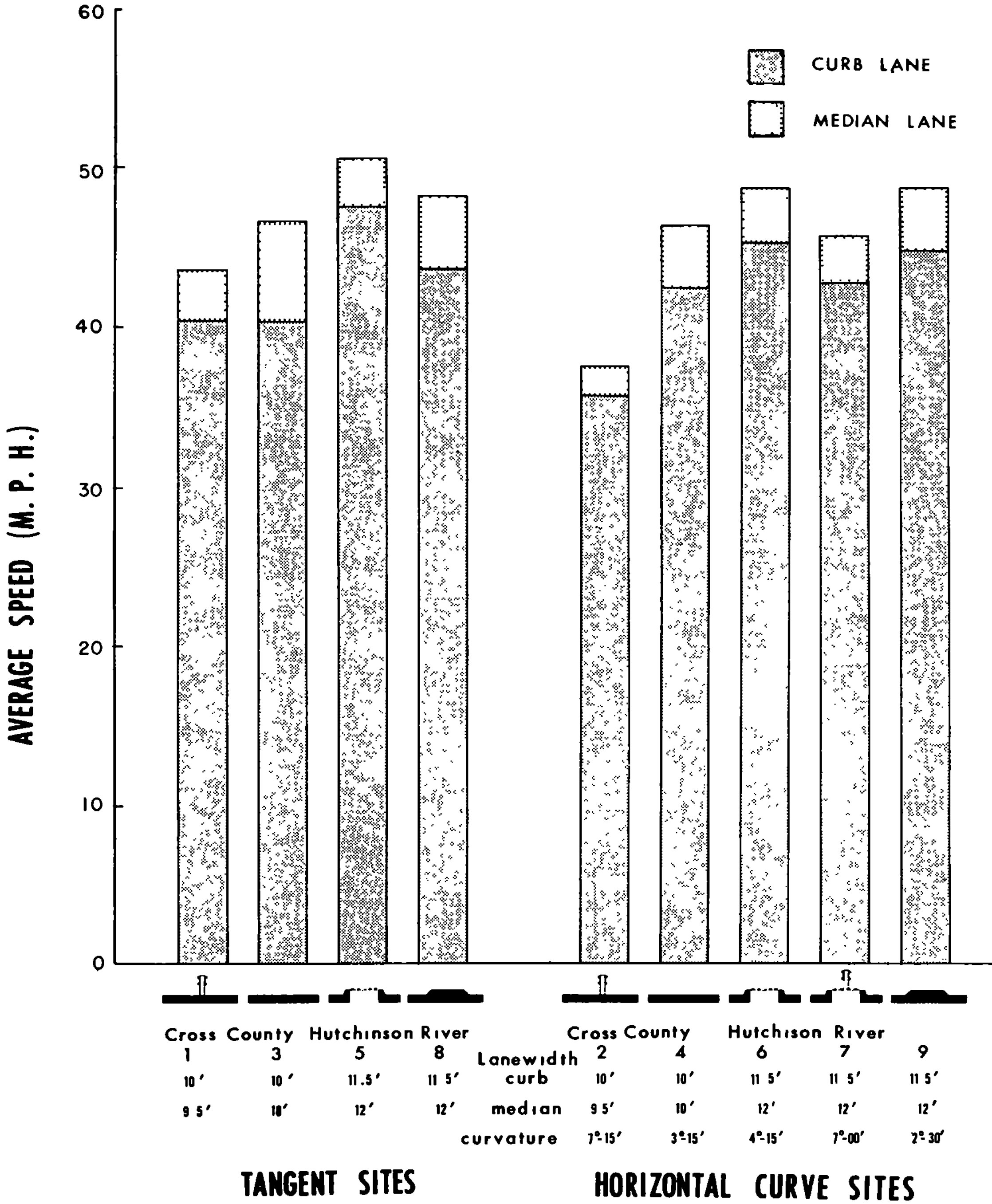


Figure 7. Average speed for a volume of 800 to 1,200 cars per hour in one direction by lane during daytime.

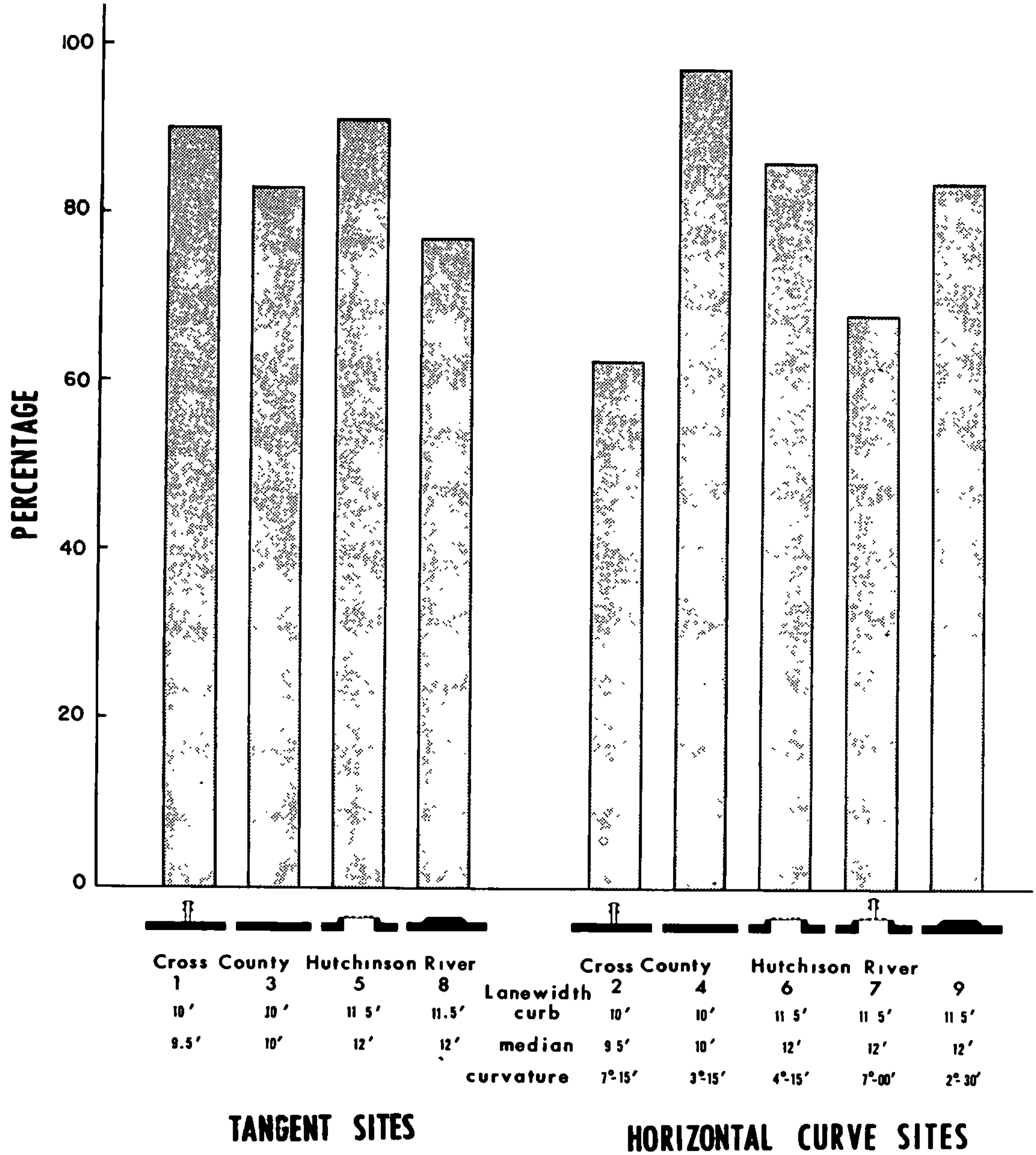


Figure 8. Percentage of cars exceeding speed limits at all volumes.

may feel that a guide rail affords safety at a higher speed than they would travel if the guide rail were not present.

Effect of Median Dividers on Lateral Positions of Cars

An extensive analysis was performed on the lateral placement data in an attempt to isolate the effect of the various median dividers studied. Detailed analyses were performed separately for free-moving cars, for adjacent traveling cars, including car clearances, and for all cars. Free-moving cars were those which were not influenced

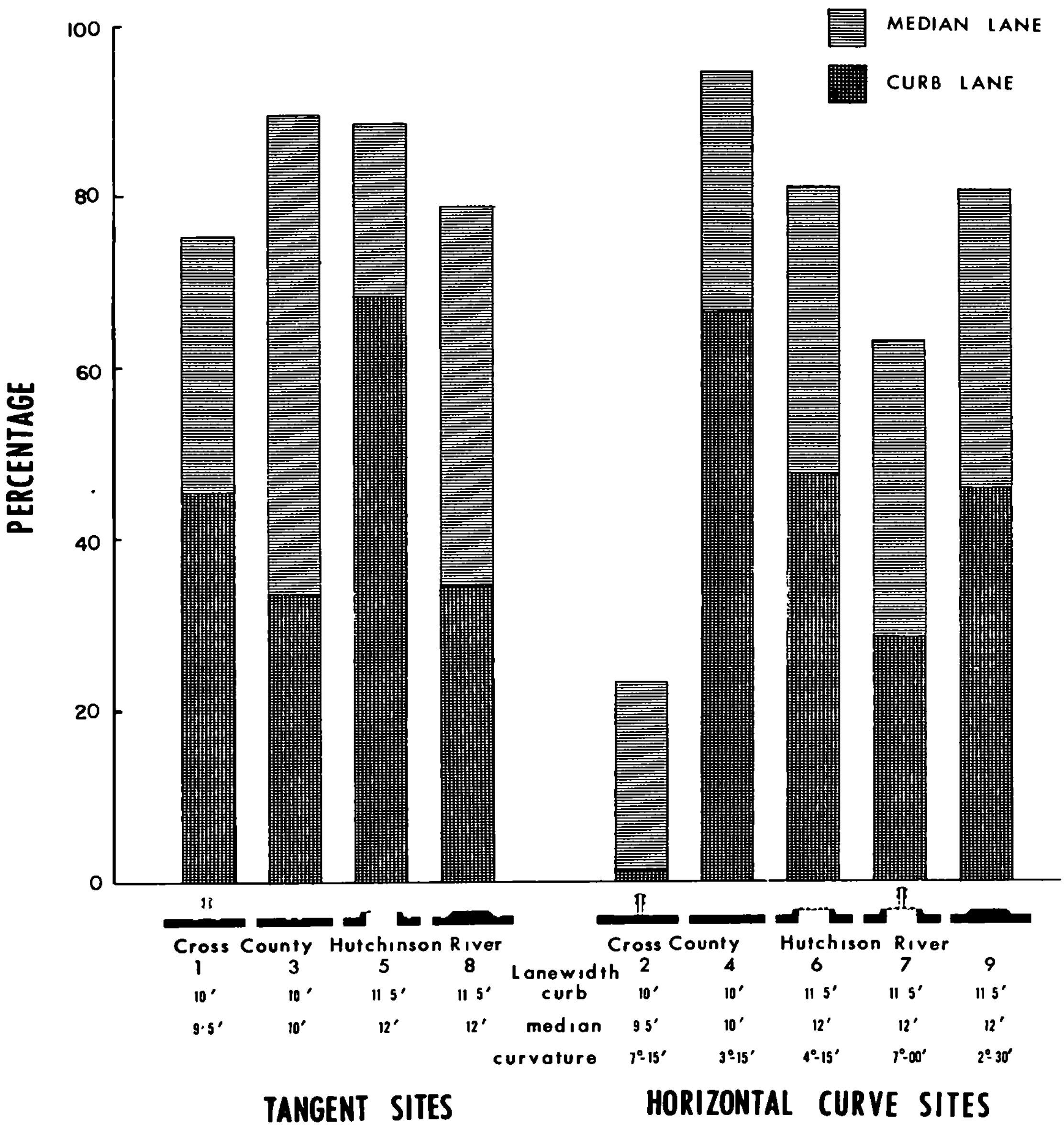


Figure 9. Percentage of cars exceeding speed limit plus 5-mph tolerance, by lane, during daytime when volume was 800 to 1,200 vehicles per hour in both lanes.

by cars ahead of them in the same lane or in the adjacent lane ahead or behind them within at least 7 sec of time interval. Adjacent traveling cars were those which had a car in the adjacent lane within 2 sec ahead or behind. Figures 10 through 13 show the distribution of the lateral positions of centers of cars for the various sites across the width of pavement for one direction of travel. The data shown on these figures are for all volumes studied during daytime conditions.

Figure 10 shows the distribution of the lateral positions of centers of cars on the tangent sections (sites 1 and 3) of the Cross County Parkway, Site 1 is the section with the guide rail median and site 3 is the section with the paint line divider. As might be expected, there is no significant difference between the distribution of positions in the curb lane for the two sites. In the left lane, however, the shift of about 1.4 ft from the guide rail is quite evident. It should be remembered that the left lane with the median divider was only 9.5 ft wide compared to the 10-ft width for the paint line divider. The percentage of cars traveling in the left lane is discussed later.

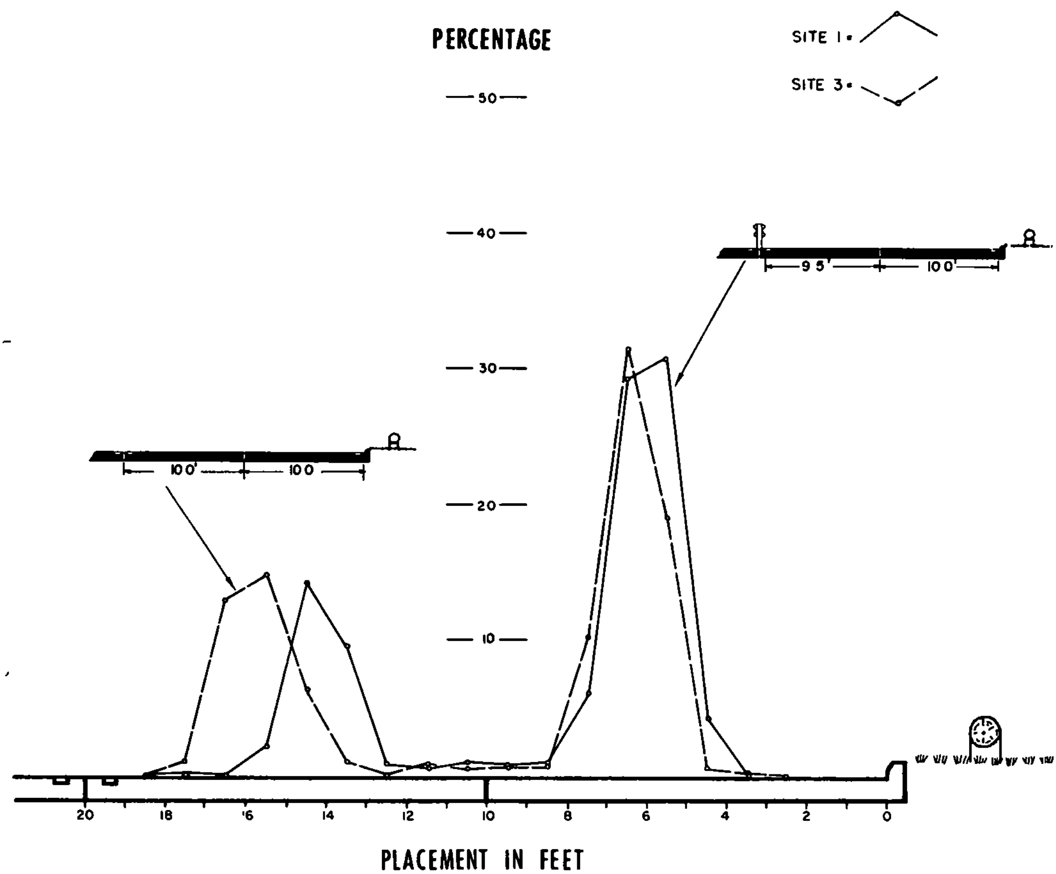


Figure 10. Distribution of lateral positions of cars on Cross County parkway for tangents with paint centerline and guide rail median.

Figure 11 shows the distribution of the lateral positions of centers of cars on the horizontal curve sections (sites 2 and 4) of the Cross County. These data show the relative effect of the guide rail median on placements both in the curb and left lanes on horizontal curves. In negotiating the curve, the drivers in the curb lane appear to shift towards the inside of the curve on the guide rail section (site 2), and on the paint line divider section (site 4) they appear to maintain their normal lane positions somewhat to the right. This may be accounted for by the relative sharpness of the two curves (7^0-15' vs 3^0-15') and the direction of the curves, left at site 2 and right at site 4. However, this relative shift towards the inside of the curve is not evident in the left lane of the site 2 section. In fact, here the cars have shifted away from the guide rail on an average of more than ½ ft as compared to the paint line divider site 4 section. It is noted that this change in position is about equal to the space occupied in the lane by the guide rail median. Moreover, the pattern of traffic appears to be more concentrated in the center of the lane with the guide rail median indicating the influence of the median as a guide for positioning of cars on curves.

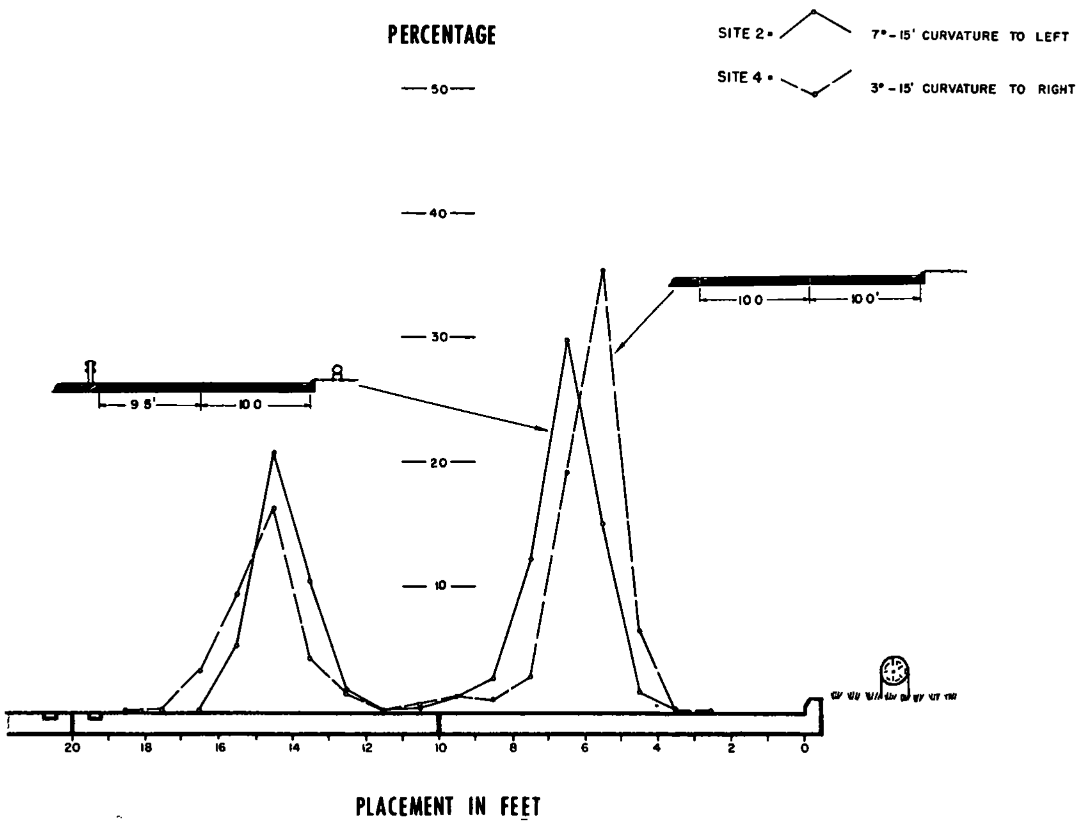


Figure 11. Distribution of lateral positions of cars on Cross County Parkway for horizontal curves with paint centerline and with guide rail median.

Figure 12 shows the distribution of the lateral positions of centers of cars for the tangent sections (sites 5 and 8) studied on the Hutchinson River Parkway and Figure 13 shows these data for the horizontal curve sections (sites 6, 7 and 9).

Site 5 is on the section having a 4-ft wide grass median, whereas site 8 has a 4-ft wide concrete median, both with low curbs. The roadway was striped for 12-ft wide lanes at both sites but at site 8 the slabs were built in three 8-ft wide sections with two construction joints each 4 ft on either side of the centerline of roadway, whereas for site 5 the slabs were built each 12 ft wide with the joint at the centerline of the pavement. From visual observation at site 8, the drivers, in general, appeared to straddle the joints in travel rather than be guided by the lane lines, thus moving the average positions of cars at site 8 to the right (about 1 ft) as compared to those shown for site 5 (Fig. 12).

This two-joint type of pavement construction was also present at the 2°-30' curve to the left with concrete median (site 9), and the resulting shift of positions of cars to the right is evident in comparison of data with the 4°-15' curve to the left with grass median (site 6, Fig. 13) which was constructed with one pavement joint, as was the 7°-00' curve to the left (site 7).

The location of the construction joints appears to account for the difference in lateral positions of cars on both the tangent (site 5 vs 8) and the horizontal curve (site 6 vs 9) sections rather than the type median. The guide rail installed on the grass median (site 7) appears to have little effect on the distribution of lateral position of cars. This is evident in the comparison of the graphs for sites 6 and 7 (Fig. 13).

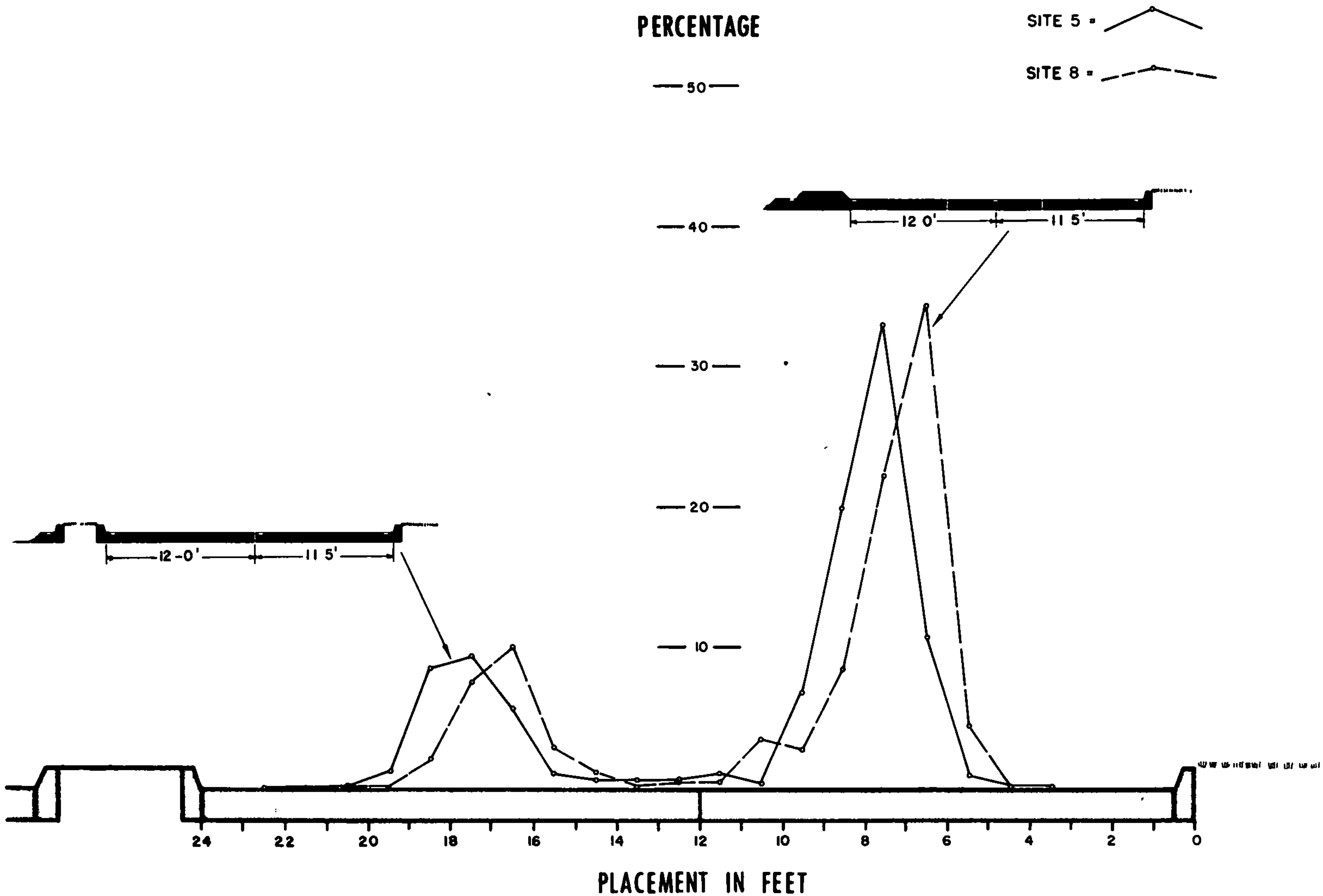


Figure 12. Distribution of lateral positions of cars on Hutchinson River Parkway for tangents with 4-ft wide raised grass median and with 4-ft wide raised portland cement concrete median.

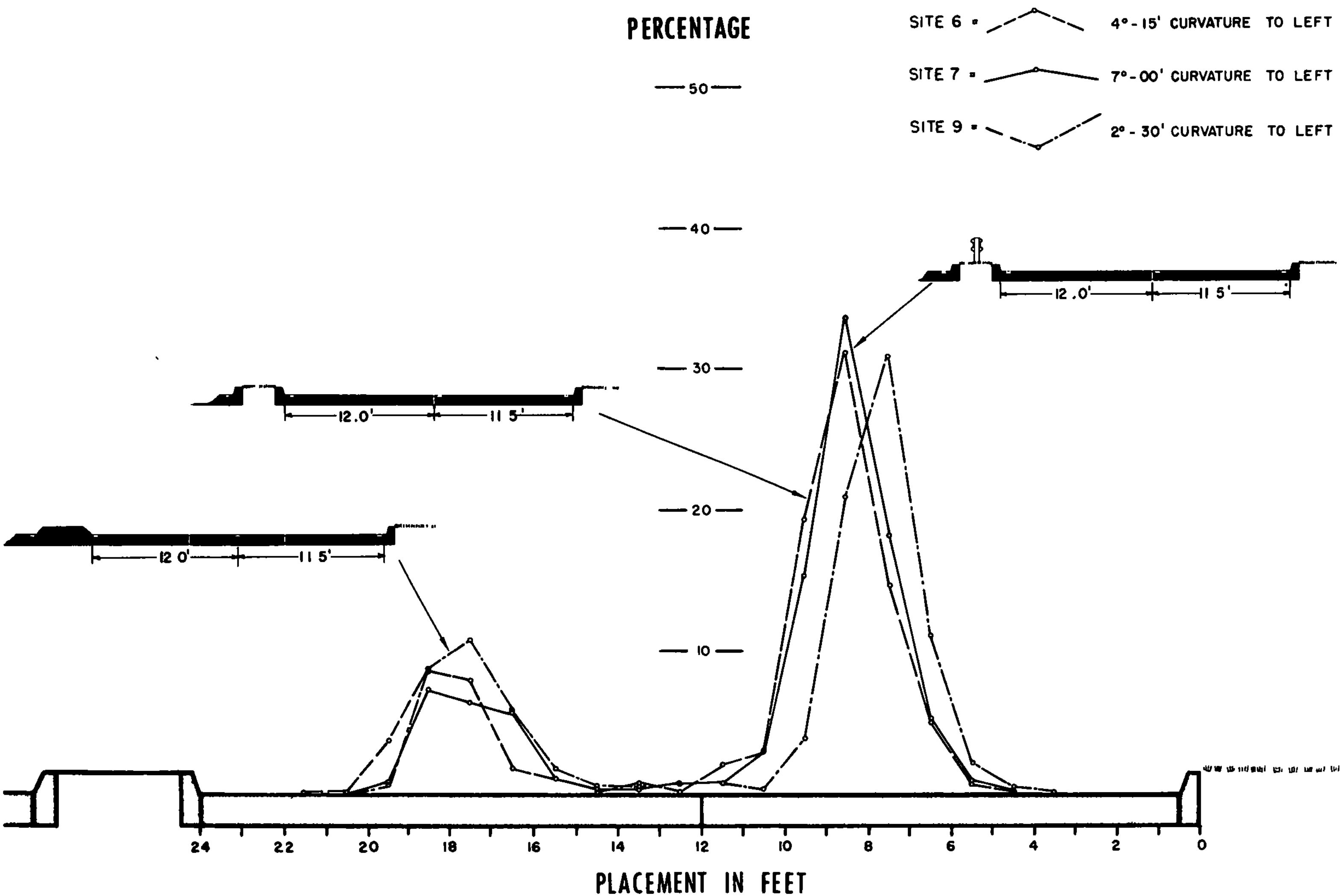


Figure 13. Distribution of lateral positions of cars on Hutchinson River Parkway for horizontal curves with raised 4-ft wide grass, grass with guide rail, and portland cement concrete median.

Lane Use

A better understanding of the effect of the median divider should be possible from the study of the percentage of cars traveling in the left or median lane. For this analysis a volume of 800 to 1,200 cars per hour in one direction at all the sites was used. Figure 14 shows the comparison.

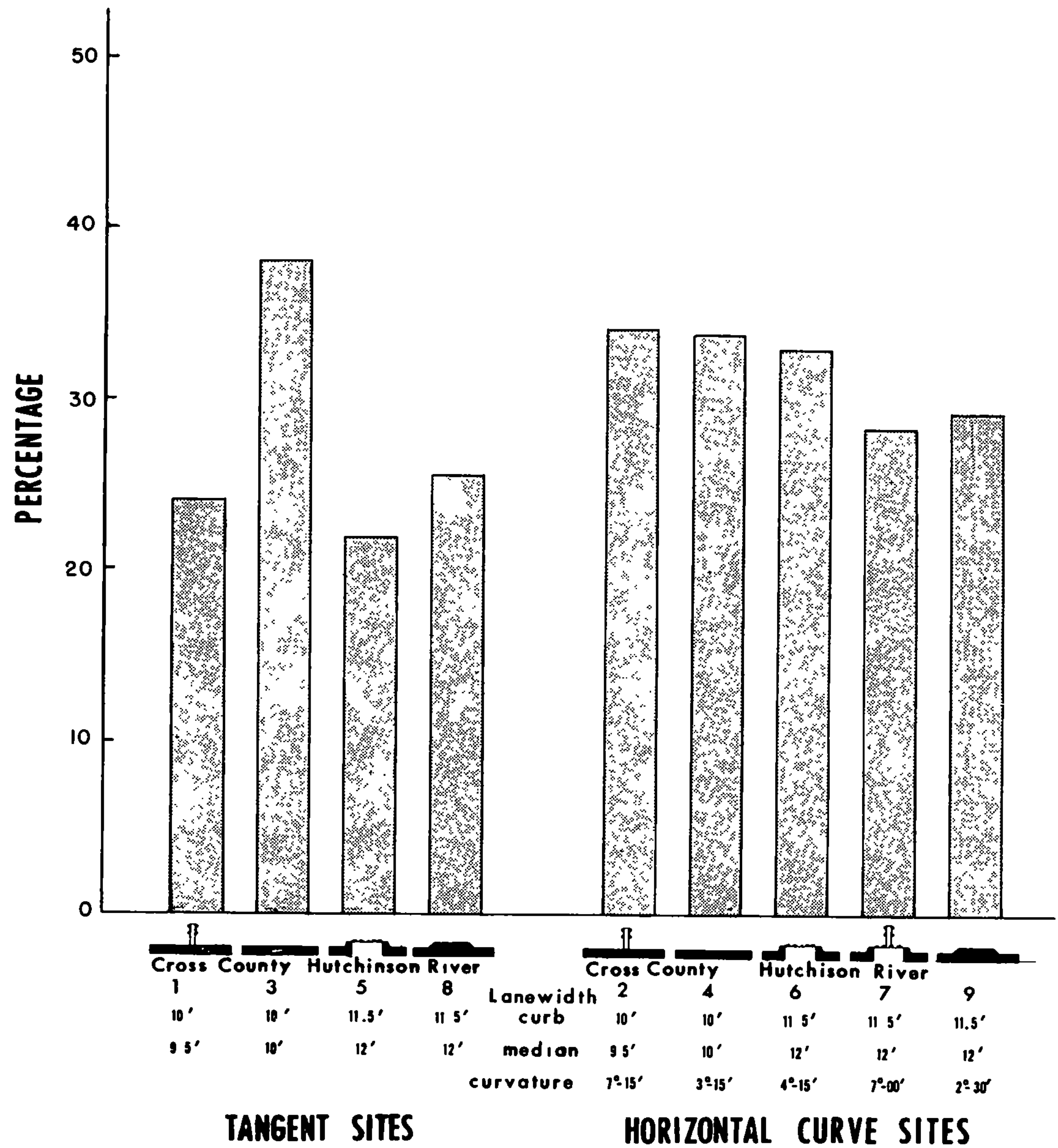


Figure 14. Percentage of cars traveling in the median lane, when traffic volumes were 800 to 1,200 cars per hour, in observed direction of travel.

For the Cross County, it will be noted that for an identical volume fewer cars used the left lane on the tangent with the guide rail median (site 1—24 percent) than with the painted line divider (site 3—38 percent). On the horizontal curves, however, no difference in the percentage of cars using the left lane was noted (sites 2 vs 4).

For the Hutchinson, the percentage of cars using the left lane was about the same for both tangent sections having the 4-ft wide median separators (sites 5 and 8). In fact, about the same percentage of cars traveled in the left lanes with the 4-ft wide grass and concrete medians as with the guide rail median (site 1) even though the roadway with the 4-ft wide medians had 12-ft wide lanes, whereas the guide rail median roadway had only 9½-ft wide lanes. It would appear from this study, at least, that the percentage of cars using the left lane does not depend on the lane width. This is even more obvious on the sections for horizontal curves (sites 6, 7, and 9 vs site 2).

When a car straddles both lanes of traffic, it occupies twice as much space as when it is within its own lane. A small percentage of cars straddling both lanes, therefore, should be an indication of the proper utilization of the highway facility and traffic control. Because the percentage of cars straddling both lanes is related to the traffic volume, a comparison of these percentages at the nine study locations was made for a volume of 800 to 1,200 cars per hour in the direction of travel studied (Fig. 15).

For the Cross County, the guide rail median on the tangent section had a smaller percentage of cars straddling the two lanes than the painted line divider (site 1 vs site 3). On the horizontal curves, however, the percentage of cars straddling the two lanes was considerably greater with the guide rail than with the painted line divider (site 2 vs 4). Thirty-three percent of the cars were straddling the two lanes with the guide rail median, whereas only nine percent did so with the painted line divider. In other words, it appears that the guide rail on the horizontal curves tends to promote single lane use of a two-lane facility. (Although the sharper curvature at site 2 (7°-15' to the left) as compared to that at site 4 (3°-15' to the right) undoubtedly was a contributing factor in increasing the percent of straddling, unfortunately it was not possible to isolate the effect of the guide rail from that of the curvature.) At the heavier volumes this condition is considerably improved, as might be expected (data not shown in Fig. 15.)

On the Hutchinson River Parkway the lowest percentage of straddling cars was found on the sections having the concrete median, sites 8 and 9. This again, reflects the influence of the offset construction joints on lateral positions of cars.

Car Clearances

With a constant pavement width and little variations in the average positions of cars from the right edge of the pavement, the choice of a median type as it affects the positions of cars from the edge of the median becomes important in providing desirable clearances between cars.

To determine the relation between the type of median and the positions of cars from the edge of the median, lateral placement data were tabulated for each of the nine study sites to indicate the percent of cars traveling in the median lane with 1 ft or less, 2 ft or less and 3 ft or less clearances between the left side of bodies and either the center of the painted line divider (sites 3 and 4) or the edge of the median divider (Fig. 16).

On the Cross County, at the tangent site 3, more than one-third of the cars traveled in the 10-ft wide median lane with 3 ft or less clearances from the center of the stripeline. The 3°-15' curve to the right at site 4 apparently had the effect of reducing this number slightly (29 percent). As expected, it is noted that of all sites, only these two showed some cars (14 percent at site 3 and 3 percent at site 4) traveling 1 ft or less from the paint line divider. At the guide rail median tangent (site 1), only 15 percent of the cars traveled in the 9½-ft wide median lane with 3 ft or less clearances from the edge of the divider. The 7°-15' curve to the left at site 2 apparently had the effect of increasing this number to 25 percent. Obviously, and particularly at the tangent site (site 1 vs 3), the guide rail median had the effect of causing cars in the median lane to increase their distance from it. (The impact of this effect on average clearances between cars is discussed later.)

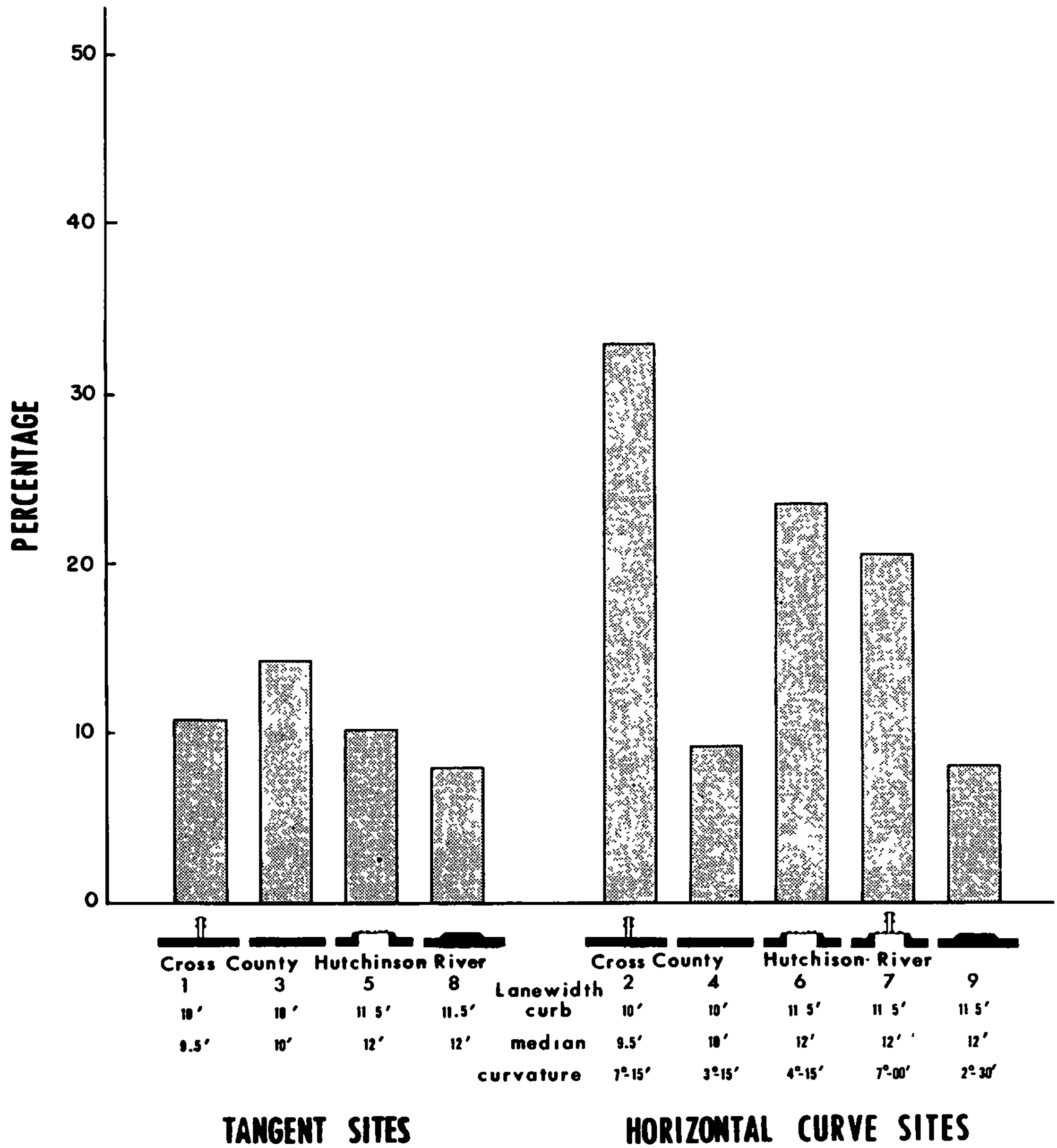


Figure 15. Percentage of cars straddling lane lines at a volume of 800 to 1,200 vehicles per hour in one direction of travel during daytime.

For the Hutchinson River Parkway with 12-ft wide lanes, the percent of cars traveling in the median lane at these close distances to the median divider were considerably smaller (between 2 and 12 percent—sites 5, 6, 7, 8 and 9) than on the Cross County (between 15 and 25 percent—sites 1 and 2). The guide rail median on the 7°-00' curve to the left at site 7 had a small but somewhat significant effect in causing the cars to move further away from it (4 percent—site 7 vs 6).

The average clearance between the bodies of cars traveling in adjacent lanes for each of the sites is shown in Figure 17. Of significance are the clearances on the 19.5-ft wide roadways at the guide rail median sites on the Cross County which are smaller (2 ft at sites 1 and 2) than those at the paint line divider sites (3.3 ft at sites 3 and 4).

In fact, the clearances between car bodies at the Cross County guide rail median sites are the only ones in the group that are less than the 3-ft minimum lateral clearance between cars desirable for driver comfort and safety (5). These small clearances apparently accounted for the 25 percent increase in the sideswipe in-the-same direction

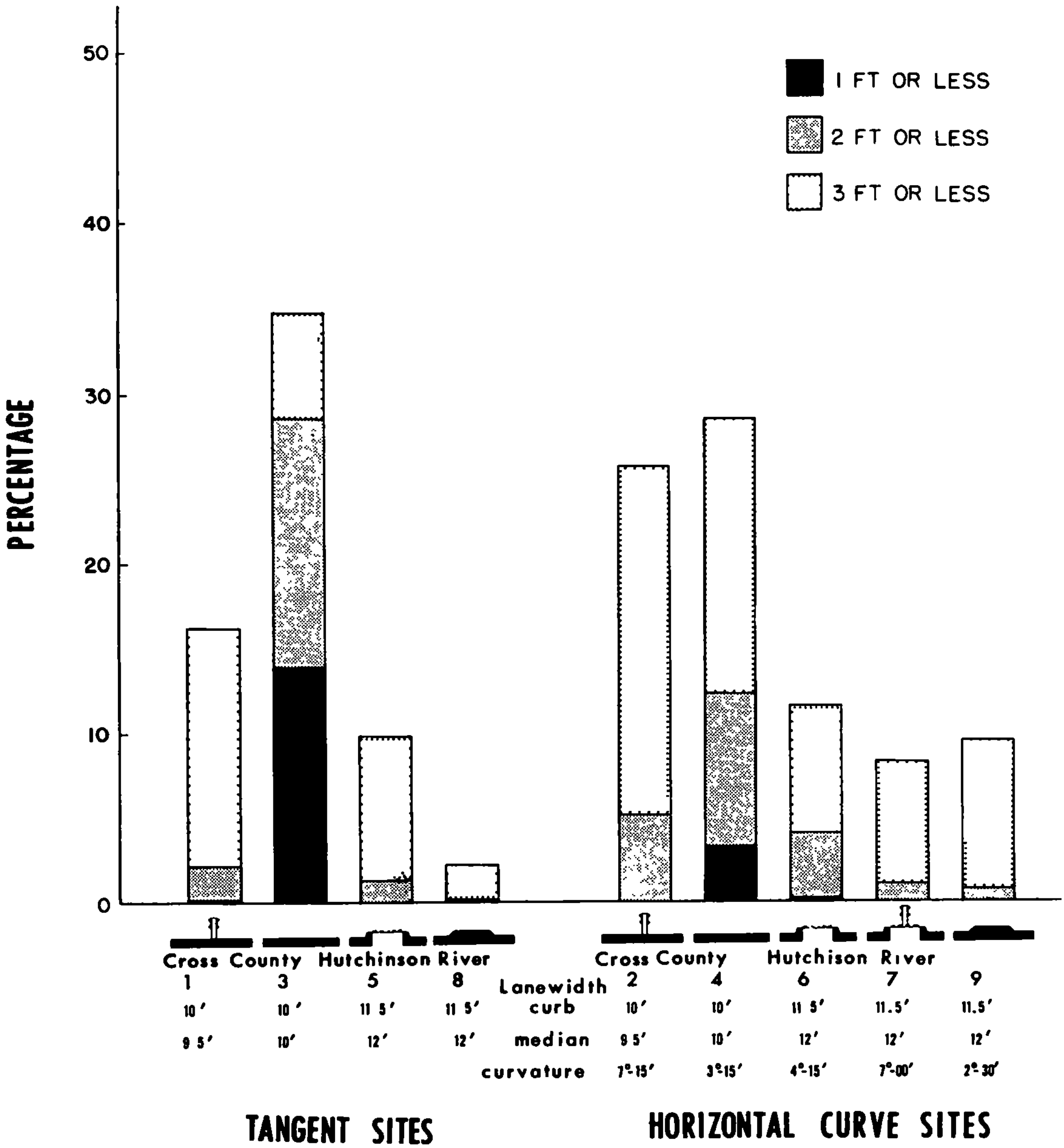


Figure 16. Percentage of cars in the median lane with various clearances between their bodies and the centerline of painted line divider or edge of median during daytime.

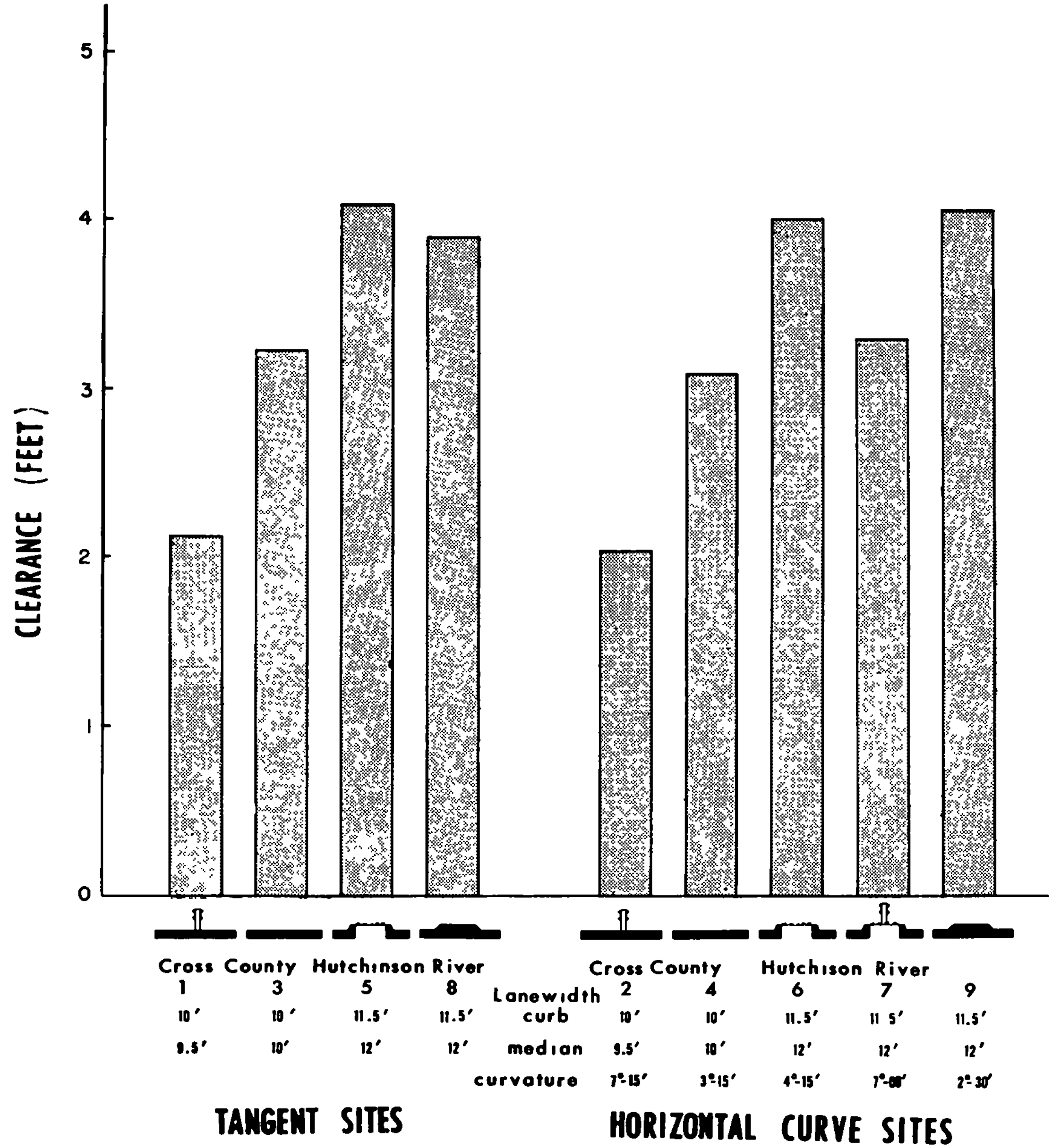


Figure 17. Average clearance between bodies of adjacent cars during daytime.

type of accident, for the "after median guide rail installation accident study," which is discussed later.

On the Hutchinson, there do not appear to be any significant differences in these clearances between those sections with the grass and concrete dividers (site 5 vs 8 and 6 vs 9) and they appear adequate for driver comfort and safety.

ACCIDENT EXPERIENCE

While the foregoing traffic performance analysis of operational characteristics is a means to determine relative safety and comfort provided the traveling public by the facility, accident experience provides the end result. All accidents of record readily available from the Westchester County Parkway Police were examined for both the Cross County and Hutchinson River Parkways.

The data for the Cross County Parkway were tabulated to indicate accident experience before and after the installation of the median guide rail. The median guide rail was constructed in two sections which were used as a basis for travel sections. The first section was constructed in the summer of 1958 and the second section, after the driver behavior study, in the spring of 1959. The first section contained study sites 1 and 2 and the second contained study sites 3 and 4.

The data for the Hutchinson River Parkway were compiled for three travel sections. The first section contained study sites 5 and 6, the second site 7 and the third, sites 8 and 9. These data were representative of the accident experience for the three types of medians examined, the 4-ft wide raised grass median, the same median with a median guide rail, and 4-ft wide raised and concrete paved median. Low curbs were an integral part of each of these medians.

The control data for this accident investigation are given in Table 6.

TABLE 6

CONTROL DATA FOR ACCIDENT STUDY

Parkway and Type Median	Travel Section Containing Sites No	Length Travel Section (mi)	Period of Study (days)	Travel (MVM)	Remarks
Cross County Parkway					
Paint line (before)	1 & 2	1 31	558	32	
Same	3 & 4	0 90	856	33	
Median guide rail (after)	1 & 2	1 31	365	21	Median guide rail installed 8/11/58-10/9/58
Same	3 & 4	0 90	100	4	Median guide rail installed 4/5/59-8/1/59[a]
Hutchinson River Parkway					All medians 4 ft wide and curbed
Raised grass (Type 1)	5 & 6	1 2	608	18	
Raised grass w/median guide rail (Type 2)	7	0 23	852	5	One section of median guide rail 1,217 ft long on horizontal curves installed 8/57
Raised and paved (Type 3)	8 & 9	3 5	608	64	

[a]Not included in the driver behavior study sites

Cross County Parkway

Table 7 gives the Cross County Parkway accident data before and after installation of the median guide rail for the two travel sections combined. Based on accident rates and disregarding "hits on the divider," these data, in parenthesis, indicate that the median guide rail in addition to eliminating fatal accidents reduced the personal injury, property damage and total accident frequency each by one-third. On a total basis, the sideswipe in opposite direction, the head-on and the hit animal types of accidents were eliminated and the frequency of all other categories of accidents were reduced by about one-fifth. The sideswipe in the same direction type of accident increased by one-fourth and was the only type of accident which showed an increase in frequency. It is to be noted that there were two overturning accidents after the installation of the median guide rail and they were in both cases the skid-hit-curb-overturn cases. There was no median guide rail involvement in either of these accidents. Examination of the personal injury rates shows that the head-on, angle and sideswipe opposite direction accidents were eliminated but there was a 40 percent increase in the frequency of the rear-end and a one-third increase in the hit fixed object types of accident.

TABLE 7

CROSS COUNTY PARKWAY ACCIDENT EXPERIENCE BEFORE AND AFTER INSTALLATION OF GUIDE RAIL MEDIAN BY TRAVEL SECTIONS CONTAINING STUDY SITES[a]

		Paint Lines - Before[b]						Median Guide Rail - After[c]					
		Personal Injury		Property Damage		Total		Personal Injury		Property Damage		Total	
Study Site No	Type Accident	No	Rate[d]	No	Rate	No	Rate	No	Rate	No	Rate	No	Rate
1, 2, 3 and 4	Rear end	13	20	63	97	76	117	7	28	17	68	24	96
	Sideswipe, opp	5	8	24	37	29	45	0	0	0	0	0	0
	Sideswipe, same direction	4	6	15	23	19	29	1	4	8	32	9	36
	Fixed object	6	9	16	25	22	34	3	12	4	16	7	28
	Hit divider[e]	0	0	0	0	0	0	UK	UK	267	1,068	267	1,068
	Overturn	0	0	0	0	0	0	0	0	2	8	2	8
	Head-on	8	12	2	3	10	15	0	0	0	0	0	0
	Angle	5	8	13	20	18	28	0	0	6	24	6	24
	Hit animal	0	0	9	14	9	14	0	0	0	0	0	0
	Unclassified	1	2	2	3	3	5	0	0	0	0	0	0
										(167)[f]	(148)[f]	(48)[f]	(192)[f]
	Total	42	65	144	222	186	287	11	44	304	1,216	315	1,260

[a]The "before period" for travel section containing study sites 1 and 2 from 1 January '57 - 10 August '58 and the "after period" from 9 October '58 - 9 October '59 Median guide rail was installed between these periods, 1 31 mi in length The "before period" for travel section containing study sites 3 and 4 was from 1 January '57 - 4 April '59 and the "after period" from 31 July '59 - 7 November '59 Median guide rail was installed between these periods, 0 90 mi in length
[b]There were ten fatalities during the study period for a rate of fifteen per 10^8 mi of travel
[c]There were no fatalities during this period of study
[d]All rates per 100 M vehicle-miles of travel (10^8).
[e]Brush hits by cars on median guide rail expanded from a count for four months of operation in 1960 equals 180 per mile year
[f]Figures in parenthesis excludes brush hits

This pattern of improvement in safety with the median guide rail is to be expected. However, it should be noted that although inspection showed that there were a good many "brush collisions" with the median guide rail (estimate 180 per year per mile), in fact, some severe enough to permanently deflect the rail or flatten its corrugations, none have appeared in the police records.

These "brush hits," recognized as often the driver's own fault, did not appear in the police records principally by reason of the fact that in New York State accidents under $100 property damage are not required by law to be reported. Furthermore, if the drivers hesitated or stopped after this maneuver they would be exposed to a more severe accident. In addition to the economic loss in property damage caused by these "brush hits" there is the cost of repairing and replacing the damaged guide rail.

In passing, it should be emphasized that there were ten fatalities during the study period before the installation of the median guide rail and none after. There was no information available to determine the effectiveness of the median guide rail on accident frequency separately for the horizontal curves and the tangents.

Hutchinson River Parkway

For the purpose of comparison the raised grass is referred to as Type 1, the raised grass with median guide rail as Type 2, the raised concrete paved median as Type 3. Examination of the data in Table 8 indicates that the total accident frequencies were 150 and 100 per 10^8 mi of travel, respectively, for the travel sections containing Types 1 and 2 medians. The frequency of total accidents for the travel section containing the Type 3 median averaged about 90 percent greater (236). The travel section with the Type 1 median showed the least personal injury frequency and was more than 50 percent less than those for the sections with the Types 2 and 3 medians. The Type 2 median section had the better property damage accident frequency (60) being less than one-half that for the Type 1 median section (129).

Further examination of these accident frequencies by type of accident showed that under total accidents the travel section with the Type 1 median had the best record for all types of accidents except the angle and hit animal types. It should be noted that there were no fatalities on the travel sections during the study period and also, all the accidents of record for the Type 2 median section involved the median guide rail.

From this analysis it appears that the travel section with the Type 1 or the 4-ft raised grass median had a significantly better over-all accident experience for the period of study.

In an attempt to explain the reasons behind the better accident experience of raised grass as compared to the raised paved median, it should be noted that many variables other than the median type and design influence the frequency of accidents. Among these are exposure in vehicle-miles of travel, design standards, traffic density, climatic conditions, speed differentials and many others. Obviously, not all of these variables could be controlled in this kind of an accident study.

TABLE 8

HUTCHINSON RIVER PARKWAY ACCIDENT EXPERIENCE FOR VARIOUS TYPE MEDIANS BY TRAVEL SECTIONS CONTAINING STUDY SITES FOR PERIODS SHOWN

Study Site No	Type Accident[a]	Personal Injury		Property Damage		Total		Accidents Per Mile
		No	Rate[b]	No	Rate	No	Rate	
(a) 4-Ft Wide Raised Grass Median with 3-In High Curb—Type 1 (1 Jan 1958 to 1 Sept 1959)								
5 and 6	Rear end	0	0	10	54	10	54	8
	Sideswipe	1	5	3	15	4	20	3
	Fixed object	0	0	5	27	5	27	4
	Head-on	1	5	0	0	1	5	1
	Angle	2	11	4	22	6	33	5
	Hit animal	0	0	2	11	2	11	2
	Totals[c]	4	21	24	129	28	150	23
(b) 4-Ft Wide Raised Grass Median With Median Guide Rail and 3-In High Curb—Type 2 (1 Sept 1957 to 1 Jan 1960)								
7	Rear end	0	0	0	0	0	0	0
	Sideswipe	0	0	0	0	0	0	0
	Fixed object	2	40	3	60	5[d]	100	22
	Totals	2[e]	40	3[f]	60	5	100	22
(c) 4-Ft Wide Raised and Concrete Paved Median With 6-In High Curb—Type 3 (1 Jan 1958 to 1 Sept 1959)								
8 and 9	Rear end	17	26	61	95	78	121	22
	Sideswipe	2	3	16	25	18	28	5
	Fixed object	3	5	24	38	27	43	8
	Head-on	3	5	1	2	4	7	1
	Angle	6	10	8	13	14	23	4
	Hit animal	0	0	3	5	3	5	1
	Single car	0	0	1	2	1	2	-
	Overturn	0	0	3	5	3	5	1
	Pedestrian	1	2	0	0	1	2	-
	Totals	32	51	117	185	149	236	43

[a]No fatalities during this period
[b]All rates per 100 M vehicle-miles of travel (10^8).
[c]No overturn, single car or pedestrian types of accidents during this period
[d]All accidents during this period involved the median guide rail
[e]One accident collision with guide rail and overturned, five persons injured One accident jumped guide rail and overturned, one person injured
[f]Does not include numerous unreported brush hits by cars on median guide rail

Exploring the effect of volume on these accident rates, it is recognized that hourly traffic volumes are a more accurate indication of the operating conditions and degree of congestion than the average daily traffic flow. However, because of obvious difficulties in relating accident rates to hourly flow, the contribution of volume to these accident rates may be approximated by using the average daily traffic volume.

The average annual daily traffic for the Type 3, or raised and paved, was 30,300, and for the Type 1, or raised grass median, 25,400 cars. Using the effect of volume on accident rates for deterring-type medians found in another New York Study (6), this increase in volume would contribute about 25 percent of the accident rates shown for the Type 3 median. Using the total accident rate of 150 for the Type 1 (Table 8) as a base, this would give a rate of 188 for the Type 3 median accounting for the effect of volume.

The difference between the total accident rate (236) for the Type 3 median and the above calculated 188 rate or 48 might be assessed against the median design and uncontrolled variables. This is a contribution of 33 percent more than the total accident rate of the raised grass median. It should be noted here that the portland cement concrete Type 3 paved is not as conspicuous as the raised grass median especially at night and lacks the color contrast provided by the green grass.

It can be seen at a further look at the types of accidents (Table 8) that the head-on and sideswipe are directly affected by the median design. The total accident rate for these two types of accidents is 25 and 35, respectively, for the Type 1 grass and Type 3 paved medians. Here again, this rate for the paved is about 40 percent higher than that for the grass median which appears to support the relative effect of these two median designs on accident occurrence.

Westchester County Public Works Department reports (7) that for the travel section containing the raised grass with median guide rail or the Type 2 median there were six reported cross-median accidents in the seven months prior to the installation of the guide rail. These accidents, of which two involved fatalities, injured nine and killed two persons. Both of the fatal cross-median accidents involved two cars and each such accident caused three injuries and one death. For the 2⅔ yr after the installation there have been five reported accidents involving the guide rail injuring six persons in two of the accidents. Five injuries occurred as a result of the car overturning after the collision with the guide rail and one occurred when a westbound car overtopped the guide rail and overturned in the eastbound lane.

It was concluded that, "Although the sample is small these facts point tentatively to the conclusion that the consequences of median guide rail collisions may be less serious than the consequences of cross median accidents involving two cars. The case of overtopping the guide rail and that of over-turning after guide rail collision, remaining in the same roadway, confirms the conclusions reached by General Motors and California following crash tests of beam-type guide rails and barriers, that median guide rail type of dividers used here are inadequate to perform their function to the expected degree." (7).

SUMMARY OF RESULTS

Traffic Performance

The data for this study were recorded on the Cross County and Hutchinson River Parkways in Westchester County. The study sites were located on tangents and horizontal curves of various radii both generally typical of the alinement and grade of the Parkways. This summary of results pertains only to these study sites and should not be otherwise construed.

The data appear to indicate that on a 40-ft roadway a guide rail median appears to have certain merits on horizontal curves. On the curves with the guide rail, drivers reduce their speed, tend to use the lanes more effectively and in general promote more orderly traffic operations. On tangents, however, no comparable benefits appear to be derived from installing the guide rail median on these narrow sections of highway. There does not appear to be any significant differences in the driver behavior pattern as measured in this study on the Hutchinson River Parkway between the sections having the guide rail median and those not have the guide rail.

Accident Experience

The installation of the double-faced beam type of guide rail median on the Cross County Parkway reduced the reported total accident frequency by about one-third for the period of study. Fatal, head-on and sideswipe in the opposite direction types of accidents were eliminated, whereas all other types were significantly reduced except the sideswipe in the same direction type which increased about 25 percent.

Unreported "brush collisions" with the median guide rail appeared to be generated at the rate of one per day for each 2 mi of guide rail median.

The travel section on the Hutchinson River Parkway with the 4-ft raised grass median had a significantly better accident experience for the period of study than either of those sections with the 4-ft raised and paved or the 4-ft raised grass with guide rail medians.

It appears that the installation of the guide rail on the 4-ft raised grass median influenced the reduction of accidents except those involving the guide rail and reduced the consequences of cross-median accidents but is inadequate to perform its function to the expected degree.

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Median Accident Study—Long Island, New York

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● LONG ISLAND comprises a land area 15 to 20 mi wide extending 125 mi easterly from New York City. The principal east-west traffic movements are limited by the geography of the Island. The highway transport facilities include parkways for passenger cars and State, county and local highway systems for mixed traffic.

Since the early thirties, the demands of high volume urban traffic throughout the Island on the various highway systems has resulted in extensive construction of divided highways incorporating various median designs in an effort to bring about an improvement in safety and comfort to the users.

Divided highways have demonstrated their ability to carry large volumes of traffic efficiently and safely. However, the most modern highway does not prevent all accidents. This results in a continuing demand for improved design and increased safety.

One of the major questions with respect to safety is the proper design of medians to meet various operating conditions. Varying terrain and high cost of right-of-way in urban areas have led to a variety of median designs.

The purpose of this study is to investigate the effect of median design on accident rates for divided urban highways with roadside development.

Three previous studies have been made on this subject. (1) California in 1953 investigated medians on 4-lane rural high-speed highways and also semi-urban highways. For the rural study, intersections, speed zones and sections with roadside development were excluded. For the semi-urban study, restricted speed zones and roadside development were included. (2) Fred W. Hurd, Yale Bureau of Highway Traffic, during the summer of 1954, studied accident experience with traversable medians of different widths on limited-access high-speed highways. (3) California in 1958 studied accident experience on freeways for both the deterring and non-traversable types of median under high volumes.

DESCRIPTION OF STUDY SECTIONS

Detailed records of accidents for the years 1955 through 1959 were collated for 34 sections of State and county highways in Long Island, New York. These sections consisted of 82 mi of urban divided, multilane, free-access (no control of access) highways. Traffic volumes up to 44,000 vehicles per day were represented. In addition, one 4.3-mi section was studied on a 6-lane limited-access expressway carrying more than 85,000 vehicles per day. The 86 mi studied included more than 64 percent of the highway mileage with medians on Long Island, but excluding parkway mileage.

The 35 study sections were representative of 4- and 6-lane highways with concrete and bituminous macadam pavements constructed during the period 1930-1958. The 86 mi studied comprised sections from 0.5 to 9.3 mi in length with an average length of 2.7 mi. Thirteen sections were representative of the grass and flush type median 10 to 54 ft wide; 17 were of the raised grass and curbed type, 6 to 54 ft wide; and 5 were of the raised paved and curbed type, 2 to 8 ft wide. In three of the sections with grass medians, intermittent shrubbery had been planted. Four of the grass median sections were equipped with positive median devices consisting of (a) double steel-beam-type guide rail with steel posts, (b) concrete posts, (c) single-beam-type guide rail with concrete posts, and (d) a 1 on 2 deep ditch slope with concrete post and wooden guide rail at the edge of the median shoulder.

The designs of the various sections included various combinations of left-turn lanes, crossovers, paved, stabilized and compacted median shoulders, highway illumination and other miscellaneous features. There were a total of 650 intersections in the study, and with the exception of 13, all were at grade. The number of intersections per study section varied from 3 to 62 or from 3 to 7 per mile.

PROCEDURE

Median Groupings

The median designs were classified in two general categories by functional type.

Deterring Type.—This type, by an obstruction, discourages deliberate entrance or crossing of the median. The flush and raised grass with intermittent shrubbery, the mountable double curb, and the earth medians with flat cross-slopes are in this group.

Non-Traversable Type.— This type, by a physical obstruction, would prevent crossing from one roadway to another without a reportable accident. Medians with a continuous obstruction (positive median device), those with concrete posts to prevent crossings and earth medians with steep cross-slopes are included in this group. Highways with separate roadways and median greater than 120 ft in width are also generally classified as non-traversable; however, there were none of these in this study.

In the study there was no mileage of the traversable-type medians such as paved medians or an earth median with a flat, smooth, or hard surface.

The deterring types of median were further subdivided for examination into earth, curbed and medians with miscellaneous features. The non-traversable types of median were divided into four sub-groups according to the type of positive median device. Typical study sections for these deterring and non-traversable groups are shown in Figure 1.

A summary of the mileage, number of accidents and related data studied by functional type of median is given in Table 1.

Accident Data Collection

The New York State Department of Public Works receives weekly reports of all accidents from each police department having jurisdiction. (In New York State all accidents of more than $100 property damage are by law reportable. More than 99 percent of reportable accident occurrence is represented by the data.)

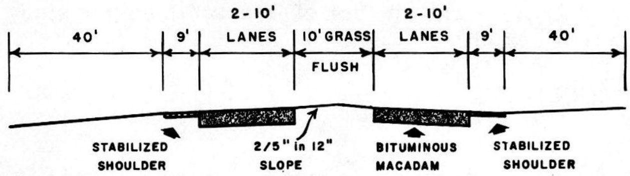
These reports are verified and checked against accident reports from the Motor Vehicle Bureau, and newspaper clippings.

Accident Data.—For the 5-yr period of study there were 1,552 accidents of record which occurred between intersections and 6,628 at the 650 intersections for a total of 8,180 accidents. Each accident was given a consecutive number by study section. Data for each accident were tabulated to indicate hour, day, month, year, location on study section, direction that vehicle involved was traveling, type of accident (27 categories), number injured and killed, kind of locale, the number of travel lanes, marginal friction and dimensions of medians and shoulders. This information was coded and placed on IBM cards for electronic data processing.

The types of accidents between intersections were classified as:

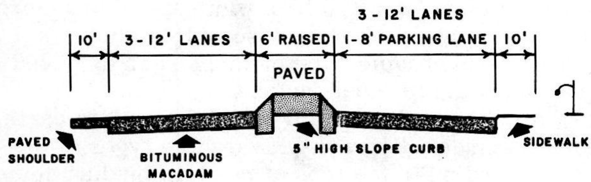
1. Approach.—Cross median collision, right or left turn head-on, collisions and other head-on collisions.
2. Overtaking.—Rear end, sideswipe same direction and opposite direction, right angle collision, collisions with cars turning left or right, skid collision, U-turn and improper right or left turns.
3. Single Vehicle.—Hit post, pole, tree, parked car, bridge abutment or animal, sideswipe median, cross median no collision and car turned over.
4. Pedestrian.
5. Other.—Other personal injury, hit bicycle, accidents caused by road construction.

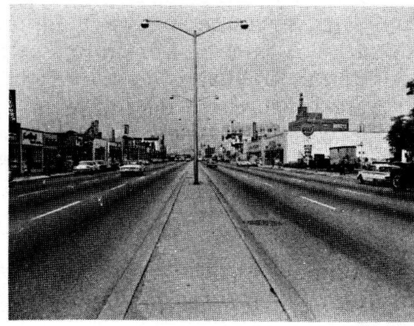
Individual site data were tabulated for control, identification of traffic and travel, fatal, personal injury and property damage accidents occurring between intersections and for accidents occurring at intersections for night, daytime and total. These tables with EDP tabulations formed the basis for examination of accident data by median groupings.



DETERRING MEDIAN

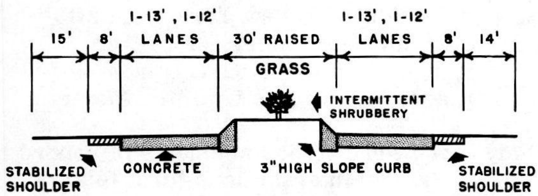
GROUP 1

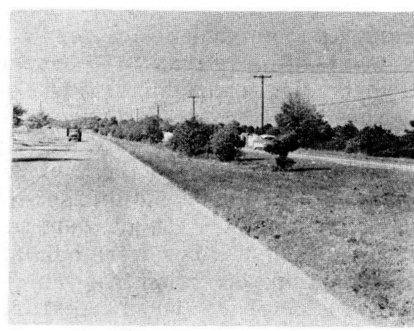




DETERRING MEDIAN

GROUP 2

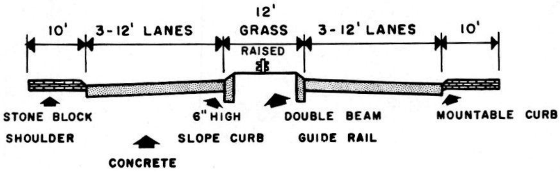




DETERRING MEDIAN

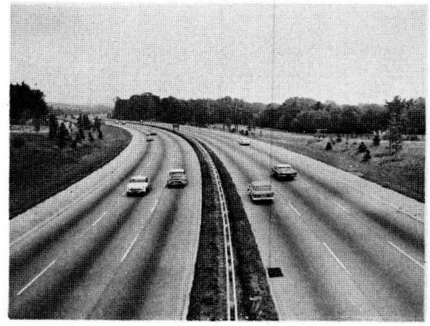
GROUP 3

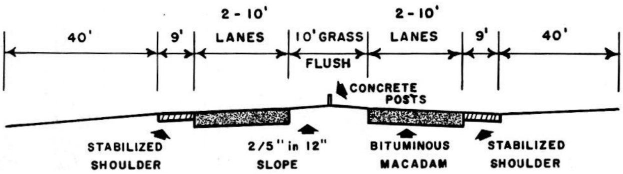
Figure 1. Typical cr



NON TRAVERSABLE MEDIAN

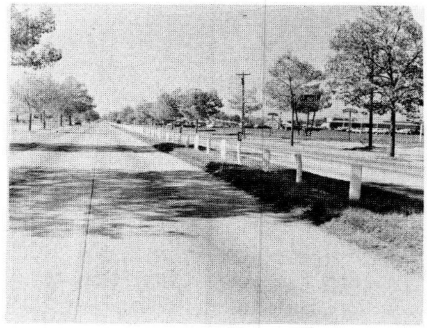
GROUP 1

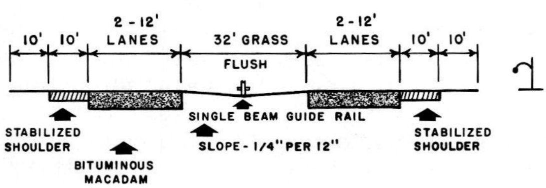




NON TRAVERSABLE MEDIAN

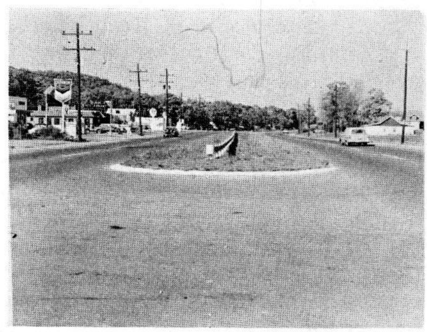
GROUP 2

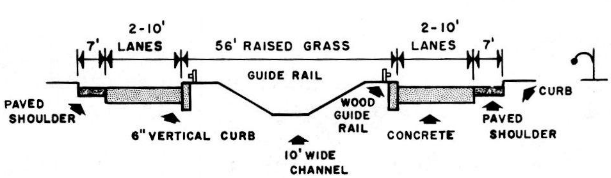




NON TRAVERSABLE MEDIAN

GROUP 3





NON TRAVERSABLE MEDIAN

GROUP 4

on and view of sites.

TABLE 1

SUMMARY OF MILEAGE AND NUMBER OF ACCIDENTS STUDIED BY FUNCTIONAL TYPE OF MEDIAN

Functional Type of Median	No. of Sections	Total Length (mi)	Study Period	No of Reported Accidents Between Intersections	At Intersections	Travel MVM for Period of Study
Deterring						
Earth grass or soft surface, slopes 1 on 4 or flatter	10	45.8	1955-60	406	1,325	893 1
Curbed. with standard curbs 6 in or less in height (vertical and mountable)	19	26 3	1955-60	860	4,607	780 7
Miscellaneous Features median with intermittent shrubbery, curbed and flush	2	3 6	1955-60	58	208	87 9
Sub-total	31	75.7		1,324	6,140	1,761 7
Non-Traversable						
Median, 12 ft wide and curbed with double type steel guide railing and steel posts	1	4 3	1959	35	1	127 1
Median, with concrete posts to prevent crossings	1	3 3	1955-60	184	435	132 4
Median, with NYSDPW single-beam-type guide railing and concrete posts	1	2 2	1959	5	9	7 8
Median, with concrete posts and railing and some large trees Deep ditch slope 1 and 2	1	0 7	1958-60	4	43	9.8
Subtotal	4	10 5		228	488	277.1
Total	35	86 2		1,552	6,628	2,038.8

Many variables other than median type and design apparently influence the frequency of accidents between intersections. Among these are the exposure as measured in vehicle-miles of travel, design standards and features of a particular facility, such as crossovers, left-turn lanes, illumination, parking lanes and marginal activity. In addition, there are traffic density, climatic conditions, speed differentials and many others, to say nothing about faulty driver behavior.

Obviously, not all of these variables can be controlled in this type of study. However, based on the mileage of roadway studied, it was considered fair to assume that variables other than traffic volume and width of median were distributed throughout the various median types.

A 4- or 5-yr continuous period of accident data was obtained to provide a reasonably large sample of motor vehicle accidents to compare the influence of the various types of medians. Apropos of the foregoing, an accident occurrence of at least ten in number for the period of study was adopted as a minimum for analysis of accident rates. Rates based on less than 10 occurrences are noted on the ensuing tabulations.

The intersection accidents were excluded in the general analysis of the accident occurrence for the various grouping of medians and are shown separately in some of the analyses where appropriate.

ANALYSIS

Daytime and Nighttime Accident Occurrence

By reason of the fact that this report will be of interest locally, as well as nationally, it appears appropriate to examine the daytime and nighttime frequency of accident occurrence.

It was found that the annual average night traffic volume, for the period of the study, was about 25 percent of the 24-hr traffic volume. This was based on hourly traffic counts from four continuous counter stations strategically located on the State Highway System on Long Island.

Using this index of night travel, the following comparison is made for the median study groups between day and night frequency of all accidents between intersections occurrence.

Median	Rate[a] Day	Night
Deterring		
Earth	33	82
Curbed	85	185
Miscellaneous features	55	91
Subtotal	58	119
Non-Traversable		
With double guide railing	17	59
With concrete posts	119	194
With single guide railing[b]	0	250
With deep ditch[b]	28	74
Subtotal	41	144
Total	48	133

[a]Number of accidents for 100 MVM of travel.
[b]Number of accidents less than 10 for period of study.

The ratio of night to day accident occurrence between intersections averaged 3.5 to 1 for the non-traversable group and 2.0 to 1 for the deterring group of medians. Further analyses of day and night accident rates, including those for intersection accidents, are shown later.

The total accidents data were used as a basis for accident occurrence in the ensuing analyses.

Influence of Traffic Volume on Accident Occurrence

For the over-all safety of a highway, the median types should be investigated for all operating conditions. Hourly traffic volumes are a more accurate indication of the operating conditions and degree of congestion than the annual average daily flow. Because of the obvious difficulties in relating accident rates to hourly flow, the annual average daily traffic volume was used as a basis for comparison of volume groups. It is believed that in a large sample such as this, the study sections within the same daily volume groups will have similar hourly flow patterns, except for the very low volume groups.

Figure 2 shows the average accident rates for the deterring-type median by volume groups. From these data it will be noted that the rate for all accidents increased uniformally from 50 at about 6,000 vehicles per day to more than 150 at 40,000 vehicles per day. The effect of daily traffic volume on injury accident rates was less pronounced although it increased as volume increased. This increase does not appear to be significant.

The volume range for the non-traversable type of median was too small (17-22,000) for determining the effect of volume on accident occurrence.

Influence of Median Width on Accident Occurrence

The all and injury accidents rates by groups of medians with the same width were plotted for the two functional median types in Figure 3. The number of accidents examined for each group of widths is shown in the circles. It was felt that the sample was too small to investigate the effect of the median width for the various median subgroups within each functional type.

Based on an accident occurrence of at least 10 in number as sufficient to permit an analysis of accident rates, the sample size for the non-traversable type was too small to interpret the width relation to accident occurrence.

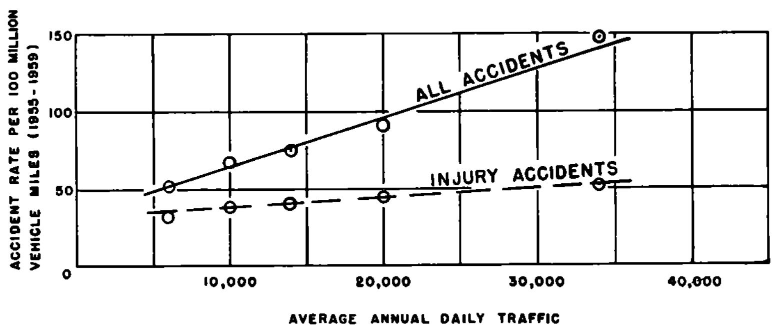


Figure 2. Effect of volume (AADT) on accident occurrence between intersections for multi-lane highways with deterring-type median.

For the deterring-type median there appeared to be no correlation between either the all or the injury accident rates and the width of medians.

As pointed out in the California Study (1), this seems to contradict the hypothesis that, for the same general conditions, the greater the lateral separation the safer the facility. In determining the optimum width of median for safety, consideration should be given to the fact that when a vehicle leaves the roadway, there is a good chance of avoiding a reportable accident if maneuvering space is available. The flush flat slope earth medians provide this space not inherent with the raised and curbed types. For the non-traversable type of median, sufficient space on each side of the positive median device should be provided for emergency stops.

Median widths in excess of 40 ft, however, apparently present an additional hazard. The accident records contain a number of incidence of fatalities and injuries which occurred at night under conditions of low visibility, due to fog or rain. Here, the median widths between 40 to 56 ft appear to have confused the drivers, and they mistakenly entered the opposing lanes of traffic in the belief they were on a two-way highway. This condition demands special signing for safety.

The effect of median width on accident occurrence between intersections was further investigated for the deterring type of median by adjusting the data for the effect of volume.

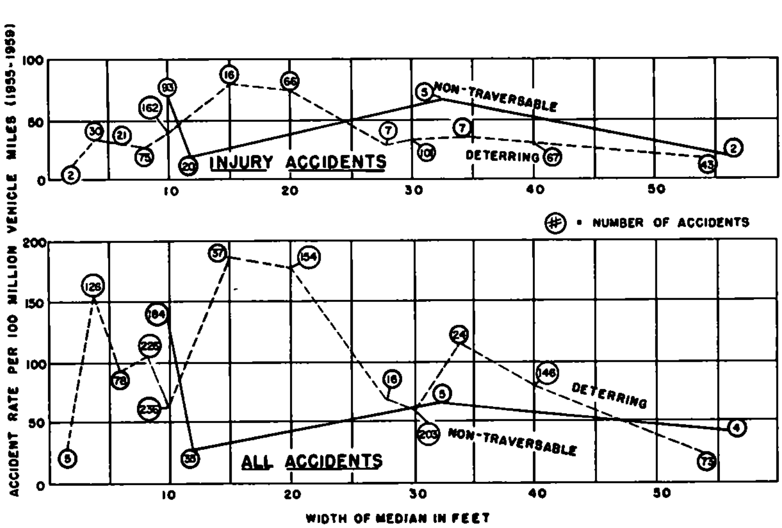


Figure 3. Effect of width of median on accident occurrence between intersections.

This relationship for all and injury accidents is shown in Figure 4. The contribution of increase in traffic volume to the all and injury accident rates was determined for each appropriate study section by the relation, as shown in Figure 2, between accident rates for the AADT of the study section (volume group) and those of the 6,000 vehicles per day group used as a base. These contributions of accident rates were deducted from the total rates and adjusted values for each width of median represented in Figure 4 were depicted.

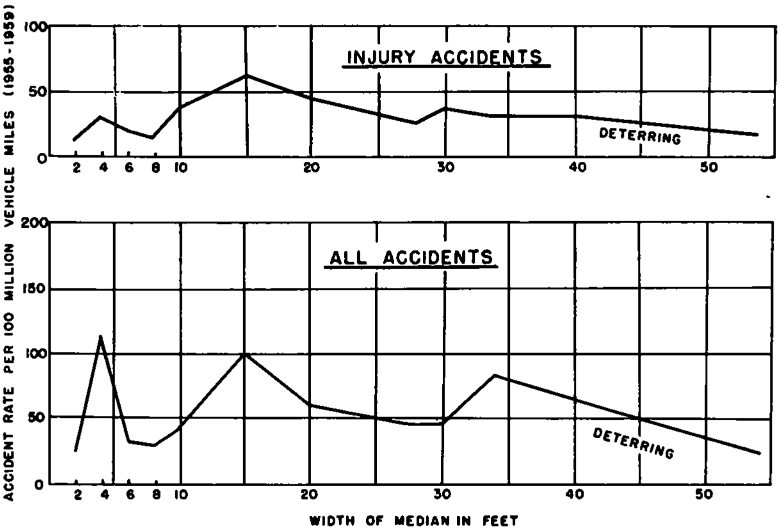


Figure 4. Effect of width of median on accident occurrence between intersections adjusted for effect of volume.

It will be noted that the pattern of this relationship for both all and injury accidents followed those shown in Figure 3 except that the peaks are smaller, but again, no consistent trend was evident. Although a decrease in accidents with increasing median width appeared between widths of 15 and 50 ft, at median widths nearer than 15 ft, however, the injury rate also decreased.

Influence of Median on Type of Accident

To investigate the influence of median type on the safety of a facility in more detail, the accident pattern by type of median for all accidents between intersections was examined. Table 2 gives the accident rates per 100 MVM by type of accident before adjustment for the effect of volume. It is noted that the overtaking type of accident accounted for more than 70 percent of all the accidents for both the deterring and non-traversable types of median. This contribution of overtaking accidents appeared to be consistent in the three sub-groups of the deterring type. For the non-traversable type, the overtaking accidents for the concrete posts accounted for more than 75 percent of the total accidents. (Concrete posts were installed in the median to eliminate indiscriminate crossings in order to gain access to regional shopping areas.)

Figure 5 depicts these data for the three subgroups of the deterring-type median. The white portion of the bars in this figure indicates the contribution of volume to accident rates. Likewise, the difference between the lower and upper numbers in each case indicates the amount of the volume contribution to rate per 100 million vehicle-miles of travel.

TABLE 2

ACCIDENT PATTERN AND RATES BY TYPE OF MEDIAN FOR ALL ACCIDENTS BETWEEN INTERSECTIONS

Type of Median	No All Accidents	Accident Rates Per 100 MVM of Travel					
		Approach	Over Taking	Single Vehicle	Pedestrian	Other	Total
Deterring							
Earth	406	2	32	9	1[a]	2	45
Curbed	860	2	78	22	5	3	110
Miscellaneous features	58	9[a]	44	10	1[a]	1[a]	66
Subtotal	1,324	2	53	15	3	2	75
Non-Traversable							
Double guide rail	35	0[a]	21	7	0[a]	0[a]	28
Concrete posts	184	6[a]	108	19	5[a]	2[a]	139
Single guide rail	5	0[a]	38[a]	26[a]	0[a]	0[a]	64[a]
Guide rail and ditch	4	30	10[a]	0[a]	0[a]	0[a]	41[a]
Subtotal	228	4	62	13	3	1	82
Total	1,552	3	54	15	3	2	76

[a]Number of accidents less than 10 for period of study

The contribution of increase in traffic volume to the all and injury rates (Figs. 5 and 6) for the three sub-groups of the deterring median was determined by use of the same method previously used for adjusting for effect of volume to width of median except that an average volume (AADT) group was used for the study sections in each subgroup and it was assumed that the effect of volume was the same for each type of accident.

Disregarding the effect of volume on accident occurrence, those data showed again that the overtaking accidents predominated, as expected. The approach type of accident appeared to be higher for the median with intermittent shrubbery than for the other types of median which were more or less the same. The rates for the other three types of accident appeared somewhat higher for the curbed median as compared to the others. The total accident occurrence showed the curbed median with the highest contribution, whereas the earth median was lowest.

Table 3 gives the accident pattern and rates by type of median for injury accidents between intersections before adjustment for the effect of volume. Again it was noted that the greatest contribution to the injury accident occurrence was the overtaking type of accident. For the deterring type this contribution is about 62 percent of the total and was more or less the same for each subgroup. For the non-traversable type the overtaking injury type of accident accounted for more than 75 percent of the total frequency and was about the same for each subgroup except the guide rail and ditch which indicated (based on a small number of accidents) that all of the accident occurrence was accounted for by the approach type of accident.

Figure 6 depicts the data in Table 3 for the deterring type of median, adjusted for the effect of volume. Again, it is noted that the overtaking type of injury accident accounted for the highest accident rate averaging about 18 compared to about 3 for the approach, 5 for the single vehicle, and less than 3 for the pedestrian types of accidents. Within the subgroups the curbed median again showed a somewhat higher contribution to overtaking accidents than the other subgroups. In both the approach and single vehicle type of accident, the median with the intermittent shrubbery showed the highest contribution, whereas for the pedestrian type the curbed median frequency was about five times higher than that for the other two subgroups. Again the total accident occurrence showed the earth median with the lowest frequency of occurrence.

Influence of Median on Severity of Accidents

The severity of accidents for the two types of medians is given in Table 4. Using the number of injuries per 100 MVM of travel as an index of severity, it is seen that both the earth and miscellaneous features medians had the smallest contribution to severity (47) in the deterring group. The curbed median was next with a rate of 55. For the non-traversable type the index of severity ranged from 79 for the double guide rail to 108 for the concrete posts. This concrete posts median index was more than twice that for the deterring. It is also higher than the index for any of the other median subgroups.

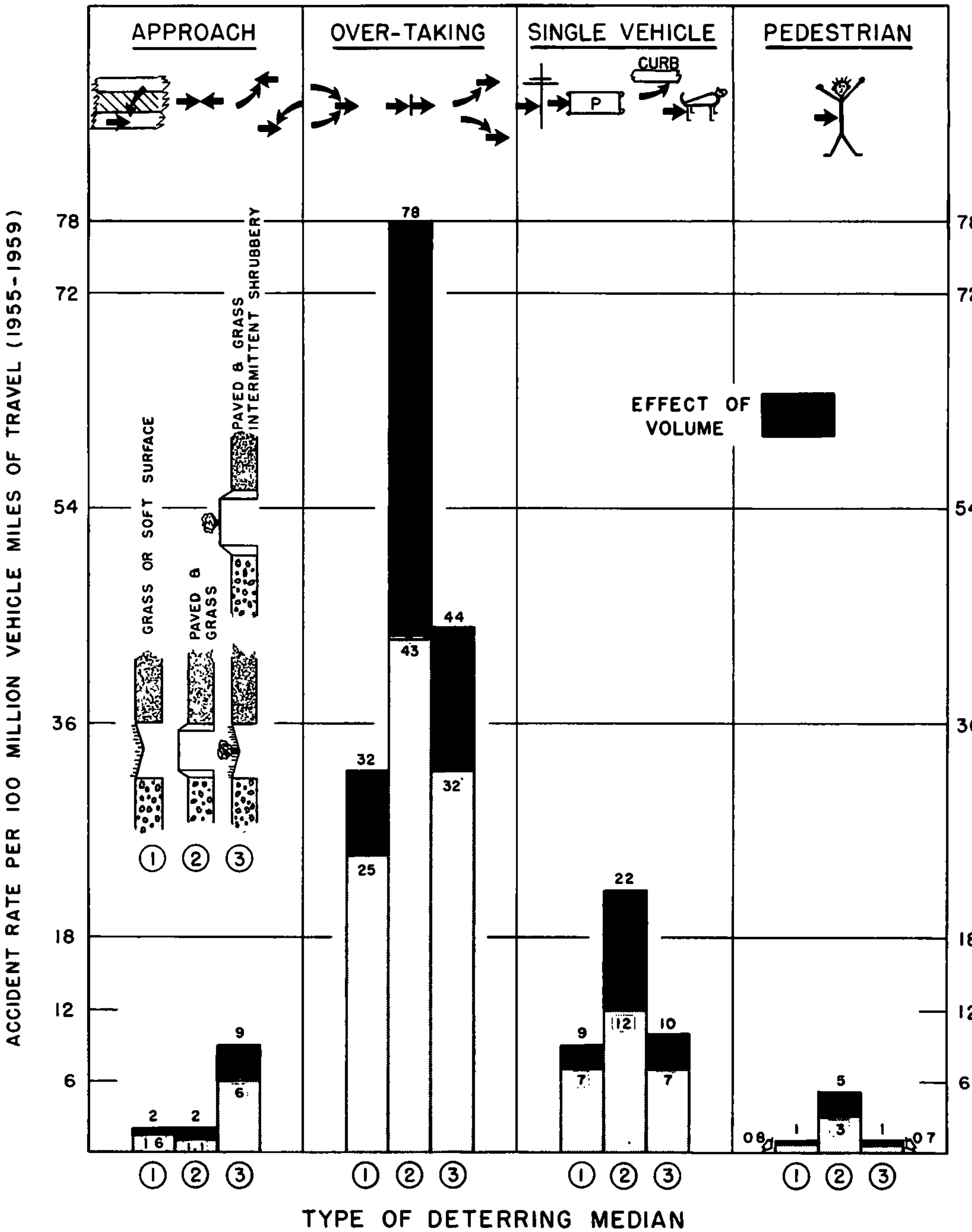


Figure 5. All accident occurrence between intersections on multilane highways with deterring-type median by type of accident.

It is interesting to note that the frequency of the overtaking type of accident for all accidents, subgroup concrete posts (Table 2), and that for injuries for the concrete posts (Table 4) were the same (108). In other words, there was an average of one person injured in each overtaking accident involving the concrete posts positive median. Also, the frequency of total injury accidents (70) from Table 3 for the concrete posts was higher than that for any of the other medians in the non-traversable type.

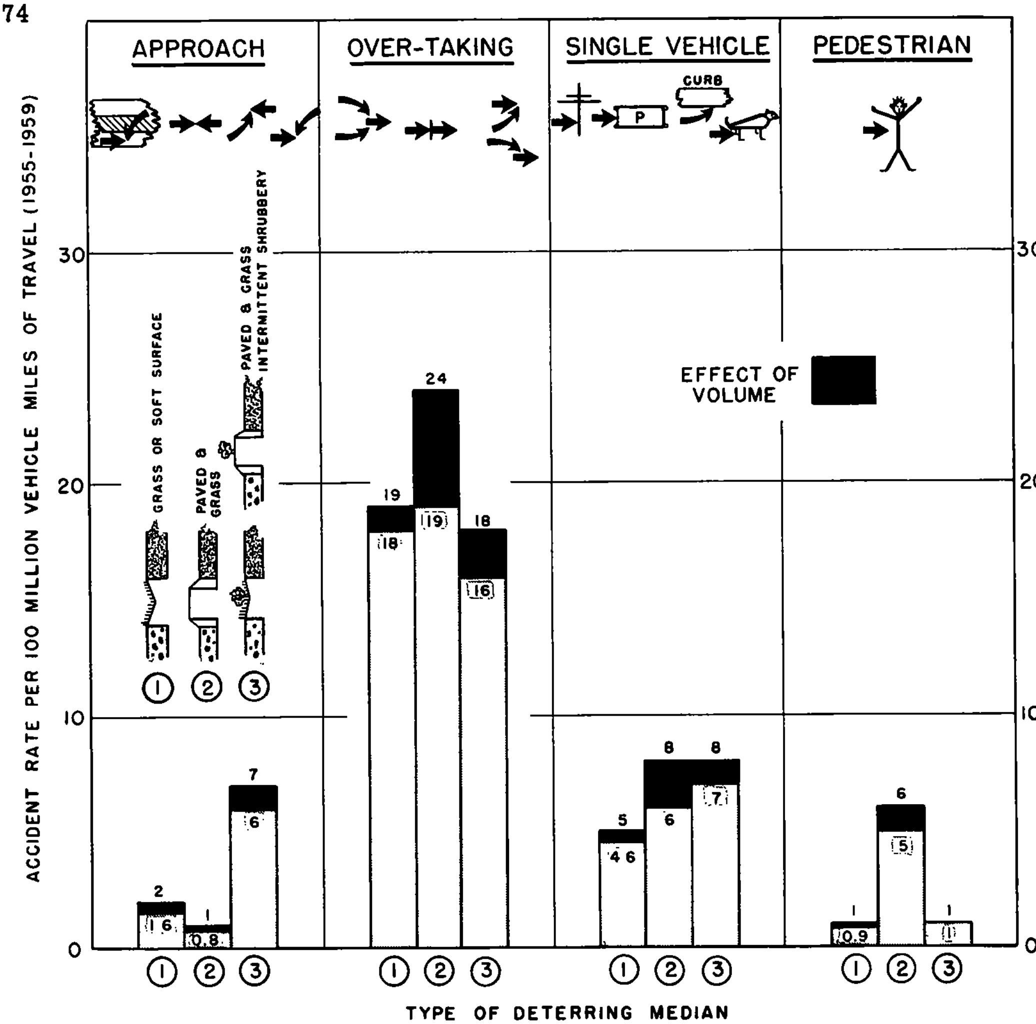


Figure 6. Injury accident occurrence between intersections on multilane highways with deterring-type median by type of accident.

One of the customary yardsticks of accident analysis is the fatality rate per 100 MVM. From Table 4 it will be seen the number of fatalities recorded in this study are too few for meaningful analysis based on the criterion of minimum 10 events. On the other hand, when these fatality rates, of 0.59 based on a 100 percent reporting over a 5-yr period, are compared with the 1959 statewide fatality rates of 4.9, the significance of the divided highway becomes apparent.

It should be remembered that these data are for a volume range from 4,000 to 44,000 vehicles per day and that the effect of volume on injury accident rates for the deterring type can be considered as negligible (Fig. 3). Also, for the non-traversable-type concrete posts the daily volume was about 22,000 and for the double guide rail on the Long Island Expressway it was about 85,000 vehicles. It is perhaps worthy to note that the index of severity for the non-traversable type was about 80 percent (92 vs 51) higher than that for the deterring type.

To further evaluate the operational characteristics of the 10-ft median with and without concrete posts, the following comparison is made. It should be noted that in this comparison all variables are the same with the exception of the concrete posts and the length of the two sections.

Comparison of Accident Rates[a] for the 10-Ft Wide Flush Grass Median With and Without Concrete Posts in the Median for Accidents Between Intersections

Median Type	Length (mi)	Traffic (mvm)[b]	Pers. Injury Accident No.	Rate[b]	Inj.	Rate[b]	Prop. Dam. Accid. No.	Rate[b]	All Accid. No.	Rate[b]
With conc. posts	3.3	132	93	70	142	108	91	68	184	139
W/out conc. posts	7.2	289	143	49	155	54	67	23	210	73

[a]Occurrences per 10^8 vehicle-miles of travel.
[b]Million vehicle-miles.

The all accident rate with concrete posts is about twice that without concrete posts. The property damage frequency of occurrence is about three times higher and the index of severitv is double.

Accidents Involving the Median

A breakdown of accidents involving the median is given in Table 5 for the various median types. It would be expected that the deterring-type medians would have the greater cross median accident rate. However, in this study the rate for the non-traversable type

TABLE 3

ACCIDENT PATTERN AND RATES BY TYPE OF MEDIAN FOR INJURY ACCIDENTS BETWEEN INTERSECTIONS

Type of Median	No All Accidents	Accident Rates Per 100 MVM of Travel: Approach	Overtaking	Single Vehicle	Pedestrian	Other	Total
Deterring							
Earth	250	2	19	5	1	1	28
Curbed	316	1[a]	24	8	6	2	40
Miscellaneous features	31	7[a]	18	8[a]	1[a]	1[a]	35
Subtotal	597	2	21	6	3	2	34
Non-Traversable							
Double guide rail	20	0[a]	12	4[a]	0[a]	0[a]	16
Concrete posts	93	4[a]	55	7	4[a]	1[a]	70
Single guide rail	5	0[a]	38[a]	26[a]	0[a]	0[a]	64[a]
Guide rail and ditch	2	20[a]	0[a]	0[a]	0[a]	0[a]	20[a]
Subtotal	120	3	33	6	2	0 4	43
Total	717	2	23	6	3	1	35

[a]Number of accidents less than 10 for period of study

TABLE 4

FATALITIES AND INJURY RATES BY TYPE OF MEDIAN FOR ACCIDENTS BETWEEN INTERSECTIONS

Type of Median	No. All Accidents: Injury	Fatalities	Accident Rates Per 100 MVM of Travel: Injury Accidents	Injuries	Fatalities
Deterring					
Earth	250	6	28	47	0 67[a]
Curbed	316	3	40	55	0 38[a]
Miscellaneous features	31	1	35	47	1 14[a]
Subtotal	597	10	34	51	0 57
Non-Traversable					
Double guide rail	20	0	16	79	0[a]
Concrete posts	93	2	70	108	1 51[a]
Single guide rail	5	0	64[a]	77[a]	0[a]
Guide rail and ditch	2	0	20[a]	61[a]	0[a]
Subtotal	120	2	43	92	0 72[a]
Total	717	12	35	56	0 59

[a]Number of accidents less than 10 for period of study

was four and less than two for the deterring. Even the frequency of all accidents involving the median was higher (4.7 vs 2.4) for the non-traversable type than the deterring.

TABLE 5

ACCIDENTS INVOLVING THE MEDIAN

Type of Median	MVM	All Cross Median Accidents No	All Cross Median Accidents Rate 100 MVM	All Median Accidents No	All Median Accidents Rate 100 MVM	All Accidents between Intersections No	All Accidents between Intersections Rate 100 MVM	Ratio Median Accidents to Accidents between Intersections	Ratio Cross Median Accidents to All Median Accidents
Deterring									
Earth	893.1	14	1.6	17	1.9	406	45	1 to 24	4 to 5
Curbed	780.7	12	1.5	19	2.4	860	109	1 to 45	3 to 5
Miscellaneous features	87.9	6	6.8	6	6.8	58	64	1 to 10	1 to 1
Subtotal	1,761.7	32[a]	1.8	42	2.4	1,324	75	1 to 32	3 to 4
Non-Traversable									
Double guide rail	127.1	2	1.6	2	1.6	35	28	1 to 17	1 to 1
Concrete posts	132.4	6	4.5	8	6.0	184	138	1 to 23	3 to 4
Single guide rail	7.8	1	12.8	1	12.8	5	64	1 to 5	1 to 1
Guide rail and ditch	9.8	2	20.4	2	20.4	4	41	1 to 2	1 to 1
Subtotal	277.1	11[b]	4.0	13	4.7	228	82	1 to 18	8 to 10
Total	2,038.8	43	2.1	55	2.7	1,552	76	1 to 29	8 to 10

[a]6 of 14, 9 of 12 and 6 of 6 for the 3 groups, respectively, were head-on collisions
[b]All head-on collisions

It is interesting to note the curbed median had the lowest ratio (1 to 45) of median accidents to accidents between intersections, whereas this ratio for the earth median was about twice that of the curbed median. This can be expected because it is well known that many vehicles enter upon the earth median to avoid having a serious accident, whereas the curbed median confines traffic to the pavement lanes and accidents cannot be avoided. This data presents an anomaly wherein the lowest ratio indicates the highest accident occurrence. A comparison of the accident ratio of 45 for the earth median with 109 for the curbed median confirms this.

For the non-traversable type the ratio of median accidents to accidents between intersections was somewhat greater (1 to 18) than that for the deterring type, whereas it appeared that almost all of the median accidents were cross median. It is also to be noted that 21 of the 32 cross median accidents for the deterring type were head-on collisions, whereas for the non-traversable type the 11 cross median accidents for the period of study were all head-on collisions.

Nighttime Accident Rates for Deterring Curbed Medians Lighted vs Unlighted

An attempt was made to evaluated the effect of illumination on night accident rates for the various types of medians. However, the data would only support an analysis for the curbed deterring type of median. These detailed data for six of the curbed median study sections are given in Table 6.

TABLE 6

COMPARISON OF DAYTIME AND NIGHTTIME ACCIDENT RATES FOR ILLUMINATED HIGHWAYS WITH CURBED MEDIANS

Study Section No	Length (mi)	No inter-sections	Median width (ft)	Length of study (yr)	Travel MVM Day	Travel MVM Night	No Accidents Between Intersections Day Fatal	Day PI	Day PD	Day Total	Night Fatal	Night PI	Night PD	Night Total	Total Rates Day	Total Rates Night	Accidents at Intersections No Day	No Night	Rates[a] Day	Rates[a] Night
(a) Illuminated																				
8	1.3	7	40	4.5	37.2	12.4	0	1	1	2	0	1	4	5	5	40	74	33	29	38
27	1.1	13	8	5.0	20.5	6.6	0	1	6	7	0	1	5	6	34	91	228	67	84	78
30	0.7	10	6	3.0	8.0	2.6	0	0	0	0	0	0	2	2	0	77	16	8	20	31
Subtotal	3.1	30	–	4.1[b]	66.0	21.6	0	2	7	9	0	2	11	13	14	60	318	108	18	18
(b) Non-Illuminated																				
17	1.2	8	30	5.0	13.3	4.1	0	8	5	13	0	5	0	5	106	122	13	11	13	34
18	1.1	2	4	2.5	12.2	4.1	0	0	0	0	0	0	0	0	0	0	22	7	90	65
26	0.6	12	6	2.0	6.6	2.3	0	3	2	5	0	1	0	1	74	43	7	9	9	33
Subtotal	3.1	22	–	3.2[b]	31.3	10.5	0	11	7	18	0	6	0	6	58	57	42	27	6	12
Total	6.2	52	–	3.6[b]	97.3	32.1	0	13	14	27	0	8	11	19	28	59	358	135	7	8

[a]Rate for travel per 10^8 vehicle-miles in direction of study per intersection
[b]Average

It will be noted that the total number of accidents between intersections for both day and night on the illuminated sections is small but meets the requirement of ten accidents for complete analysis. Also, the nearly four years of experience provides stabilization of the accident occurrence. Likewise, for the non-illuminated sections, the total of six night accidents and the some three years of experience must be weighed in the interpretation.

These data indicate that the total night accident rate for the illuminated sections of curbed deterring medians is about four times that during the day, whereas for the non-illuminated sections the day and night total accident rates are about the same and are of the same frequency as the night rate at the illuminated sections. From this sample it cannot be said that highway lighting reduces accident rates between intersections on multilane highways with curbed medians.

Looking at the data for accidents at intersections, it is apparent that nighttime illumination reduces the accident frequency to that of daylight. This effect is further emphasized by a comparison of the night and day rates at non-illuminated intersections which is in the ratio of 2 to 1.

Other Analyses

It can be seen that the variety of data collected supports the analysis of many other factors than those related to medians. For instance, in Table 7, the accident data has been arranged to show the yearly trend of accident rate between intersections and at intersections. These data show that the rates of occurrence of injury accidents for

TABLE 7

INJURY ACCIDENTS ALL SITES BY YEAR

	Non-Intersectional Total		Intersectional Total		
Year	No.	Rate[a]	No.	Rate[b]	MVM
1955	74	34	340	0.2358	221.8
1956	130	44	455	0.2352	297.6
1957	109	31	543	0.2343	356.5
1958	173	36	723	0.2318	479.9
1959	219	33	773	0.1767	673.0
Total	705	35	2,834	0.2139	2,038.8

[a]Rate per 100 MVM.
[b]Accidents per year per double intersection (both directions on Expressway) per MVM.

accidents between intersections, as well as accidents at intersections, did not vary significantly from year to year.

Table 8 indicates the accident pattern for the period of study by route including the frequency of accidents between intersections and at the intersections. This type of information may be of interest locally for comparative purposes and is an indication of where the need for enforcement and education is most urgent for the system studied.

Other analyses that are applicable from these data—not included in the study—are available to interested agencies from the New York State Department of Public Works.

TABLE 8

ROUTE ACCIDENT PATTERN FOR PERIOD OF STUDY

Route No	Length (mi)	Control Sections	Day and Night	Accidents Between Intersection Per 100 MVM: Fatal No	Fatal Rate	Injury No	Injury Rate	Property No	Property Rate	Total No	Total Rate	Accidents at Intersection: Fatal	Injury	Property Damage	Total	All Accidents: Fatal	Injury	Property Damage	Total	For Period of Study: Average AADT	Total MVM
24-A	5 7	1	Day	0	0	37	43	48	55	85	98	0	195	143	338	0	232	191	423	27,400	87 0
			Night	0	0	26	30	43	49	69	79	0	124	94	218	0	150	137	287		
			Total	0	0	63	73	91	104	154	177	0	319	237	556	0	382	328	710		
25-B	3 2	2, 3	Day	0	0	28	25	33	30	61	55	1	277	243	521	1	305	276	582	18,050	111 7
			Night	1	1	9	8	16	14	26	23	3	149	113	265	4	158	129	291		
			Total	1	1	37	33	49	44	87	78	4	426	356	786	5	463	405	873		
27-West	8 3	4, 5, 6	Day	1	0 3	82	22	182	49	265	71	5	479	1,144	1,628	6	561	1,326	1,893	28,996	371 5
		7, 8, 9	Night	1	0 3	75	20	127	34	203	55	7	318	610	935	8	393	737	1,138		
		10	Total	2	0 6	157	42	309	83	468	126	12	797	1,754	2,563	14	954	2,063	3,031		
27-East	19 0	11, 12	Day	1	0 2	161	25	125	19	287	45	5	470	395	870	6	631	520	1,157	17,018	642 0
		13	Night	4	0 6	115	18	63	10	182	28	7	241	185	433	11	356	248	615		
			Total	5	0 8	276	43	188	29	469	73	12	711	580	1,303	17	987	768	1,772		
107	2 4	14, 15	Day	1	0 2	6	10	11	18	18	29	1	8	13	22	2	14	24	40	14,070	61 2
			Night	0	0	11	18	12	20	23	38	0	5	4	9	0	16	16	32		
			Total	1	0 2	17	28	23	38	41	67	1	13	17	31	2	30	40	72		
109	3 2	16, 17	Day	0	0	20	31	19	30	39	61	0	49	60	109	0	69	79	148	10,270	64 1
			Night	0	0	13	20	5	8	18	28	2	49	44	95	2	62	49	113		
			Total	0	0	33	51	24	38	57	89	2	98	104	204	2	131	128	261		
110	8 1	18, 19, 20, 21,	Day	0	0	13	12	6	5	19	17	0	56	28	84	0	69	34	103	12,520	110 4
		22	Night	2	2	14	13	6	5	22	20	2	37	20	59	4	51	26	81		
			Total	2	2	27	25	12	10	41	37	2	93	48	143	4	120	60	184		
115	1 8	23, 24	Day	0	0	11	13	15	18	26	31	0	66	56	122	0	77	71	148	18,691	82 6
			Night	0	0	6	7	5	6	11	13	0	36	24	60	0	42	29	71		
			Total	0	0	17	20	20	24	37	44	0	102	80	182	0	119	100	219		
Long Island Express-way	4 3	25	Day	0	0	8	6	8	6	16	13	0	1	0	1	0	9	8	17	85,000	127 1
			Night	0	0	12	10	7	6	19	15	0	0	0	0	0	12	7	19		
			Total	0	0	20	16	15	12	35	28	0	1	0	1	0	21	15	36		
CR-1	0 8	26	Day	0	0	3	33	2	22	5	55	0	2	5	7	0	5	7	12	15,600	9 1
			Night	0	0	1	11	0	0	1	11	0	4	5	9	0	5	5	10		
			Total	0	0	4	44	2	22	6	66	0	6	10	16	0	10	12	22		
CR2-1	1 1	27	Day	0	0	1	3	6	19	7	22	0	68	158	226	0	69	164	233	13,067	31 4
			Night	0	0	1	3	5	16	6	19	2	31	34	67	2	32	39	73		
			Total	0	0	2	6	11	35	13	41	2	99	192	293	2	101	203	306		
CR-2	3 8	28, 29	Day	0	0	10	16	25	39	35	54	0	31	118	149	0	41	143	184	15,326	64 5
		30, 31	Night	0	0	12	19	34	53	46	71	0	13	40	53	0	25	74	99		
			Total	0	0	22	35	59	92	81	125	0	44	158	202	0	66	217	283		
CR-76	14 0	32, 33	Day	0	0	17	11	11	7	28	18	0	48	91	139	0	65	102	167	8,455	157 6
			Night	0	0	6	4	20	13	26	16	1	21	38	60	1	27	58	86		
			Total	0	0	23	15	31	20	54	34	1	69	129	199	1	92	160	253		
CR-80	5 2	34	Day	0	0	0	0	0	0	0	0	0	0	2	2	0	0	2	2	4,160	39 5
			Night	1	3	0	0	0	0	1	3	2	0	2	4	3	0	2	5		
			Total	1	3	0	0	0	0	1	3	2	0	4	6	3	0	4	7		
CR-85	5 3	35	Day	0	0	0	0	0	0	0	0	2	32	55	89	2	32	55	89	10,225	79 1
			Night	0	0	7	9	1	1	8	10	2	24	28	54	2	31	29	62		
			Total	0	0	7	9	1	1	8	10	4	56	83	143	4	63	84	151		
All																				19,923	2,038 8
Day				3	0 15	397	19 47	491	24 08	891	43 70	14	1,782	2,511	4,307	17	2,179	3,002	5,198		
Night				9	0 44	308	15 11	344	16 87	661	32 42	28	1,052	1,241	2,321	37	1,360	1,585	2,982		
Total	85 1			12	0 59	705	34 58	835	40 95	1,552	76 12	42	2,834	3,752	6,628	54	3,539	4,587	8,180		

SUMMARY OF RESULTS

The results of this investigation of the effect of median design on accident rates for divided urban highways carrying traffic volumes up to 44, 000 vehicles per day and having unrestricted roadside access are summarized as follows:

1. There appears to be no correlation between accident rates and width of deterring-type medians between intersections.
2. Accident rates increased linearly with traffic volumes for deterring types of medians between intersections.
3. The overtaking type of accident accounted for more than 70 percent of the accidents between intersections for both deterring and non-traversable types of median. In the deterring-type median, the grass flush median had the lowest rate for all accidents between intersections and the curbed-type median, the highest rate.
4. On highways with deterring curbed medians and without illumination, the night intersection accident rate is twice that of the day rate, whereas on highways with illumination, the night and day rates are the same. On these highways, illumination apparently has no effect in accident rate reduction between intersections.
5. Medians with double beam rails had the lowest all accident and injury accident rates followed by the earth and curbed medians. Medians with concrete posts had the highest accident rates.
6. On highways having the deterring-type median, the curbed median section has nearly 2½ times the accident ratio of the earth median section for all accidents between intersections.

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Effect of Pavement Edge Markings on Traffic Accidents in Kansas

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●RECENT SURVEYS of State Highway Departments indicate wide-spread use of pavement edge markings. Some states have now placed these markings on all major highways. Other states have made steady use of the device on selected highways, based on an anticipated reduction in accidents and fatalities.

Kansas is situated among the latter grouping and beginning in July 1957 placed pavement edge markings on 453 mi of rural highways. No research had been attempted in Kansas prior to this time to determine the effectiveness of this device or of its economic justification. After the initial marking of highway sections, however, "before and after" accident comparisons were compiled. The results of the marking program showed a desirable downward trend on the highway sections so marked.

The study in 1957, included eight study sections totaling 88 mi. The 1958 study included 12 sections totaling 98 mi. Combining both studies showed a 21 percent reduction in total accidents, a 26 percent reduction in the number of personal injuries and a 59 percent reduction in fatalities. In general, most of the highways included in the initial marking program are 2-lane and have bituminous surfaces at least 20 ft wide. It is pointed out that most of the sections had comparatively high initial accident experience rates.

In the fall of 1959, the State Highway Commission in cooperation with the Bureau of Public Roads, undertook a research study of possible motor user benefits to be derived from the use of pavement edge markings. None of the previously marked sections could be used in this research study due to the controlled type of study which was to be undertaken. Therefore, additional highway sections were selected and marked prior to the end of 1959. The objective of this study is to determine what effect these pavement edge markings may have on accident rates.

This report covers the period January 1, 1960, to December 31, 1960.

PROCEDURE

A controlled type of before and after accident comparison study was selected for the survey. Twenty-nine pairs of study sections were selected totaling 384 mi of rural highway that had not been previously edge marked. One-half or 192 mi were selected for the placement of edge markings (marked sections) with the remainder to remain unmarked for the duration of the survey period. The latter are "control" sections, which were employed to cancel out year-to-year variations in accident rates not associated with the edge markings themselves. One section of each pair was selected as the "marked" section with the other being the "control" section and the selection was done by chance, in order to avoid possible bias. The 29 pairs of study sections were distributed throughout the State highway system as shown in Figure 1. It was not possible to include high volume and high accident rate sections in the study because most of these sections had previously been edge marked as part of the regular marking program of the Commission.

SELECTION OF STUDY SECTIONS

Each pair of 29 study sections was located adjacent to each other. The pairs were selected so that traffic and roadway characteristics were uniform in the judgment of the engineer making the selection. In order to maintain study variables to a minimum, the study design characteristics and criteria were established as follows:

1. A bituminous pavement, 20 to 26 ft wide, throughout the study sections.
2. Turf shoulders 1 to 6 ft wide.

3. Traffic volume variation small with a minimum average daily traffic of 1,000 vehicles.
4. Uniform roadside culture in rural areas.
5. A minimum rate of one accident per mile, per year.
6. Total length of each study pair is a minimum of 10 mi with the lengths of the marked and control sections about equal.
7. Sections end at convenient points, such as highway junctions and city limits, to facilitate defining accident location.
8. Centerline and no-passing zone lines were to be in place and maintained in accordance with the Commission's regular marking program.

Initially, about 1,200 mi of highway sections were proposed for inclusion in the study. More detailed investigation showed that many of these sections did not meet the study criteria or were unsuitable for inclusion in the study because of extensive surface repairs, which were undertaken during 1959 or planned for 1960. From this procedure, 29 pairs of marked and control highway study sections totaling 384 mi were selected for inclusion in the study.

Table 1 gives the characteristics of the study sections by the section number shown in Figure 1. It may be seen that the sections are well matched and generally met the established criteria.

Figure 2 shows the study standard for edge marking application. The markings were carried through private driveways but were terminated in advance of intersections with public roads.

Striping of all marked sections was completed prior to December 31, 1959.

Field checking of all marked sections was completed April 27, 1960, to determine where markings had deteriorated by spring edge breakup. Wherever necessary because of line raveling and surface breakup, the edge markings were replaced prior to September 10, 1960, in order that they would retain their maximum effectiveness throughout the study period.

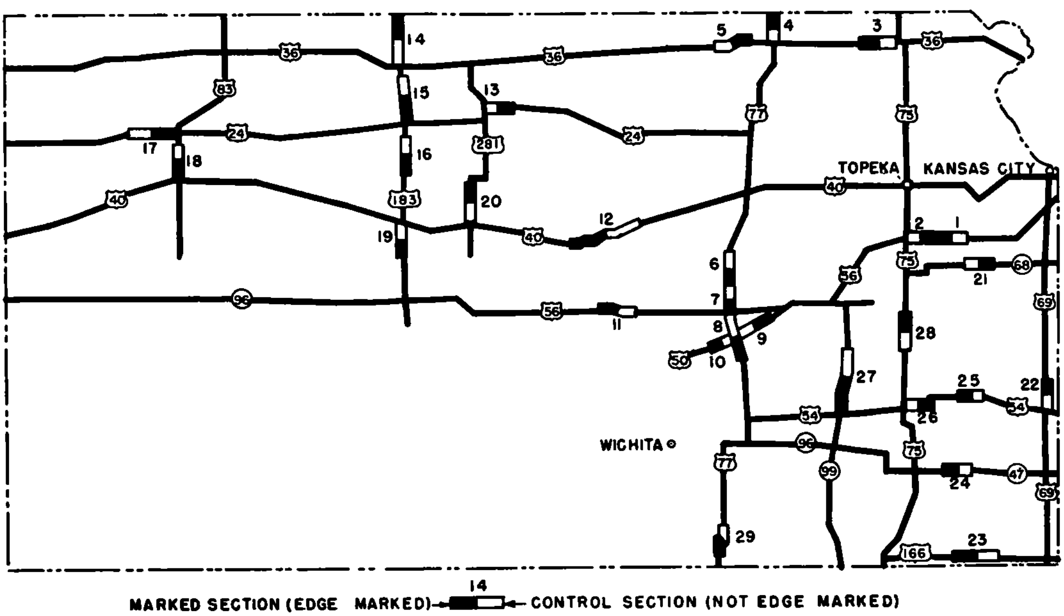

Figure 1. Kansas pavement edge marking study sections.

TABLE 1

SECTION CHARACTERISTICS FOR 2-LANE BITUMINOUS PAVEMENTS WITH TURF SHOULDERS INVOLVED IN STUDY OF PAVEMENT EDGE MARKINGS AND ACCIDENTS IN KANSAS
(Study Period January 1, 1959 to December 31, 1959 and January 1, 1960 to December 31, 1960)

Section No	Highway Length (mi)		1960 ADT (vehicles)		Pavement Width (ft)		Shoulder Width (ft)	
	Marked	Control	Marked	Control	Marked	Control	Marked	Control
1	6 3	6 4	870	950	24	24	2	2
2	5.1	5 1	1,218	1,630	24	24	2	2
3	6 3	6 4	1,543	1,650	24	28	6	2
4	5 3	5 9	980	1,290	26	26	5	5
5	6 5	6 7	2,323	2,585	24	24	5	5
6	5 6	5.6	1,115	1,245	24	24	6	6
7	5.0	5 0	1,373	1,045	22	22	3	3
8	7.6	7.7	995	945	22-24	22-24	4-5	4-5
9	6 6	8.0	1,810	1,985	25	24	5	6
10	5.7	4 9	2,253	1,735	22	22	8	7
11	6.6	6.6	1,695	2,240	26	26	3	3
12	13.3	13.3	1,339	2,163	22-24	22-24	4-6	4-6
13	5 5	5.5	1,523	1,325	26	26	4	4
14	9 0	8 7	558	688	26	26	6	6
15	8 0	8 0	930	785	24	24	4-6	4-6
16	6.5	6.6	1,705	1,250	24	24	5-6	5-6
17	9 0	8.3	2,225	1,918	24	24	6	6
18	5.5	4 7	990	740	22	22	3	3
19	6 0	5 4	1,055	1,883	22	22	4	4
20	6 8	6.8	700	1,080	24	24	5	5
21	5.2	5.1	823	1,045	22	22	4	4
22	5 3	5.3	2,095	2,625	20	20	2-3	2-3
23	8 2	8 2	1,768	1,528	20-24	20-24	4-6	4-6
24	5 5	5 5	730	750	22-24	22-24	3-6	3-6
25	4.9	4.5	2,165	2,300	21	21	2	2
26	5.0	5.1	1,218	1,615	25	25	2	2
27	9.6	10.7	710	1,020	30	24	3	2
28	6.2	5.7	1,360	1,618	24	24	2	2
29	6.3	6.3	3,575	3,223	22	22	4-6	4-6
Total	192.4	192.0	41,644	44,856				

ACCIDENT REPORTING DATA

With reference to accident reporting, Table 2 indicates an increasing level of accident reporting for the State of Kansas, beginning on January 1, 1958, since the enactment of the Safety Responsibility Act. This increased rate of accident reporting does support the completeness of the accident data used in this survey.

ACCIDENT ANALYSIS

Analyses were made of all reported accidents on each test and control section both before and after edge marking. In this case, the "before" period was the year 1959 and the "after" period was the year 1960.

The accident reports were summarized by number of persons injured and number of persons killed, location, type of collision, light conditions, pavement condition and property damage.

The statistical analysis of the total accident data, compiled and listed in Table 3, shows a 1 percent net increase in over-all accident potential between the two types of study sections. This net increase was computed as follows:

The control sections showed a decrease of 13.5 percent in the number of accidents between the before (1959) and after (1960) periods: $\frac{200-173}{200} \times 100$. If the marked sections had not been treated with edge marking, it may have been expected that the edge marked sections, also, would have shown a decrease of 13.5 percent to 144 accidents: 166 - (166 x .135). The difference between the anticipated total number of accidents on marked sections (144) and the actual number (146) is two accidents or a 1 percent net increase: $\frac{146-144}{146} \times 100$. A chi-square reliability test of the sample data taken

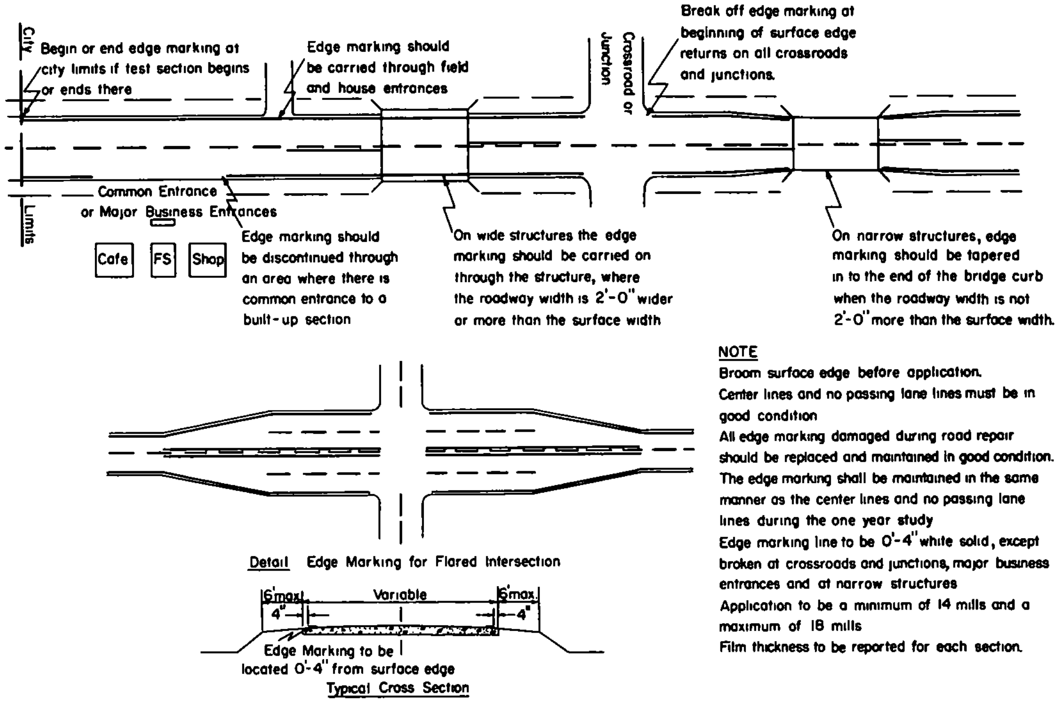

Figure 2. Standard for edge marking application.

separately, shows a significance level greater than 0.25 indicating, as is obvious, that the 1 percent net increase is not statistically significant.

Neither the total number of persons killed and injured nore the number of persons injured (when analyzed separately) was significantly changed after edge marking (Table 4). The number of persons killed, however, showed a net reduction, significant at the 0.07 level. In reviewing the benefits of the edge marking program, certain accident categories were affected to a large and significant degree and these are discussed in detail.

Accidents at Access Points Reduced

As shown in Table 5 and Figure 3, there was a 46 percent net decrease in the number of accidents at intersections and driveways after edge marking of 2-lane rural State highways with a corresponding significance level of 0.04.

Using a marked vs control comparison of accidents not located at intersections and driveways, a 27 percent net increase in accidents in 1960 is found, compared to the same period in 1959 as shown in Figure 3. This net change has a low level of significance, however, 0.17. In other words the net increase of 27 percent could easily have occurred by chance alone.

These findings are consistent with a recent study reported by Musick (1) for Ohio highways. In the latter study, a net reduction of 63 percent in accidents at access points was reported—statistically significant at the 0.01 level. The number of accidents occurring between access points was not significantly changed.

Further analyses of both the Ohio study and the present one showed that these comparisons are similar for both day and night conditions.

To explain these findings, it has been suggested that pavement edge markings encourage drivers to look farther ahead and thus become aware of vehicles about to enter or leave the highway at points of access. Another explanation is that the gap in edge markings at intersections makes drivers aware that there is an intersection ahead. Carefully planned research is needed to test one or preferably both of these theories.

TABLE 2

ACCIDENT REPORTING DATA BEFORE AND AFTER PASSAGE OF SAFETY RESPONSIBILITY ACT IN KANSAS

Year	No. of Statewide Accidents	No. of Fatalities	Ratio of Fatalities to Accidents
1956	25,435	683	1 to 37
1957	26,481	585	1 to 45
1958[a]	45,080	554	1 to 81
1959	46,173	367	1 to 81
1960	40,044	512	1 to 78

[a]Safety responsibility act effective January 1, 1958.

TABLE 3

TOTAL NUMBER OF ACCIDENTS BEFORE AND AFTER PAVEMENT EDGE MARKING OF TWO-LANE RURAL STATE HIGHWAYS IN KANSAS

Section	No of Total Accidents		No of Total Accidents Anticipated After Marking	Net Change (%)	Significance Level
	1959	1960			
Marked	166	146	144	+1	0 25+
Control	200	173	—	—	—

Other Comparisons

Table 6 summarizes several other comparisons. The various types of collision showed no significant change in accidents except for turn collisions and "other" or miscellaneous types.

Figure 4 and Table 6 show that daytime accidents were reduced 18 percent and night accidents increased 42 percent. Neither net change was statistically significant, however.

A more detailed analysis of the two light conditions by the three types of surface condition showed no significant change for any combination except that night accidents on dry pavements increased 72 percent with a significance level of 0.08.

Analysis of combinations of various light condition by various types of collision showed few significant changes. The net decrease in turn collisions was due entirely to daylight data, and a 67 percent reduction in turn collisions during daytime conditions occurred, significant at the 0.04 level. Turn collisions at night were not significantly changed. The increase in "other collisions" was due principally to night data. In addition, fixed object collisions increase significantly (0.04 level) at night.

As shown in Table 7, a net increase in amount of property damage of 44 percent or nearly $50,000 resulted after pavement edge markings.

TABLE 4

TOTAL FATALITIES AND INJURIES BEFORE AND AFTER PAVEMENT EDGE MARKINGS OF TWO-LANE RURAL STATE HIGHWAYS IN KANSAS

Section	No. of Total Fatalities and Injuries		No. of Total Fatalities and Injuries Anticipated After Marking	Net Change (%)	Significance Level
	1959	1960			
Marked					
Killed	5	4	18	78	0.07
Injured	105	100	77	+30	0.25+
Total	110	104	90	+16	0.25+
Control					
Killed	4	15	-	-	-
Injured	136	100	-	-	-
Total	140	115	-	-	-

The substantial increase in property damage on the marked sections bears some explanation. This increase was caused by four accidents. Two accidents involved trains and large trucks at railroad crossings, and two involved large truck accidents. These four accidents caused a total of $69,050 property damage. If the

TABLE 5

TOTAL NUMBER OF ACCIDENTS AT INTERSECTIONS AND DRIVEWAYS BEFORE AND AFTER PAVEMENT EDGE MARKINGS OF TWO-LANE RURAL STATE HIGHWAYS IN KANSAS

Section	No of Total Accidents 1959	1960	No of Total Accidents Anticipated After Marking	Net Change (%)	Significance Level
Marked	52	38	70	-46	0 04
Control	41	55	-	-	-

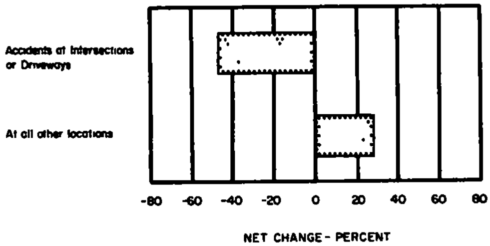

Figure 3. Net change in accidents by location after edge marking of two-lane rural State highways in Kansas.

TABLE 6

NET CHANGE IN ACCIDENTS AFTER EDGE MARKING OF TWO-LANE RURAL HIGHWAYS IN KANSAS SUBDIVIDED BY LOCATION, TYPE OF COLLISION, LIGHT CONDITION AND WEATHER

Item	Net Change (%)	No. of Accidents[a]	Significance Level[b]
Total Accidents	+1	685	0.25+
Persons Killed and Injured	+16	469[c]	0.25+
Location			
At Access Points	-46	186	0.04
Between Access Points	+27	499	0.17
Type of Collision			
Pedestrian	d	1	0.25+
Turn	-60	72	0.06
Angle	-58	28	0.25+
Rear-end	-31	151	0.25+
Head-on	-11	27	0.25+
Sideswipe	-6	50	0.25+
Other collision	+326	33	0.03
Non-collision	+6	22	0.25+
Fixed objects	+6	44	0.20
Run off road	+34	257	0.23
Light condition			
Day	-18	421	0.25+
Night	+42	264	0.15
Pavement Condition			
Dry	+4	471	0.25+
Wet	0	119	0.25+
Ice	+3	95	0.25+

[a]Refers to the total sample and includes both edge marked and control sections for the year before and the year after edge marking.
[b]Indicates the probability that the net change could have occurred merely by chance. A significance level of 0.04; for example, indicates that there are only four chances in 100 that a "net change" as great or greater than that shown could have occurred merely by chance.
[c]Refers to number of persons killed and injured.
[d]Total sample is too small to warrant computing net change.

TABLE 7

TOTAL PROPERTY DAMAGE BEFORE AND AFTER PAVEMENT EDGE MARKING OF TWO-LANE RURAL STATE HIGHWAYS IN KANSAS

Section	Total Property Damage ($) 1959	Total Property Damage ($) 1960	Total Property Damage Anticipated After Marking($)	Net Change (%)
Marked	103,700	159,900	111,100	+44
Control	112,300	120,300	-	-

costs of these four accidents conformed more nearly to the average costs of all accidents involved in the study, there probably would be only a slight increase in property damage costs on the marked sections over the control sections in 1960.

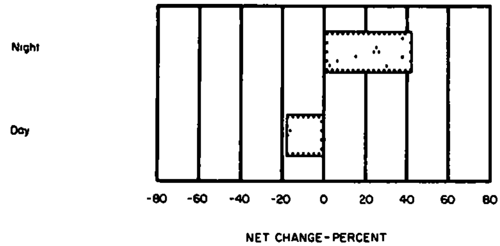

Figure 4. Net changes in accidents during daylight and darkness hours after edge marking of two-lane rural State highways in Kansas.

CONCLUSIONS

The significant conclusions from this study are:

1. On two-lane rural State highways in Kansas, the use of pavement edge markings resulted in a reduction in the number of fatalities.
2. There was no significant change in number of persons injured or in total number of accidents.
3. Accidents at intersections and driveways were significantly reduced during both daytime and nighttime conditions. Accidents between access points were not significantly changed.
4. The turning collisions associated with access points were reduced during daytime conditions.

ACKNOWLEDGMENT

This study was conducted by the Traffic and Safety Department of the State Highway Commission of Kansas with the cooperation of the Bureau of Public Roads. The procedure and analysis for this survey were developed in collaboration with David Solomon, Highway Research Engineer and R. V. White, Programing and Planning Engineer, Bureau of Public Roads. The survey work and compilations progressed under the supervision of James R. Preisner, Traffic and Safety Department, State Highway Commission of Kansas.

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A Theory of Traffic Flow for Evaluation of Geometric Aspects of Highways

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● IN RECENT YEARS several theories have been proposed wherein the car-following characteristics of a traffic stream have been developed from a variety of considerations such as continuous fluid flow (1), "follow-the-leader" philosophies (2, 3), and other concepts (4). The purpose of these analyses was to obtain a mathematical description of highway capacity as reflected by flow-density relationships. Of interest in this paper is the development of an analytical approach whereby the effects of various aspects of a highway (such as curves, grades, lane width, traffic control devices, type and condition of surfacing) can be evaluated quantitatively under varying traffic density and weather conditions and hence provide rational basis for geometric design procedures.

Unlike previous work, where the pattern of traffic flow (for example, single-lane, congested conditions) is assumed a priori, in the present analysis it is postulated that traffic reacts to a motivating "pressure potential"; that is, the vehicle operator reacts to the prevailing road, traffic, and weather conditions—characterized herein by the pressure potential—in a pattern that determines the behavior of any particular vehicle traversing a given section of roadway. This concept can be expressed quantitatively in a manner analogous to that used in describing one-dimensional viscous flow of a compressible fluid. Solution of the resulting differential equation yields a parameter which is a numerical measure of the salient characteristics of a road. Procedures are then proposed to determine this parameter, using measurable vehicular velocities, which eliminate the need for evaluating the motivating pressure potential.

Application of the new theory in the development of rational procedures for the geometric design of highways is illustrated by suitable examples.

NOTATION

The following notation is used throughout the paper:

- p = motivating pressure potential;
- x = a position along a roadway;
- L = a particular length of roadway;
- N = number of vehicles;
- N_x = number of vehicles entering a section of roadway at position k;
- v = velocity of vehicle;
- w = effective width of roadway;
- ρ = vehicular density, number of vehicles per unit of effective area,
- t = time;
- C_1, C_2 = coefficients of proportionality;
- k = coefficient of proportionality; and
- a^2 = $(C_1 + C_2)/k$ = a constant.

FORMULATION OF PROBLEM

The theoretical development is based on the following assumptions:

The vehicular velocity is proportional to the gradient of the pressure potential,

$$v = -k\frac{\partial p}{\partial x} \qquad (1)$$

Making use of Eq. (1), and noting that the quantity $\left[\frac{\partial p}{\partial x}\right]^2$ is of higher order,

$$\frac{\partial^2 p}{\partial x^2} = a^2 \frac{\partial p}{\partial t} \tag{7}$$

in which, $a^2 = (C_1 + C_2)/k$ = a constant. Eq. 7 is the governing differential equation of the traffic flow process and provides the basis for a quantitative evaluation of salient features of a highway. As the variation in pressure potential with distance and time cannot be measured directly, it is necessary to obviate this requirement if the theory is to be of practical utility. The method adopted to accomplish this end depends on the particular features selected for study, which can best be illustrated by examples.

APPLICATIONS

Consider a section of highway of length L containing some geometrical aspect to be evaluated quantitatively. This may be a grade or a curve (bridge, traffic control device, etc.) to be studied over a range of traffic volumes. To solve Eq. 7 a suitable set of boundary and initial conditions must be selected. These will depend on the situation being studied; however, for the purpose of illustrating the procedure, the following simplified set of initial and boundary conditions will be assumed:

$$\begin{aligned} p(x,0) &= p_0 \\ p(L,t) &= p_1 \\ p(0,t) &= p_0 \end{aligned} \tag{8}$$

For the conditions given by Eqs. 8, the complete solution of Eq. 7 is

$$p(x,t) = p_0 + \frac{\Delta p}{L} x - \frac{2}{\pi} \Delta p \sum_{n=1}^{\infty} \frac{(-1)^{n-1}}{n} e^{-n^2 F_0} \sin \frac{n\pi x}{L} \tag{9}$$

in which $\Delta p = p_1 - p_0$ and $F_0 = \pi^2 t/a^2 L^2$.

Recalling that $v = -K\, \partial p/\partial x$, Eq. 9 becomes

$$\frac{-v(x,t)}{\frac{k\Delta p}{L}} = 1 - 2 \sum_{n=1}^{\infty} (-1)^{n-1} e^{-n^2 F_0} \cos \frac{n\pi x}{L} \tag{10}$$

which, noting that $v_{avg} = -k\,\Delta p/L$, can be expressed as

$$\frac{v(x,t)}{v_{avg}} = 1 - 2 \sum_{n=1}^{\infty} (-1)^{n-1} e^{-n^2 F_0} \cos \frac{n\pi x}{L} \tag{11}$$

If the instantaneous velocity is measured at some point on the roadway, say at $x = L/2$, Eq. 11 gives the relation

$$\frac{v(\frac{L}{2},t)}{v_{avg}} = 1 - 2 \sum_{m=1}^{\infty} (-1)^{m+1} e^{-4m^2 F_0} \tag{12}$$

which is seen to be a function of F_0 only. Eqs. 11 and 12 can be evaluated once and for all and either tabulated or represented graphically. For example, a plot of Eq. 12 is given in Figure 1 with the parameter $F = \left[F_0\right]^{-1}$. The ratio of the instantaneous velocity at a particular point on a section of roadway to the average of the velocity over the entire section defines a unique F-value that is a numerical rating of the particular geometric aspect being studied. Both of these velocities can be measured directly with present-day devices (5).

The change in vehicular density with respect to the pressure potential is proportional to the density,

$$\frac{\partial \rho}{\partial P} = C_1 \rho \tag{2}$$

from Eq. 2,

$$\frac{1}{\rho}\frac{\partial \rho}{\partial t} = C_1 \frac{\partial p}{\partial t} \tag{2a}$$

and

$$\frac{1}{\rho}\frac{\partial \rho}{\partial x} = C_1 \frac{\partial p}{\partial x} \tag{2b}$$

The change in effective width of the roadway with respect to the pressure potential is proportional to the effective width,

$$\frac{\partial w}{\partial p} = C_2 w \tag{3}$$

From Eq. 3,

$$\frac{1}{w}\frac{\partial w}{\partial t} = C_2 \frac{\partial p}{\partial t} \tag{3a}$$

and

$$\frac{1}{w}\frac{\partial w}{\partial x} = C_2 \frac{\partial p}{\partial x} \tag{3b}$$

In the ensuing theoretical development it is assumed that neither access nor egress is available to the section of highway under study. Thus the traffic flow can be considered as a conserved system. It should be noted, however, that the theory can be extended to account for the effects of access and (or) egress provided the number of vehicles entering and (or) leaving the section of roadway is determined.

THEORY

Considering the traffic flow as a conserved system, the change in the number of vehicles on a length of road, dx, in an interval of time, dt, must equal the difference between the number of vehicles entering the section at position x and the number of vehicles leaving the section at position x + dx (in time, dt). If N is the number of vehicles initially on the length of roadway dx, ($N = \rho w dx$), and N_x is the number of vehicles entering in time, dt, at position x, ($N_x = \rho w v dt$), the preceding statement can be expressed symbolically by

$$\frac{\partial(\rho w)}{\partial t} dt dx = \rho w v dt - \left[\rho w v dt + \frac{\partial(\rho w v)}{\partial x} dx dt\right]$$

or

$$\frac{\partial(\rho w)}{\partial t} + \frac{\partial(\rho w v)}{\partial x} = 0 \tag{4}$$

Note: If N_E equals the net number of vehicles entering and leaving per unit length of roadway, dx, per unit time, dt, Eq. 4 becomes

$$\frac{\partial(\rho w)}{\partial t} + \frac{\partial(\rho v w)}{\partial x} = N_E$$

Expanding Eq. 4 and dividing by ρw

$$\frac{\partial v}{\partial x} + \frac{v}{\rho}\frac{\partial \rho}{\partial x} + \frac{v}{w}\frac{\partial w}{\partial x} + \frac{1}{w}\frac{\partial w}{\partial t} + \frac{1}{\rho}\frac{\partial \rho}{\partial t} = 0 \tag{5}$$

which upon introduction of Eqs. 2a, 2b, 3a and 3b, reduces to

$$\frac{\partial v}{\partial x} + (C_1 + C_2)\, v \frac{\partial p}{\partial x} + (C_1 + C_2) \frac{\partial p}{\partial t} = 0 \tag{6}$$

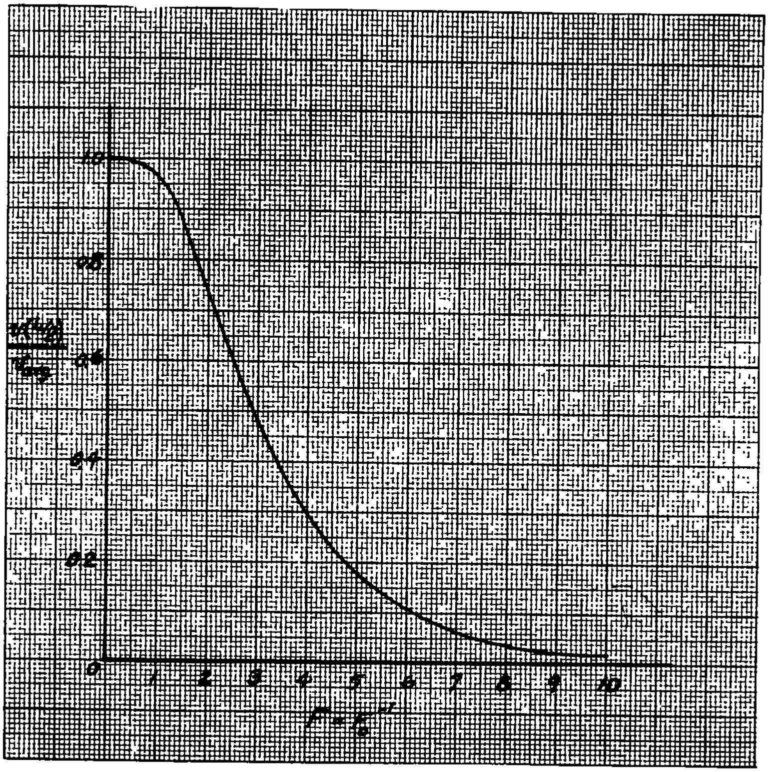

Figure 1. Solution of Eq. 12

By repeating the velocity measurements on the same section of roadway under different ambient conditions (such as minimum and peak traffic densities during the day or night, or different climatic conditions), the resulting changes in the F-value will provide a numerical rating of the particular geometric feature being studied as influenced by the changes in ambient conditions.

By making simultaneous measurements on two sections of the same road, each having a different geometrical feature (such as a grade vs a curve), comparisons of the effects of these two (or any other) features on the resistance to traffic flow can be made on a quantitative basis. A rational answer can them be given to the time-honored question: is a G percent grade more (or less) objectionable than a D degree curve? Repeating the simultaneous measurements at various times will permit making such comparisons under different ambient conditions.

Similarly, velocity measurements before and after traffic control devices are installed, or the road is widened, or an overpass, interchange, acceleration and/or deceleration lane is constructed will permit evaluation of the effectiveness of these devices on a numerical basis. Indeed, the number of potential applications of the new theory appears to be unlimited.

In time, a catalog of F-values can be developed for all types of geometrical and psychological highway appurtenances which will permit future highways to be designed on a more rational basis.

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